

BUILDING INTEGRATION SOLUTIONS

PROCEEDINGS OF THE 2008 ARCHITECTURAL ENGINEERING
NATIONAL CONFERENCE

September 24–27, 2008
Denver, Colorado

SPONSORED BY
The Architectural Engineering Institute (AEI)
of the American Society of Civil Engineers

EDITED BY
Mohammed Ettouney, Ph.D., P.E.

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ISBN: 978-0-7844-1002-8

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Preface

There are many types and flavors of buildings which include residential, office, health care, government, as well as many others. Advancing the performance of buildings at reasonable costs has been perhaps one of the most important goals of human ingenuity. This is very well evidenced by the evolution of buildings since the dawn of time until now. Here, at the Architectural Engineering Institute (AEI) of the American Society of Civil Engineers (ASCE) we are proud that our mission is to promote an integrated, multi-disciplinary approach to the planning, design, construction and operation of buildings. As such, we feel that AEI is at the forefront of enhancing the very quality of human experience at reasonable costs.

One of the strategies that AEI uses to achieve such a mission is a two-dimensional strategy. First, AEI tries to incorporate several stakeholders in the building community such as architects, builders, construction managers, and civil, structural, mechanical, electrical engineers, as well as specialty engineers such as fire protection, acoustical, control systems, geotechnical, and environmental engineers. Second, AEI tries to address the many technical subjects that relate to the building community including aesthetics, functionality, performance, life safety, security efficiency, economy, and sustainability. The smooth, efficient and clear integration of the stakeholders and the technical issues is the roadmap to achieving the above-mentioned goal.

To that end, the 2008 Architectural Engineering Conference was held in Denver, Colorado from September 25 to September 27. The sixty-seven papers in this conference Proceedings cover a wide range of topics of interest to the architectural engineering community. The wealth of topics reflects both the breadth of the Architectural Engineering discipline as well as its importance to everyday safety, efficiency, optimal cost-benefit and comfort of buildings as an integral symbol of modern societies. Topics covered in the Proceedings include: Building Information Modeling (BIM), Multi-hazard Designs, Forensics, Construction Management, Design, Structural Behavior, Building Envelope, Structural Glazing, Noise and Vibration Control in Buildings, Building Safety and Security, Efficiency, Comfort, Planning, Facilities Management, Sustainability, and Life Cycle Cost Analysis. The Proceedings reflects the goal of the Architectural Engineering Institute which is the integration of engineered systems within buildings, thus ensuring building safety, security, and comfort efficiently and at a reasonable cost.

The Conference owes its success to the contributions of many individuals.

The authors provided much of the technical content of the Conference. The Reviewers (Gary Brown, P.E., Nanci Buscemi, P.E., Scott Campbell, Ph.D., P.E.,

Amar Chaker, Ph.D., Kenna Chapin, P.E., Adam Hapij, P.E., Richard Klinger, Ph.D., P.E., Ali Memari, Ph.D., P.E., Kevin Parfitt, P.E., James Ruggieri, P.E., David Weggel, Ph.D., P.E.) ensured the quality of the Proceedings through their expertise. Dr. Amar A. Chaker, Director of the Architectural Engineering Institute, provided extensive technical assistance and logistical support. Lucy King and Megan DiGerlando of ASCE provided timely assistance. The efforts of the Session Moderators and Workshop Instructors contributed to the success of the Conference. The Steering Committee provided advice and guidance in many critical phases including the planning, reviewing, program development, student activities, career fair, and sponsorship. The editor expresses his sincere appreciation to all of these dedicated individuals.

This Conference would not have been possible without the financial support of the many generous Sponsors whose contributions are gratefully acknowledged.

Editor

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A Team Approach to
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Abstract

The role of university educators and administrators is to develop the most complete student possible through the educational process. This development should include intellectual competence, interpersonal competence, and the ability to create mature interpersonal relationships. Typically, engineering curricula have focused primarily only on the development of intellectual competence, therefore not addressing all student needs. The incorporation of problem-based learning and the use of teams in Architectural Engineering capstone courses provides an environment in which all of these development objectives can be achieved as well as an opportunity to teach problem-solving and team skills. When implemented in engineering capstone courses, problem-based learning can satisfy the developmental needs of students, the demands of employers hiring our students following graduation, as well as the criteria defined by the Accreditation Board for Engineering and Technology (ABET).

Introduction

The intent of this paper is to introduce a method of teaching upper level engineering coursework, especially capstone classes, that will not only enhance the students' technical competence but also prepare them more for the professional world. This paper is written for university administrators influencing the learning outcomes for departments of engineering and professors of these capstone courses. Traditionally engineering curriculums focus on technical content and individual work although it is being discovered that this alone is not what is needed for the students to be successful as a professional nor does it match the demands of today's employers. The teaching method introduced in this paper to satisfy these professional and career demands has two primary components. First, it is important that the students be required to work in teams during their college career because most engineering design in the practicing world is not an individual task but a team effort. For the purposes of this paper the term team will refer to a group of four to five students that are assigned to work together on a series of tasks for an entire semester. It is important to make a clarification between a group and a team since a group is typically self-selected by the students and are assigned short-term projects. Groups therefore require different instructional strategies and encounter different complications than teams. Secondly, the task to be completed as a team should be as practical and application-based as possible thus introducing problem-based learning (PBL) as an ideal method of instruction.

The purpose of this paper is to convince those that influence the engineering curriculum and teaching methods that changing to PBL with a team approach is in the best interest of the department and the students. Many may ask why an alternative method from those traditionally used such as this should be considered. There are a number of reasons to consider change which will be discussed in the following sections. Critical components needed for successful use of PBL in a team format will also be outlined to better assist those considering its implementation in their classroom.

Problem-Based Learning

PBL is a teaching method that utilizes complex real world problems to motivate students to learn and explore concepts and principles needed to create solutions (Duch, Groh, & Allen, 2001). "The more formal classroom experience can be made immeasurably more interesting if the work is presented in the context of real world problems, with multiple solutions possible and multiple disciplines engaged" (Fenster, 2002, p. 3). This process is especially effective if implemented as a team exercise so that the students work together maximizing their own and each other's learning (Smith, 1995). "Students are assigned to work together and, given the complexity of the task and the necessity for diverse perspectives, they are relieved to do so. They know that their success depends on the efforts of all group members" (Smith, 1995, p. 2b5.14). This method of teaching most closely replicates the process of questioning, learning, and solving used in the professional environment (Duch, Groh, & Allen, 2001). "Problem-based learning is very suitable for engineering because it helps students develop skills and confidence for formulating problems they've never seen before. This is an important skill, since few professional engineers

are paid to formulate and solve problems that follow from the material presented in the chapter or have a single “right” answer that one can find at the end of a book” (Smith, 1995, p. 2b5.16). Not only does this format provide the student an experience replicating their future career, but it can also create an environment that encourages many significant elements for growth and development as listed below (Duch, Groh, & Allen, 2001):

- Critical thinking and application
- Evaluation and use of appropriate learning resources
- Interpersonal competence – listening, cooperating, and communicating
- Continuous learning skills
- Positive interdependence
- Appreciation and tolerance of differences (Duch, Groh, & Allen, 2001)

Most people in the university environment would agree that all of these items are considered to be important areas of growth for college students. These are many of the skills that the university aims to enhance when considering Chickering’s first vector of human development – developing competence (Evans, Forney, & Guido-DiBrito, 1998). The fact that PBL can incorporate all of these in one course is a strong argument for its implementation in every discipline of engineering. Each of these growth and development items will be addressed in greater detail in this paper.

PBL has the advantage over other more traditional teaching methods in engineering such as the lecture format because of the critical thinking that is required. “In contrast to memorization, thinking is an intellectual activity in which interaction between people can be particularly valuable. Whether the skill is critical thinking, practical thinking, or creative thinking, learning how to incorporate ideas and perspectives of several people and learning how to work through differences can greatly enhance each student’s own ability to think effectively” (Michaelsen, Knight, & Fink, 2002, p. 23). The focus of this method of teaching is on thought or solution process rather on facts or particulars of a particular specialization. The reason this is important is because the specifics of a discipline, especially in a technical field of study such as engineering, obsolete at a very rapid rate while the solution process will remain relatively the same forever (Fenster, 2002). As stated by Fenster in the book *Engineering Education and Research*, “The strength of the engineering disciplines, whatever their specializations, lies in their integrative character, their focus on identifying, describing, and seeking various approaches to problem solving. Students ideally will understand that there may be many solutions, that the solutions may vary with time and circumstance, and that the road to solutions involves many variables and usually factors involving other disciplines” (2002, p. 3). Students recognize the need to support their choices and opinions since there are many correct approaches. The key to this level of development is the student must justify why they have chosen to apply or pursue a particular choice for a given application, recognizing that it is contextually defined and based on evidence provided or assumed (Evans, Forney, & Guido-DiBrito, 1998). The team component is also valuable when considering critical thinking. In this team environment the student are “learning how to incorporate ideas and perspectives of several people and learn how to work through differences greatly enhancing each student’s own ability to think effectively” (Michaelsen, Knight, & Fink, 2002, p. 23). These critical thinking skills will not

become outdated, but rather can be applied to each new problem encountered both as a student and a professional.

The team function of PBL is very important to the development of technical competence but is equally important to development of interpersonal competence. “*Engineering Education for a Changing World*, a joint report of the Engineering Deans Council and the Corporate Round Table of the American Society for Engineering Education, has asked colleges of engineering to accelerate implementation on programs that help engineering undergraduates develop softer skills to meet the challenges of the 21st Century. These softer skills include team skills such as collaborative active learning; communication skills; leadership; an understanding and appreciation for diversity of students, faculty, and staff; and understanding of societal, economic, and environmental impacts of engineering decisions” (Tsang, 2000, p. 1). This skill set has been further reinforced by the Accreditation Board for Engineering and Technology (ABET) which accredits university engineering colleges and programs of minimum levels of educational competence. Engineering colleges and disciplines around the country strive to attain and maintain their accreditation with ABET in order to ensure that their graduates will be able to take the Professional Engineering Exam. In recent years ABET has set criteria that requires that “engineering programs demonstrate that their graduates have among other competencies, an ability to function on multidisciplinary teams and an ability to communicate effectively” (Tsang, 2000, p. 2). ABET has implemented this new criteria recognizing that one of the primary tasks of an engineering curriculum is to prepare the graduates for employment in the professional world.

Although the thought of adding team skills to the engineering curriculum is recognized as important it is not done as frequently as one would hope for several reasons. The first being that the stereotypes of engineers being anti-social and content with their computer and calculators exists for a reason. The more common personalities of engineers lend credence to these stereotypes. As stated by Jennifer Moffat, “There is no silence quite so pregnant as the one when engineers, both faculty and students alike, are asked to talk about how they feel. This is not all together unexpected. Engineers and physical scientists are trained to take self out of the problem solving process. Analysis must occur within a context of objectivity” (2000, p. 31). This characteristic especially impacts the implementation of reflectance as an important step in the team learning process. It is important that both faculty and students recognize that they need to get beyond these stereotypical and social issues in order to not impede learning. In addition to having been trained to objectively leave feelings out of the engineering process it is also found that “students resent the interruption and devalue the effects of good interpersonal skills” (Heywood, 2005, p. 339). This hurdle is one that should be addressed very early in courses. The professor should communicate the value of team skills to the students’ professional success and identify this as a valued learning outcome for the course.

In addition to preparing the student as demanded by the professional world, PBL can also attribute to other important factors in student development including cultural exposure and appreciation of differences. As stated by Edmund Tsang, the editor of *Projects That Matter*, “Future engineers will need more than technical knowledge and skills to assume the moral leadership required to meet the challenges

of the 21st century. Because solutions to many societal problems require extensive interaction and communication with people of diverse social, cultural, and economic backgrounds the engineer of the future will have to communicate effectively” (2000, p. 5). Based on this statement it is in the best interest of all students to have multicultural exposure. In addition to helping the majority student by increasing exposure to a diverse perspective working in PLB teams can benefit minority students by lessening their sense of isolation when working toward a common goal (Duch, Groh, & Allen, 2001). Students from different ethnic, cultural language, social class, ability and social groups tend to develop cooperative, positive and supportive relationships when working in assigned teams (Liao & Chen, 1998). Multicultural exposure resulting in increased tolerance and acceptance would not have the opportunity to form in the traditional classroom.

Professionals and academics recognize the necessity of teamwork and problem-solving skills but it is important that this message be communicated to the student. Emphasis on the fact that learning involves more than just mastery of technical content must be communicated and given value. As identified by Barbara Duch the author of *The Power of Problem Based Learning*, “Many students would still prefer that I revert to previous teaching methods and tell them everything they need to know. I continue, however, to believe that using active teaching methods is key to helping the student become lifetime learners. No profession allows members to use only the information they acquired as students. It is crucial for members of the profession to continue to learn. I believe that PBL helps students acquire the skills the need to continue their education on their own.” Not only does this method of teaching prepare the student for lifelong learning but also “creates conditions that can enable students to learn a great deal about the way they interact with others” (Michaelsen, Knight, & Fink, 2002, p. 48). The team component of this type of course “typically results in higher achievement and greater productivity, more caring, supportive, and committed relationships, and greater psychological health, social competence, and self-esteem” (Smith, 1995, p. 2b5.17). Engineering faculty should have no problem convincing the students of the importance of team interaction given the above advantages.

The significant contributions to learning and student development of using PBL as teams in the classroom make it difficult to argue that changing to PBL should not be incorporated into all engineering curriculums. PBL is difficult to implement partly because of the unfamiliarity of the method among educators and it introduces challenges of effectively teaching and assessing students in this format. The following section of the paper addresses these issues to help faculty members considering the transition.

Critical Components for Success

PBL is not a new teaching method as it has been used in medical teaching schools for a very long time but it is significantly differently than the traditional methods used in most engineering classrooms. The largest change is the role of faculty takes in the classroom. The professor is a facilitator/coach rather than simply a giver of knowledge (Smith, 1995). As with any teaching technique the learning

outcomes must be identified. Implementation of this approach could provide the following general learning outcomes:

- Work with and contribute to a team
- Display understanding of the decision process and clearly communicate justification
- Exhibit ability to define a problem and purpose solutions
- Apply technical content knowledge
- Find, evaluate, and use appropriate resources

There are many other learning outcomes that could be added to this list based on the project selected and the technical knowledge learning objectives.

In order to effectively implement PBL with the positive results identified in the earlier section and to attain the learning outcomes above, there are certain critical components that must be incorporated and the method of application needs to be considered. The components that will be necessary for success of this teaching method are: proper project selection, team selection, and student assessment.

The faculty member is responsible for selecting the project that the teams will be using as a base of their study. This can be done so that each team is assigned a separate project or each team is responsible for solving the same problem. The advantage of using the same problem is that when students' group work is shared with other student teams there is shared learning and reflection taking place. The reason reflection is more prevalent when all teams work on the same project is that although each team approached the same set of problem parameters, they often have very different reasons for their solution of choice. This is an excellent opportunity to be exposed to alternate justifications and thought processes than those they have experienced as a team. The project selection does pose a challenge to the faculty member to find a project that has multiple potential solutions while not overwhelming the teams in complexity. "Projects have to be carefully chosen so that the range of expertise required reflects that available to the team or provision needs to be made for the students to familiarize themselves with that area" (Heywood, 2005, p. 340). This is often the greatest challenge in developing a PBL course. One of the important items to consider in selection is to choose a project that is real or perceived to be real to the students and as closely representative to what they will encounter as a project professionally. Knowing that they will be asked to do this as professionals motivates the students to research, learn and apply the needed information in a self-directed process. This leads not only to the student learning the information but also contributes to their ability to approach a similar problem in the same or similar manner which facilitates lifelong learning.

As previously discussed, the team element of a PBL class leads to many of the learning and developmental outcomes related to PBL. Team selection, therefore, is an educational factor requiring critical attention from the educator. "Team learning seems to work best when the instructor, not the students, organizes, the groups because students learn from others that are different" (Liao & Chen, 1998, p. 54). Students will often select a team of persons they already know or that they feel are similar to themselves therefore "self-selected groups allow at-risk minorities to stand the risk of isolation" (Deibel, 2005, p. 292). The instructor can use many different approaches for selection of the teams, but the method used should "ensure that each

team has members with necessary skills and abilities to complete the assignment” (Michaelsen, Knight, & Fink, 2002, p. 80). The manner in which this is accomplished is dependent on the project and the discipline of study. Whatever the method used, “forming groups in class in the presence of the students eliminates students concerns about any ulterior motives the instructor may have in the forming of the groups” (Michaelsen, Knight, & Fink, 2002, p. 40). Once established, the teams should be maintained for the entire semester since changing periodically consumes valuable time for reassigning teams but more importantly because of the time it takes for the individuals to begin to function again as a team (Michaelsen, Knight, & Fink, 2002). Although it would be convenient and less complicated for the students and faculty, it should be expected that not all teams will avoid interpersonal conflict. The instructor should make it clear from the beginning that they are willing to assist in resolving issues including direct intervention if necessary. Conflict resolution can be challenging for all those involved, but it should be remembered that this experience should be thought of as positive in its own regard because it creates a valuable learning opportunity (Duch, Groh, & Allen, 2001). Those groups that experience conflict will often learn more than those that do not because of the conflict resolution experience opportunity.

Faculty often question how to fairly assess progress and assign grades when using a team based approach for assignments. The question is typically whether one grade should be allocated to all of the group members. “Most team based learning proponents would agree that undifferentiated group grades are potentially problematic” (Michaelsen, Knight, & Fink, 2002, p. 17). The students should be held accountable for both individual and team performance and have grades reflect both. One method of incorporating individual grades in a team setting is to provide a test over a set of information at the beginning of each major unit of instruction. After this test is taken individually and answers submitted, it should then be immediately taken a second time as a team within the same class period. The first test holds the student individually accountable for the learning of the information thus ensuring that they are prepared to contribute to a team effort. The second test, taken as a group, allows the students to discuss and defend their position on each question thus learning from one another (Michaelsen, Knight, & Fink, 2002). “Test score results from the individual and the group should be provided in a very timely manner so that a comparison between the group and individual can be made. This allows for a number of things to happen – the group is able to see that they do better together than individually and they learn that they may have failed to capitalize on the knowledge of one of the group members and are motivated to do something about it. As a result they very quickly learn the importance of including everyone in the decision making process. Thus, over time naturally extroverted or more assertive members do more listening and less talking, quieter students become much more active in team discussions, and cohesiveness increases because members develop a genuine appreciation of other’s contributions to the team success” (Michaelsen, Knight, & Fink, 2002, p. 34). The instructor is also able to quickly assess the areas of weakness of the entire class based upon common weak areas on the individual and group test and can address these areas for all students. This allows class time to be most effective and informative (Michaelsen, Knight, & Fink, 2002). Both the individual

and group grades of these preliminary tests should influence the final grades of the individual student. Grades assigned to team project or homework submittals should be the same for all team members since it was completed as a team effort. In order to encourage participation by all the group members and to highlight work of a teammate above that of the rest of the group or to acknowledge the lack of work by a teammate, peer assessments should be a portion of the final grade. Peer assessments are important because “the students on the teams, not the teacher have the best knowledge of quantity and quality of each member’s contribution” (Michaelsen, Knight, & Fink, 2002, p. 17). “Free loading students seldom occur in team-based learning when used effectively. When groups start functioning as a team, individuals who might be inclined to be freeloaders become very uncomfortable in that role and tend to become contributing members” (Michaelsen, Knight, & Fink, 2002, p. 14). In the case that a student does not ever perform, the groups are large enough that the task can be accomplished without this individual and the non-performer’s final grade at the end of the semester can reflect the input from peer evaluations (Michaelsen, Knight, & Fink, 2002). The peer evaluations should be shared with the students to enhance their learning experience, but it should not be revealed as to which team member provided the comments or ratings.

If the components discussed in this section are adequately incorporated into a course, the likelihood of success in enhanced student learning is improved through PBL. Although the class structure is different from more traditional courses, it should be noted by faculty that the new format will not require the same type or level of preparation that a lecture class requires. The PBL format will require significant time and effort, but this time in teaching will be in response to student questions, assisting the team through conflict, and timely grading and feedback of group submittals rather than lecture preparation, presentation, and individual student grading.

Conclusion

The role of educators and administrators is to develop the most complete student possible through the educational process. This development should include intellectual competence, interpersonal competence, and the ability to create mature interpersonal relationships. The typical engineering curriculum has focused primarily only on the development of intellectual competence, therefore not addressing all the needs of the student. The incorporation of PBL and the use of teams provides an environment in which all of these development objectives can be achieved as well as an opportunity to teach problem-solving and team skills. This method can be implemented in engineering capstone courses as a way to satisfy the development needs of students, satisfy the demands of the employers hiring our students following graduation, as well as the criteria defined by the ABET.

PBL and team assignments require a different approach when planning the course and a change in the role of the instructor. The instructor becomes a coach/facilitator of the teams rather than simply a provider of knowledge. The instructor also must also give careful consideration as to what project is selected for the teams to work on, how the teams are selected and the manner in which the students are assessed. If implemented correctly, PBL can have a significant positive impact on student learning and development.

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BLAST EFFECTS AND PROTECTIVE STRUCTURES? AN INTERDISCIPLINARY COURSE FOR MILITARY CADETS

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ABSTRACT

The increase of terrorist threats over the last decade highlighted the need to protect both civilian and military facilities from explosive accidents or hostile activities. We have observed an increasing need to protect against industrial explosive accidents, criminal activities, and social/subversive unrest. This problem may, in fact, exceed the previous reasons for addressing blast effects and protective structures (i.e., military-sponsored work on fortifications). Obviously, the future of engineering education programs has to be shaped accordingly. Careful attention must be devoted to typical modern civilian and military facilities whose failure could severely disrupt the social and economic infrastructure of nations. Therefore, military engineering cadets need to:

- Know how to assess threats, hazards and abnormal energetic loading incidents.
- Have knowledge of how such facilities (office buildings, schools, hospitals, power stations, industrial facilities, etc.) behave under blast, shock, impact, and fire loads.
- Know how to design such facilities to protect lives and property.
- Know how to conduct effective rescue operations and forensic investigations.

This elective course on blast effects and protective structures is aimed at addressing a broad range of technical issues dealing with mitigating the severe loading effects associated with abnormal loading incidents (e.g., blast, shock, impact, etc.). This elective course will employ extensive course notes, references, various manuals, other handout materials, and a collection of computer applications to be used by cadets.

1. Introduction

Terrorism is not a new phenomenon, and one can find historic references that such activities have existed for more than two thousand years. More recently, the use of terrorism as a means to achieve national objectives was a primary cause for WWI.

Many regions around the world have been increasingly burdened by this phenomenon during the last quarter century (e.g., the Middle East). The locations, causes, participants, intensities, and means have been changing rapidly. Vehicle bombs have become the preferred mechanism for terrorist attacks, followed by the use of hand-carried explosive devices, and the newly evolving threat of weapons of mass destruction (WMD).

Clearly, in today's geopolitical environment, the need to protect both military facilities and civilian populations from terrorism and social/subversive unrest has escalated significantly. This situation is true for many parts of the world, and it may exceed the previous reasons for the development of protective technologies.

This brief summary highlights the fact that the free world has always been reacting to terrorism. The previously observed levels of anti-terrorist preparedness and protective prevention measures have been unimpressive. Unless these conditions change dramatically, the consequences for society could be quite grave.

Since the indications are that explosive devices will continue to be a primary hazard, the emphasis in this write-up will be on blast, shock, and impact. Structural design for safety and physical security requires a sound background in fortification science and technology. One must realize that loading environments associated with many relevant threats (impact, explosion, penetration, etc.) are extremely energetic, and their duration is measured in milliseconds (i.e., about one thousand times shorter than typical earthquakes). Structural responses under short-duration dynamic effects could be significantly different from the much slower loading cases, requiring the designer to provide suitable structural details. Therefore, one must explicitly address the effects related to such severe loading environments, in addition to considering the general principles used for structural design to resist conventional loads. One must be familiar with the background material available for structural consideration and design, as well as up to date on the experience gained from recent terrorist bombing incidents.

Typical engineering curriculums within the military academies have focused on preparing cadets to become engineering officers with hardly any emphasis dedicated to blast effects, protective structures and anti-terrorism security.

2. Development and Implementation

The following summarize the recommended actions required to address our effort:

- To mobilize the local scientific community (including government, private, and academic) for this effort.
- To establish comprehensive and complementary short-term and long-term research and development activities in protective science and technology to ensure the safety of government, military, and civilian personnel, systems, and facilities under evolving terrorist threats.

- To launch effective technology transfer and training vehicles that will ensure that the required knowledge and technologies for protecting government, military, and civilian personnel, systems, and facilities from terrorist attack will be fully and adequately implemented.
- To establish parallel and complementary programs that address the non-technical aspects of this general problem (i.e., culture, religion, philosophy, history, ethnicity, politics, economics, social sciences, life science and medicine, etc.) to form effective interfaces between technical and non-technical developments.

This paper describes an interdisciplinary course in blast effects and protective structures, aimed at educating future military engineers against any potential threat of terrorism. A course outline will be briefly discussed, which spans from; threats and hazard assessment; Conventional and Nuclear Environments; Conventional and Nuclear Loads on Structures; Behavior of Structural Elements; Dynamics Response and Analysis; Connections, Openings, Interfaces and Internal Shock and Structural Systems – Behavior and Design Philosophy. It is not the intent of this paper to discuss the course plan in any great detail, since the course planning and implementation are still an ongoing process.

3. Course Contents

This course can be taught to undergraduate seniors in both science and engineering majors. The issue of blast effects and protective structures is so broad and diverse that to make the course very useful and attractive to cadets, we consider the course topics as essential for a 3-credit course.

Course Description:

The purpose of this course, as stated explicitly above, is aimed at addressing a broad range of technical issues dealing with mitigating the severe loading effects associated with abnormal loading incidents (e.g., blast, shock, impact, etc.). This course will introduce students to the effects of conventional and nuclear weapons on structures and equip them with the tools to define the threat of hazards, define the loads on structures, make a structural assessment of potential cause-and-effect relationships, and recommend mitigation measures for designing or modifying the structural system to protect occupants from the effects of explosive devices.

This course will present the latest information on designing buildings to save lives—from understanding the nature of threats to analysis and design—and will provide students with practical information on performance and design requirements for hardened facilities. In addition, this course will provide information on blast damage assessment issues that will provide forensic and rescue personnel with information vital to their efforts after a catastrophic structural failure. The course may be outlined as discussed below.

*Course Outline:**Block 1: Threats and Hazards Assessment*

Block one begins with a general background discussion addressing the need for man to evolve his efforts in warfare to create a multi-layered approach of taking preventative measures to ensure a safe, sound society. Each student must learn how to conduct a thorough threat and hazard assessment, employing a significant measure of probabilistic analysis and data available in a wide variety of sources as well as a complete understanding of the adversary. Students should utilize a threat and hazard assessment method similar to that provided in *ASCE, 1999*. Understanding the characteristics of explosive processes, devices, and weapons aids the student in preparing a thorough assessment of adversary capabilities. The adversary will use his capabilities against a desired target, requiring the student to conceptualize how to protect a given structure; a basic protective planning and design philosophy is needed. Realizing the threats allows the student to assess the hazards inherent on a target. Thus, a brief summary explaining blast effects on structures, both locally and globally, is necessary. Additional research in many areas of blast effects is crucial to reflect the need to understand ever-evolving weapon systems and utilize new experimental and analytical tools to ensure overall public safety.

Block 2: Conventional and Nuclear Environments

Block two addresses the technical side of analyzing blast effects in conventional and nuclear environments. An explosion can create airblast, ground shock, ejecta, fragments, fire, thermal, chemical, radiation, and electromagnetic pulse effects. Each explosive effect is given an “equivalent TNT” value that translates into a tangible force on a structure that can be included in a structural design.

Block 3: Conventional and Nuclear Loads on Structures

Conventional and nuclear loads differ significantly, a point that the design approach must accommodate. Another stark difference exists between current design procedures and information gathered from recent research. Block three addresses the technical information concerning these two topics.

Block 4: Behavior of Structural Elements

Block four contains an overview of the behavior of structural elements to include dynamic loading, material properties of steel and concrete, flexural resistance, shear resistance, column loading, and frame response. This chapter provides a review of the current materials available for blast-resistant design. The *Technical Manual 5-866-1* used by the Army and *Technical Manual 5-1300* used by the Army, Navy, and Air Force, and *ASCE, 1985*, *ASCE, 1997*, and *ASCE 1999*, contain the bulk of information used by non-governmental organizations.

Block 5: Dynamics Response and Analysis

Dynamic response and analysis is a crucial evaluation step when analyzing an explosive structural impact. Different methods of analysis are required for specific situations and loading cases. The two mentioned in block five are the Single Degree Of Freedom (SDOF) and Multiple Degree Of Freedom (MDOF) analysis methods. A few not expounded upon in the paper are the finite element, finite difference, and hybrid methods.

Block 6: Connections, Openings, Interfaces and Internal Shock

Block six focuses on the behavior of specific structural elements and the necessary implementation for design, as well as on the effects of internal shock. Connections make modeling a beam or structure difficult, but are necessary to analyze since in reality, simply supported, fixed, hinged or free conditions rarely are completely accurate. Connections must withstand the blast effects in order for the structure to function properly. Therefore, it is necessary to spend, in many cases, more time ensuring connection components are adequate than on other structural components to assure exceptional structural performance.

Block 7: Structural Systems – Behavior and Design Philosophy

The threats discussed are very energetic, and with such a short duration require unconventional structural analysis methods. The design philosophy involves architectural and functional considerations, load considerations, structural system behavior, structural system and component selection, as well as other safety considerations. Block seven also discusses the development of a “loading envelope” to encompass a multi-protection design (earthquake plus blast effects.)

4. Case Study and Tools:

The Blast Effects and Protective Structures course had a case study due for each of the seven blocks, making seven total case studies. To apply a definition in this course, a case study is a challenging practical problem aimed at introducing the student to the given block’s concept. Many of the case studies required students to gather information from instructors and professors in multiple departments with various specialties and fields of study, as well as information from a plethora of sources.

The blast effects resources used for the majority of the case studies are as follows: *Technical Manual 5-866-1*; *Technical Manual 5-1300*; and *ASCE 1999*. In addition, the following software programs were used: “CONWEP”; “BLAST-X”; “SHOCK Version 1.0”; and “FRANG Version 1.0.” The software allows for calculation of internal shock and gas pressures. Each program has its specific strengths. “SHOCK Version 1.0” allows for the calculation of average barrier reflected shock pressures and impulses due to an incident and reflected waves from one to four reflection

surfaces. “FRANG Version 1.0” allows for the “calculation of the time history of gas pressure and impulse resulting from an explosion within a room” (FRANG Version 1.0 statement). It can account for venting through both an uncovered vent and a vent covered by a frangible panel. The “CONWEP” software is an assortment of conventional weapons effects calculations based on TM 5-855-1, allowing the user to apply a variety of blast effects ranging from airblast to fragment and projectile penetrations to ground shock and receive empirical results as output. “BLAST-X” is intended to perform shock wave and pressure calculations of internal or external explosions on a structure.

5. Conclusion

Terrorism is not a new concept, striving to strike fear in the hearts of citizens in rival nations. Historical examples of such activities date back two thousand years. Most recently in the United States, non-state actors have demonstrated that one of the world’s most powerful nations is not immune to terrorism. Many regions around the world have been increasingly burdened by this phenomenon during the last quarter century (e.g., the Middle East). Techniques and methods to most effectively execute terrorism are evolving every day, creating new and challenging problems for security personnel.

Clearly, in today’s geopolitical environment, the need to protect both military facilities and civilian populations from terrorism and social/subversive unrest has escalated significantly. Current levels of preparedness and protection in many key, nationally important structures are inadequate. This brief summary highlights the fact that in order to protect the civilian and military populace from disaster, blast, shock, and impact on certain structures must be accounted for in the design portion of such buildings.

Structural design for safety and physical security requires a sound background in the fortification science and technology. The loading environments associated with many relevant threats (impact, explosion, airblast, penetration, etc.) are extremely energetic and short-lasting (i.e., about one thousand times shorter than typical earthquakes.) The dynamic structural effects will typically be significantly different from slower, conventional loading cases, requiring the engineer to utilize an unconventional paradigm to create design solutions to complicated blast effects. One must be familiar with current world events and terrorist tactics, as well as current analysis and design methods to create effective designs. Therefore, one must explicitly address the effects related to such severe loading environments, in addition to considering the general principles used for structural design to resist conventional loads.

Acknowledgement

Funding from the Dean’s Fund to Promote Academic Excellence (ACE) is acknowledged and appreciated.

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Robotics and Automation in Construction Industry

by

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Abstract: Construction phase is one of the important aspects of civil engineering structures. The success of a project depends on how well the construction phase is carried out. Efficient and economical construction is particularly important because of the increasing complexity of structures being built, the availability of improved materials and construction equipment. Typically in manufacturing field, robots are stationary and product moves along the assembly line. Automation is easier to incorporate because each product is identical with respective tasks done over and over. However, construction robots face with different demands than conventional industrial robots. They must move about the site, because buildings are stationary and of a large size. They require engines, batteries or motors and drive themselves. Construction robots also faced with changing site conditions and must be reprogrammed with each new condition. They must be able to function under adverse weather conditions including variations in humidity and temperature. Additionally, they are constantly exposed to dust and dirt on the site. Thus, there is a need to develop a robotic system for full-scale experimentation for realistic assessment of automation in the construction industry. This paper presents some aspects of robotization and automation and a case study is presented to demonstrate the applicability of robotics in construction industry.

Key words: Automation, robotics, construction industry, and productivity.

Introduction

The construction industry is conservative in accepting new approaches and is dependent upon the conventional methods and equipment. Slowly, it is mechanized and computerized, keeping in view of the time of completion of the project, shortage of labor force, and quality of work to be executed. It is now looking into the robots for different works for execution. Technology leading to a higher degree of machine and equipment automation will yield productivity improvement higher than a skilled worker before full robotization in practice. Construction robots are faced with problems of changing sites and must be reprogrammed with each new condition. The primary factors such as need based feasibility, technological feasibility, and economic feasibility are identified for the use of robots and to evaluate benefits, thereby inviting future robotization for Indian construction sector.

A robot designed for tasks such as painting, and jointing in buildings certainly result in higher efficiency with respect to pay load, reach, number of degrees of freedom, etc. Further, the use of robots in construction will result less use of labor force, there by reducing the risk to the lives of

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the labor, cost of the project, and time for completion. Robotics has been extensively used in many fields because it enhances workers safety, increases production efficiency, and improves quality of products. Robot are used in phases of construction work for production of material, construction of different works (quality control), maintenance and operations (including inspection and monitoring), and performance in hazardous environment.

Robotization for interior finishing tasks such as painting, plastering, masonry and tiling is an important element of an integrated and automated building realization system. A building technology suitable for such system includes precast structural exterior wall components with its interior finishing works done by a multipurpose robot. The development of the multipurpose robot enabled a realistic assessment of the productivity of the robotic system and estimation of costs associated with robotic work. Consequently, these costs could be compared to the cost of work performed with conventional manual methods. Feasibility of using robots is determined from comparison of robot versus manual performance of pertinent building tasks. The following study presents a quantitative assessment of two aspects of robotic feasibility saving in human labor and its impact on costs.

Literature Review

Rapid advancements in robot technology, control theory, and computer technology will accelerate automation and vastly broadened the applicability of robots. The growing interest of construction automation and robotics by various construction professionals leads the application of robots in civil engineering departments. A small amount of effort spent for identifying the appropriate tasks will result in more efficient research budgets and more successful deployment of new technologies. Several studies have identified the different types of construction works for automation.

Everett (1995) proposed the automation and robotics are solutions to problems in productivity, quality, safety, and skilled labor availability. He evaluates construction automation and robotics in the context of their ability to satisfy the often-conflicting demands of managers and owners, workers, and society. Difference in cultural, economic, and business practices help explain why construction automation and robotics are generating so much activity and investment in Japan and so little in the United states.

Kangari and Halpin (2001) have discussed the major factors influencing the robotization of construction process and provided a model based on fuzzy set theory. Navon (1989) stated that several tools and techniques have been developed for the optimal design or the selection of a robot for a given task such as computer simulation, and mathematical techniques. Removal of existing coatings (mainly lead based primers) is normally done by blast cleaning. Smith (1990) stated that according to national safety council, 70% of all series injuries to coating workers are caused by falls. Development of robotic systems for construction applications has advanced dramatically for the past few years

Kangari (1989) described major factors influencing the robotization of construction processes, and to provide a model based on fuzzy set theory for feasibility analysis of robots. Major factors influencing implementation of construction robots are described.

The primary factors driving the adoption of robotics in construction are identified as need-based

feasibility, technological feasibility, and economic feasibility. Whittaker and Bandari (1986) reported that robots were emerging in construction as a way to increase productivity, improve quality, and decrease hazards to human workers.

However, these industrial robot forms, though necessary, are not sufficient to achieve typical construction goals. The requirement for multiple capabilities at the automated work site must be served by multiple cooperative robot agents.

Competitive Advantages

Traditionally, construction requires labor-intensive procedures. Many tasks demand skilled personnel. Some consider the wages and benefits paid to construction workers to be costly. Reducing the labor requirements and greatly improves contractors ability to compete. Through labor savings, Construction robots have the potential to lower building costs dramatically.

As found in other industries, robots typically work faster than humans do. Many believe that robots are also more dependable. With machines, there is less concern about fluctuating productivity due to the ups and downs of life. In the construction industry, time is money. Since these devices are considered faster than humans at work operations, contractors should expect to complete project sooner. When construction work is finished ahead of schedule, builders save money due to reduced job site overhead. Additionally, they have an opportunity to bid and work other project sooner than expected. More ever, building owners are pleased to gain early occupancy to buildings and a quicker return on their investment. Robots that save time will make builders more competitive.

Properly designed robots produce a higher quality product than humans; conceivably, building owners may be more satisfied with the final product. Therefore, contractors should enjoy a better relationship with their clients and expect to be invited back for additional projects. Additionally, by using robots when possible, builders are less likely to experience callbacks to replace defective workmanship.

Safety is an important aspect to any construction project. However, the cost of safety is significant. Investing in fall protection, personal protective equipment and safety management can be costly. Some of these costs be reduced if robots are utilized in lieu of humans.

Private construction companies do not invest significant funds for research of new methods. Much of the research is government sponsored and conducted at universities and government laboratories. These research endeavors have been conducted primarily to obtain new knowledge and develop new principles.

Inherent Barriers

There are many barriers that discourage the development and use of construction robots today. It is customary for the design and construction operations to be handled by two different companies. According to wing (1992), this is clearly one of the main reasons for the delay of adopting robotics in construction sector. In the manufacturing industry, the link between design

and fabrication is strength with this cooperation; engineers are better able to design for automation.

Construction robots can eliminate workers from being exposed to dangerous, dirty and heavy labor, however, they take considerable time for set up, adjusting and clean up. Skilled personnel are needed to operate and continuously monitor the robots. As noted earlier, single task robots are limited since they are not capable of identifying and fixing problems in real time. Therefore, skilled workers must be present during work operations. Some question the labor cost savings by using single task robots.

More over, robot operators need training, as robots become common on the site, a new breed of worker will be needed. Special training in computer usage will be mandatory. Workers will need a stronger background in academic areas such as maths, reading and problem solving to be productive. Even with robots, contractors must still invest heavily in training and educating the worker.

Objectives of the Research

The basic objectives of the paper are to

- identify major factors that influence the robotization of construction process,
- identify the parameters depending on the nature of building including the nature of task to be performed,
- calculate the cost of robot using parameters, and
- develop the robotic system for full-scale experimentation for realistic assessment of productivity.

Cost elements of Robot

Parameters dependent on the robotic system include cost of work envelope, its speed of movement and mode of operation. The cost per hour is identified as

$$C = (P \times P_r(I, N) + C_m)/H + C_o$$

Where

P is investment of the robot (including the cost of carriage, effectors, sensors & others adoptions)

$P_r(I, N)$ is capital recovery factor

C_m is Cost of repairs and high level maintenance of the robot per year.

C_o is the Operation g costs per hour and,

H is number of robot employment hours / year.

Research Methodology

The methodology for the application of robotics consists of the following steps:

- Identify feasibility and technological and organizational problems for implementation
- Select robot configuration such as pay load capacity, and sensor attributes for each task
- Develop the preprogrammed path to operate the robot by remote control
- Identify the places where robot can reproduce the elements
- Calculate the cost of robot using parameters including depreciation, interest on investment, maintenance and operating expenses
- Analyze the cost per hour 'C' for using robot
- Develop system for full scale experimentation for realistic assessment of productivity

The above research methodology has been adapted to the following case study:

Case Study

The economic analysis for interior building task of painting, plastering, masonry and tiling is examined with Technion of Autonomous Multipurpose Interior Robot (*TAMIR*) for the construction of outpatient block of Rajiv Gandhi Institute of Medical Sciences at Kadapa, Andhra Pradesh, India.

Setting Out

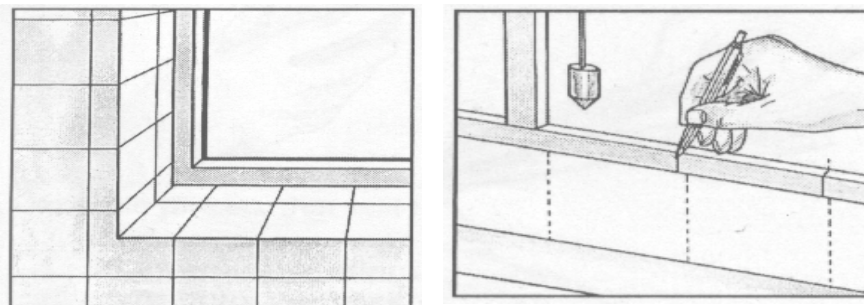
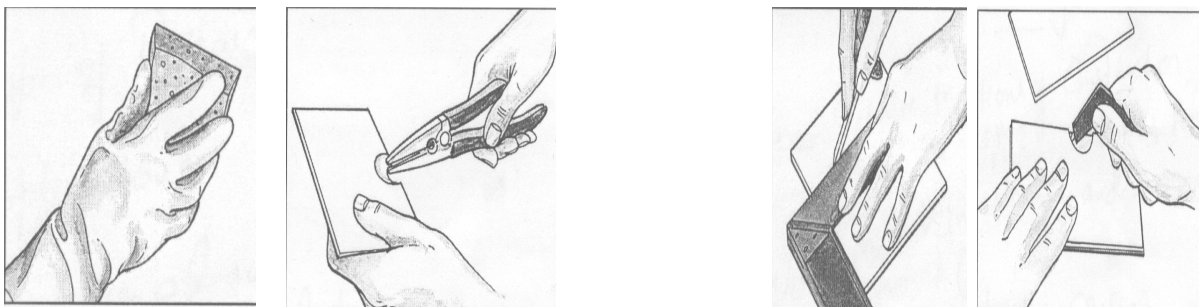


Fig 1 Setting Out

Placing



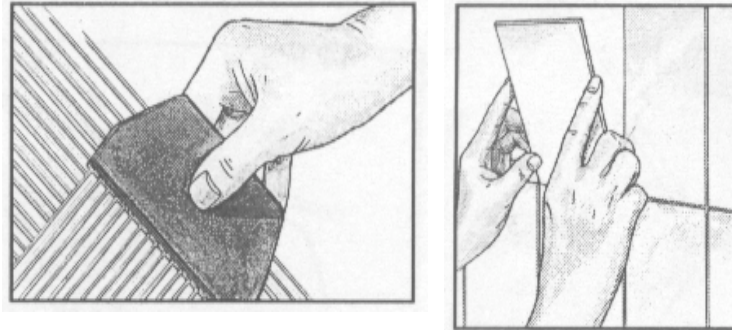


Fig 2 Placing

Robotized Method

Wall tiling

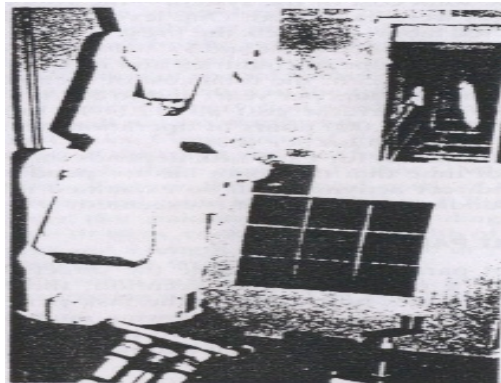


Fig 3 Wall Tiling

Robotized tile setting consists of these subtasks

- picking a tile from a box (or) cartridge with a vacuum gripper
- receiving glue (or) cement on the reverse side of the tile by pressing it against an ejector, attached to the robot; and
- placing the tile as its designated place on the wall. Alternatively, cement or glue can be applied manually on the wall prior to attachment of the tiles by the robot.

Wall tiling time in put for the robot is 0.75 min per tile including picking up the tile, gluing the tile and placing the tile on the wall. Using 15cm x 15cm tiles; the input is 33 min/m². The manual input for the same work is 90 min/m². It must be noted however that this time includes measuring and cutting of tiles in irregular surfaces. In well-structured tiling areas, the labor input will be some what smaller. The number of tiles needed is approximately 45 per m² or about 900 tiles for a medium sized room. Such number of tiles can be prepackaged on a 50 cm x 50 cm pallet and pre-placed near the work area are attached to the robot carriage. In addition and adhesive supply canister must be placed near the robot or attached to it. The cost of material for both types of work robotized and manual is about the same.

Table 1 Cost of Robot per Hour under Various Conditions

Economic life cycle n (years)	Total Cost per Hour (Dollars)					
	H = 1,500 h/yr			H = 2,000 h/yr		
	i=5%	i=7%	i = 10%	i=5%	i=7%	i = 10%
4	35.11	36.23	37.97	26.83	27.68	28.98
5	30.68	31.81	33.55	23.51	24.36	25.66
6	27.74	28.87	30.60	21.30	22.15	23.45

The cost per hour for various alternatives of economic life cycle, interest on investment, and work hours per year, is shown in Table. It can be seen from Table, that the cost of the robot per hour is not very sensitive to changes in the rate of interest or to changes in its economic life. It is however very sensitive to changes in the number of the employment hours per year. The cost of the robot in the subsequent economic comparison was based on the most plausible alternative of 5 years of economic life, 7% of interest rate, and two employment possibilities-of 1,500 and 2,000 h/yr.

The output of the robot for each task depends on the nature of the task and the amount of time that the robot has to spend on indirect activities. The burden of indirect activities will be examined in the context of each task and of the building site where it is executed.

Table 2 Comparison of Robotic versus Manual Performance in Wall Tiling

Variable	Robotic	Manual
<i>(a) Labor Breakdown</i>		
Direct labor	33.33 min/m ²	90 min/m ²
Movement between Stations	0.05 min /m ²	-
Positioning at workstation	0.75 min/m ²	-
Transfer between floors	0.02 min/m ²	-
Setup at site	0.05 min/m ²	-
Maintenance	2.12 min/m ²	-
<i>(b) Total Labor Required</i>		
Skilled labor input	36.32 min/m ²	90 min/m ²

Conclusions

The use of automation and robotization in construction industry is rapidly expanding and reduce accidents and improves working conditions of construction workers. The reduction in the members of highly skilled craftsmen, increase in sophistication of construction materials and constant demand for increase in quality and reduction of construction time all indicate that greater degrees of automation will be required in construction industry. Significant advances have already been made in both large and small scale applications. The main conclusions drawn from this research are

- Automation and robotics are proposed as solution to problems of productivity, quality, safety and skilled labor availability.
- The robot can bring considerable savings in skilled manual labor on site. The savings in skilled labor ranges between 60% - 90% for typical tasks.
- The cost wise comparison of robotized versus manual work depends upon prevailing wages of labor. The savings may be 10 – 50% in cost of work excluding materials.
- In mechanized manual tasks the cost will be lower than in robotized construction. Some special requirements should be made to improve the robotic performance in this task.
- The employment of a robot for interior finishing works has considerable potential for productivity improvement on the building site.
- Under proper planning, robotized method can be reduced in direct construction cost.
- There is constant demand for increase in quality and reduction of construction time.
- Construction managers need to advocate more standardized designs that allow for easier automation.

In addition to above benefits, using automated construction equipment reduces healthy risks and potential deaths. This can lead to substantial savings through increased productivity.

Acknowledgements

The authors expresses their sincere thanks to the Department of Science and Technology, Govt. of India for funding this research work through permit number SR/S3/RM/55/2002 and also wishes to express gratitude to the Department of Civil Engineering, OU for providing facilities to carry out this research work.

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The Nation's Architectural Engineering Programs: A Diverse Group

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Abstract

There are currently 17 ABET accredited Architectural Engineering programs in the United States. This paper presents curriculum data for these Architectural Engineering programs and discusses the challenges of finding common ground and conveying program needs to the rest of the academic community – especially when the programs are so small in number and so diverse in their structure and course offerings.

Introduction

There are currently 17 ABET accredited Architectural Engineering (AE) programs in the United States. Most are four year undergraduate programs, but almost 25% are five year programs (4 out of 17). Some offer masters degrees, while others do not. A couple of programs offer Ph.D.s in Architectural Engineering or Building Systems. About half of the AE programs are located within departments of civil engineering, while the other half are departments in their own right. Most fall under the college of Engineering, but two of the programs are housed in the School of Architecture. One is located in the College of Engineering, but admits students through the College of Architecture. The ABET program accreditation criteria require coverage in two of the three areas of structures, electrical/ mechanical systems, and construction. Some programs are very balanced and offer a full complement of courses in electrical, mechanical, acoustical and lighting design. Others focus heavily on the structures area and offer little in at least one of the other categories. Two of the programs are very focused on architecture, with nearly half of the AE content concentrated in architecture. This paper will examine the undergraduate curriculum data on the 17 ABET accredited Architectural Engineering programs and will discuss the challenges of finding common ground and conveying program needs to the rest of the academic community – especially when the programs are so small in number and so diverse in their structure and course offerings.

The Nation's Architectural Engineering Programs

Because there are so few AE programs, many incoming students are confused about the major and how it differs from Architecture or Civil Engineering. Architectural Engineering involves the engineering design, construction and operation of safe, functional, efficient, economical, aesthetically-pleasing buildings. AE deals with all engineering aspects of a building's performance that support the architectural and functional requirements. These engineering aspects commonly include structural systems; mechanical systems such as heating, ventilation, air conditioning, and plumbing; electrical

systems such as lighting, power distribution, control and communications; and other systems such as acoustics and fire safety.

Table 1 shows a listing of the 17 Architectural Engineering programs in the United States that are currently accredited by ABET. Geographically, seven of the programs are located in the Midwest and three are in the South. There are two programs in the Southwest, Rocky Mountain, and Northeast regions, with only one program located on the West Coast. All of the AE programs reside in a College/School of Engineering, with the exceptions of Oklahoma State and Cal Poly San Luis Obispo, which are included in the Colleges of Architecture. Ten of the programs are collocated in departments that also manage civil engineering programs while the remaining seven are either separate departments or are collocated with programs other than civil engineering such as facilities or construction engineering,

	University	Department	College/School
1	Drexel University	Civil, Architectural and Environmental	Engineering
2	Kansas State University	Architectural Engineering and Construction Science	Engineering
3	Illinois Institute of Technology	Civil and Architectural	Engineering
4	University of Nebraska	Architectural Engineering	Engineering and Technology
5	Milwaukee School of Engineering	Architectural Engineering and Building Construction	Engineering
6	University of Wyoming	Department of Civil and Architectural Engineering	Engineering
7	Oklahoma State University	School of Architecture	Engineering , Architecture and Technology
8	Penn State University	Architectural Engineering	Engineering
9	California Polytechnic State University	Architectural Engineering	Architecture and Environmental Design
10	University of Kansas	Civil and Architectural	Engineering
11	University of Texas	Civil, Environmental and Architectural Engineering	Engineering
12	North Carolina A&T State College	Civil, Architectural and Agricultural and Environmental Engineering	Engineering
13	Tennessee State	Architectural and Facilities Engineering	Engineering, Technology and Computer Science
14	University of Colorado	Civil, Environmental and Architectural Engineering	Engineering
15	University of Miami	Civil, Architectural, and Environmental	Engineering
16	University of Missouri Rolla	Civil, Architectural, and Environmental	Engineering
17	University of Oklahoma	Civil and Environmental	Engineering

Table 1. A listing of the seventeen accredited architectural engineering programs and their associated departments and colleges

The relationship between architectural and civil engineering is an interesting one since there are so many more civil engineering programs in the U.S. In 2005, there were 722 bachelor degrees awarded in architectural engineering while there were 8247 awarded in civil engineering (ASEE 2005) – more than ten times as many. While the relative difference in enrollments will vary from school to school, civil engineering gets more attention and emphasis at the national and professional society level. One can draw inferences about the relative influence within a university from the number of degrees conferred. Figure 1 shows the number of bachelor degrees in both AE and CE awarded in the 2004-

2005 academic year for those schools with accredited AE programs (ASEE 2005). Milwaukee School of Engineering is the one school in the study that does not offer a civil engineering program. Also, the University of Oklahoma was not accredited by ABET until 2006.

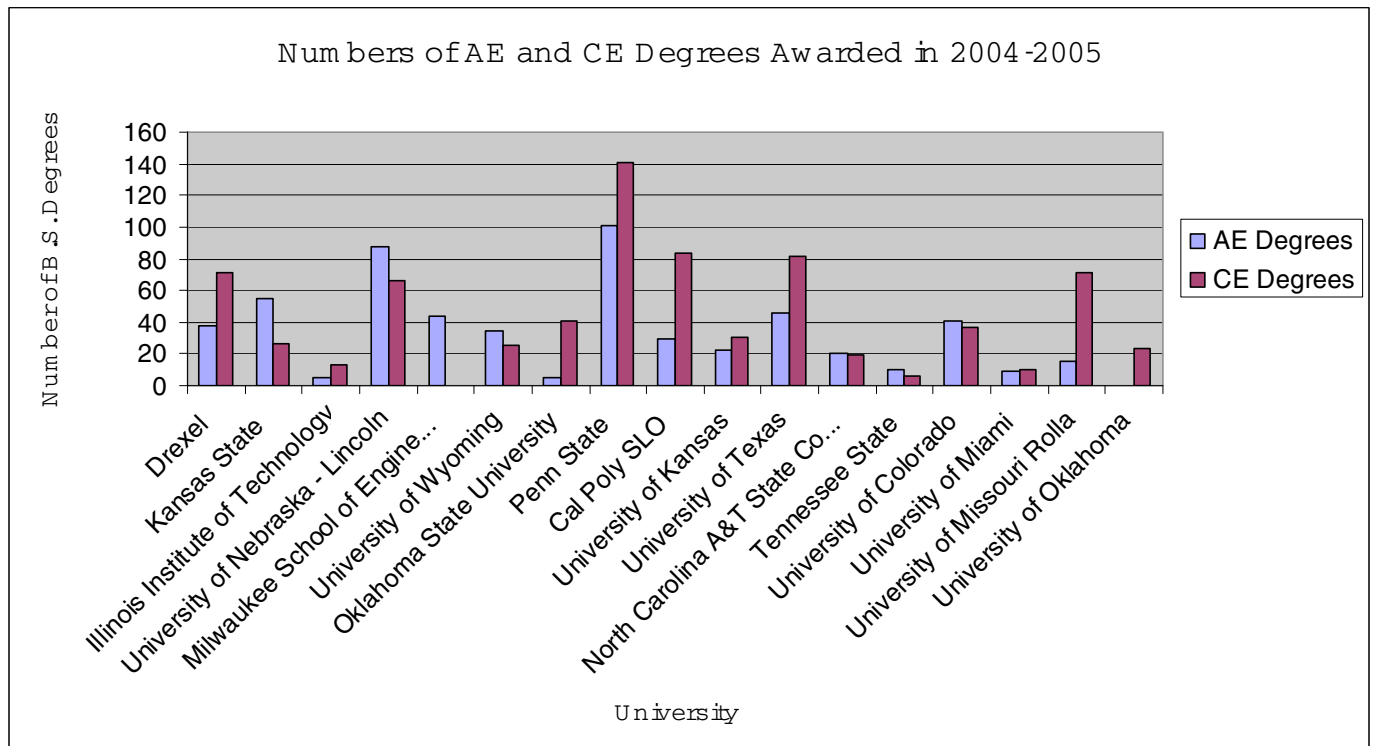


Figure 1. A comparison of the number of bachelor degrees awarded in the 2004-2005 academic years awarded in both Architectural Engineering (AE) and Civil Engineering (CE) for those universities with an accredited AE program (ASEE 2005).

The most common overlap between the two disciplines is in the area of structures. While structural engineering is one of the most common sub-disciplines of civil engineering, all of the current AE programs include structural design of buildings in their curricula. In cases where the AE and CE programs are collocated in the same department, it is common for faculty with a structures background to teach both AE and CE students, often in the same class. In universities where they are in separate departments and in some cases separate colleges; there is either some duplication of effort or at least some cross-listing of courses.

It is difficult to compare the 17 AE programs since four of the programs (Penn State, University of Kansas, Kansas State, and Oklahoma State) are five year programs, while the others are four year programs. Most of the schools are on a semester system with the exception of Cal Poly, Drexel, and Milwaukee which are on a quarter system. Table 2 shows the seventeen AE programs and lists the length, type of system, credit hours and equivalent semester credit hours. For schools on the quarter system, the equivalent semester credit hours were obtained by multiplying by two-thirds.

The Academic Council of the ASCE Architectural Engineering Institute (AEI) recently discussed how the various national programs could be ranked with the possible inclusion of AE programs in the annual U.S. News and World Report rankings, or other such ranking avenues. Because the programs

have different areas of emphasis and are of different lengths, such a ranking would require making some value judgments. Is a program that is balanced in the designated AE specialties better or worse than a program that focuses more heavily in one area at the expense of others? Is the material that is covered in the extra year of a five year program better or worse than the year of experience gained by those who graduate in four years? It is probably better to advertise the strengths of the available programs and let the students and industry help decide. However, the exposure at the national level from such a high profile ranking would help strengthen the status of AE as an engineering discipline, and provide advertisement for the profession.

University	Reference	Length of Program (Years)	Semester or Quarter	Credit Hours	Equivalent Semester Credit Hours
Drexel University	DU(2008)	4	Quarter	200	133
Kansas State	KSU(2008)	5	Semester	158	158
Illinois Institute of Technology	IIT (2008)	4	Semester	136	136
University of Nebraska	UN (2008)	4	Semester	126	126
Milwaukee School of Engineering	MSOE (2008)	4	Quarter	197	131
University of Wyoming	UW(2008)	4	Semester	132	132
Oklahoma State University	OSU (2008)	5	Semester	160	160
Penn State	PSU(2008)	5	Semester	160	160
Cal Poly SLO	SLO(2008)	4	Quarter	203	135
University of Kansas	KU(2008)	5	Semester	165	165
University of Texas	UT(2008)	4	Semester	126	126
North Carolina A&T State College	NCA&T (2008)	4	Semester	128	128
Tennessee State	TSU(2008)	4	Semester	126	126
University of Colorado	CU (2008)	4	Semester	128	128
University of Miami	UM (2008)	4	Semester	129	129
University of Missouri Rolla	UMR (2008)	4	Semester	128	128
University of Oklahoma	OU (2008)	4	Semester	128	128

Table 2. A listing of the length and type of program for the seventeen accredited AE programs

Curriculum Analysis

The seventeen AE programs have a number of differences, but share more similarities than it initially appears. Once the number of credits are adjusted to equivalent semester hours, the four year programs range between 126 and 138 credits hours. The five year programs are closer to each other containing between 158 and 165 credit hours.

The program accreditation criteria undoubtedly account for a number of similarities in the programs. The ABET program criteria for AE programs (ABET 2006) require “proficiency in

mathematics through differential equations, probability and statistics, calculus-based physics, and general chemistry” which mandates a substantial math and science component to every program. Because there is a specific requirement for proficiency in statics, strength of materials, thermodynamics, fluid mechanics, electric circuits, and engineering economics, those topics appear in each program, usually through a dedicated course. In the specific areas associated with architectural engineering, the criteria are more flexible requiring proficiency in only two of the three areas of structures, electrical/mechanical systems, and construction. Finally, architecture is included into each curriculum to adhere to the minimal requirement for “understanding of architectural design and history leading to architectural design that will permit communication, and interaction, with the other design professionals in the execution of building projects.”

For each of the programs, the curriculum was analyzed and each course was classified into one of the following categories:

- *A: Communications* –includes any course with the purpose of writing, public speaking, technical presentation, or a required English elective. This category was separated from general humanities electives because of the increased industry emphasis on producing graduates with adequate communication skills, and ABET specifically requires this as part of the outcomes assessment criteria. Admittedly, some universities include their communication courses on a larger list of electives that were included as humanities electives.
- *B: Humanities* – most programs had some breadth requirement that included courses in political science, economics, history, psychology, sociology, etc.
- *C: Math and Science* – these courses included math (calculus, statistics, differential equations), basic science (chemistry, physics, biology, ecology), and computer science where the emphasis is on programming or computer theory rather than CAD drawing or computer applications.
- *D: Engineering Science* – courses such as statics, thermodynamics, electrical circuits, fluid mechanics etc. that provide the theoretical concepts that will be used for engineering design.

The next three categories are all engineering analysis and design courses

- *E: Structures* – this categories include those courses commonly included in structural engineering. In this study, structural analysis and soil mechanics were included in this category
- *F: Electrical/Mechanical Systems* – this category includes courses that support the design of electrical and mechanical systems in buildings. Courses that cover lighting, acoustics, fire safety, and environmental controls fell into this category.
- *G: Construction* – courses that would normally be part of a construction program fell into this category and include project management, contracts and specifications, construction materials, construction methods, construction law and construction estimating. In cases where the engineering economics course had an obvious emphasis on construction financing or investment, that course was included in this category.
- *H: Technical Electives* – this category became too cumbersome to be useful. Every program offers a different number and variety of electives. Some offered program options into one of the areas listed in categories E, F, and G above. For this study, the

technical electives were incorporated into categories E, F, and G in the relative proportion that they appeared on the electives or options list.

- I: *Other* – this category included those courses that did not fit logically elsewhere. It included courses such as physical education, university experience, engineering economics, professional practice, and surveying.
- J: *Freshman Engineering* – these are courses offered to freshman that precede any of the engineering science courses. The courses in engineering graphics, introduction to engineering, and the engineering profession were all included in this category.
- K: *Architecture* – courses taken from the university's architecture program are included here. Almost every program has something in this category because of the accreditation requirement for the history of architecture. Those AE programs that require students to participate in architecture design studio courses have a much greater number of these courses.
- L: *Capstone Design* – many programs offer a senior project course that requires a complete design of a complex system. It is a culminating experience that requires students to synthesize and use all of the skills developed in the program. The content of these courses would mostly fit into the categories E, F, and G above, but is listed here.

The information for making this assessment was taken from the website postings for the individual universities listed in the references from Table 2. In many cases, the on-line course catalogue description was consulted to place a course in the appropriate category. The advantage of this approach is the consistency achieved when the same individual conducts the assessment for each program. The disadvantage is the lack of familiarity with a program that could lead to a mistaken classification or misrepresentation of a program. In many cases, compromises were made, particularly with electives, where a different individual might make a different classification for some courses. In any event, the assessment is sufficiently accurate to highlight the various strengths and differences within these AE programs.

Figure 2 shows the curriculum analysis for the AE program at Drexel University. It indicates that a student attending the AE program at Drexel will receive significant coverage of math, basic science and engineering science. There appears to be a stronger emphasis on electrical/mechanical systems than on structures or construction and there is significant exposure to courses in architecture. Similar figures are available for the other 16 programs, but limited comparative information can be gained from looking at the programs in isolation.

Figure 3 offers comparative information in the specific AE areas of structures, electrical/mechanical systems, and construction. Courses in architecture are included as well. Because the senior design consists primarily of the AE specialty areas, it is also present. Several key differences are immediately apparent. The programs at Cal Poly and Oklahoma State offer a heavy emphasis in structures and architecture at the expense of both construction and electrical/mechanical systems. A student at the University of Kansas or the University of Oklahoma will take a substantial number of Architecture courses. The most balanced program with the most number of semester hours seems to be Penn State. Since Figure 3 is based on semester hours, the four five-year programs stand out over the four year programs.

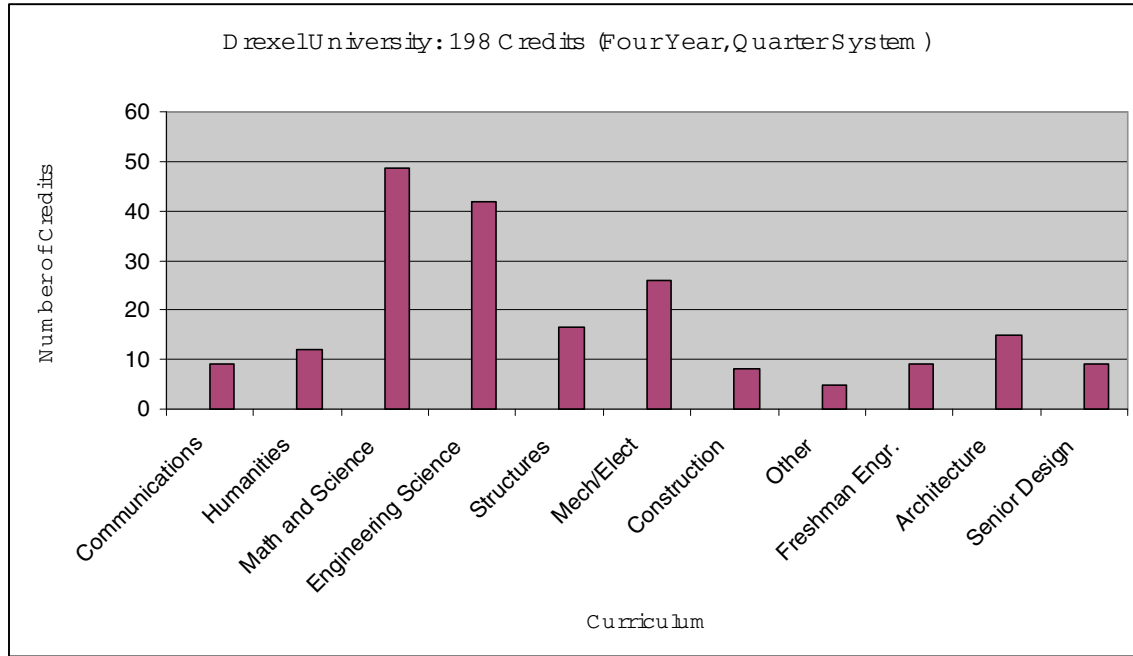


Figure 2. Curriculum analysis for the architectural engineering program at Drexel University.

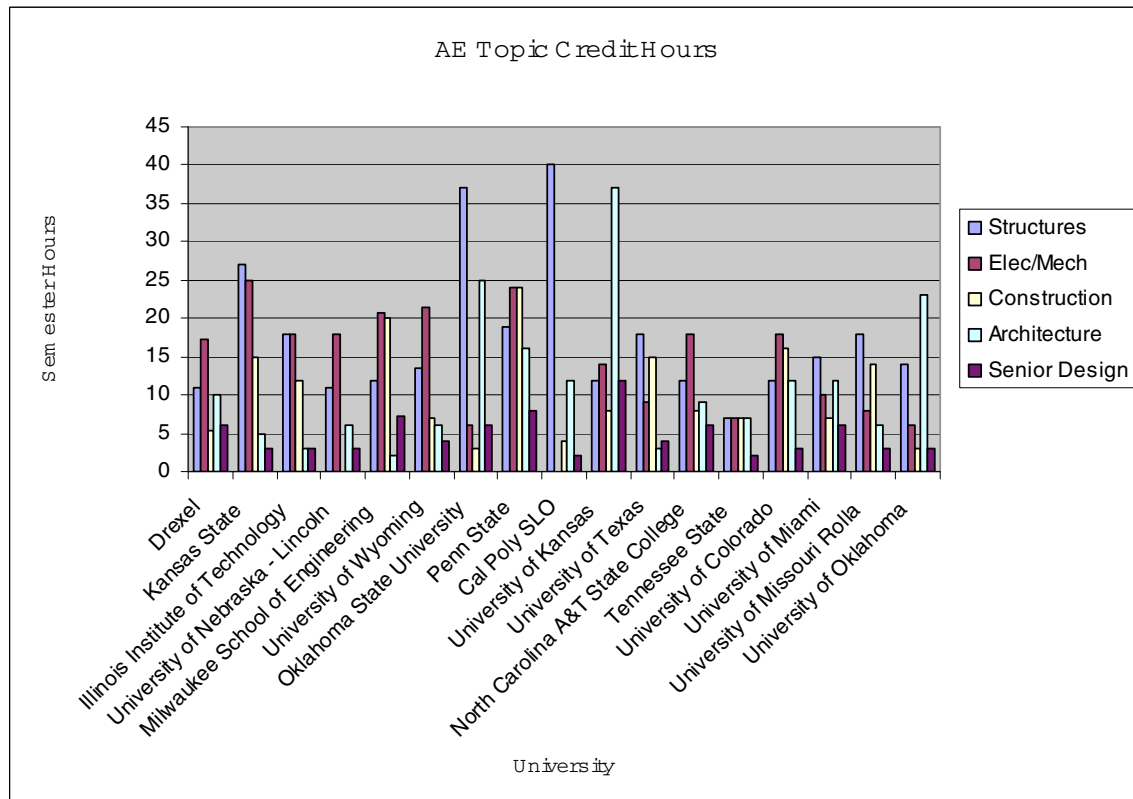


Figure 3. An analysis of the AE specialty areas by semester hours for the 17 accredited AE programs.

Figure 4 attempts to remove this bias towards the five year programs by looking at the AE specialty areas as a relative percentage of emphasis. It becomes clearer from this figure that the University of Colorado and University of Miami also have created balanced programs that draw somewhat equally from all of these AE specialty areas. Conversely, Tennessee State has a perfectly balanced program in these areas, but as Figure 3 indicates, they offer relatively few credit hours in any of these areas. These figures indicate that a student who wants more emphasis in a particular area can choose an AE program accordingly. For example, a student wanting to study electrical/mechanical systems in a four year program would be well served by either the University of Wyoming or the University of Nebraska. However, the cost is that there is very little Architecture at Wyoming and almost no construction taught at Nebraska. Similarly, the University of Texas would be an excellent program for someone wanting additional emphasis in construction.

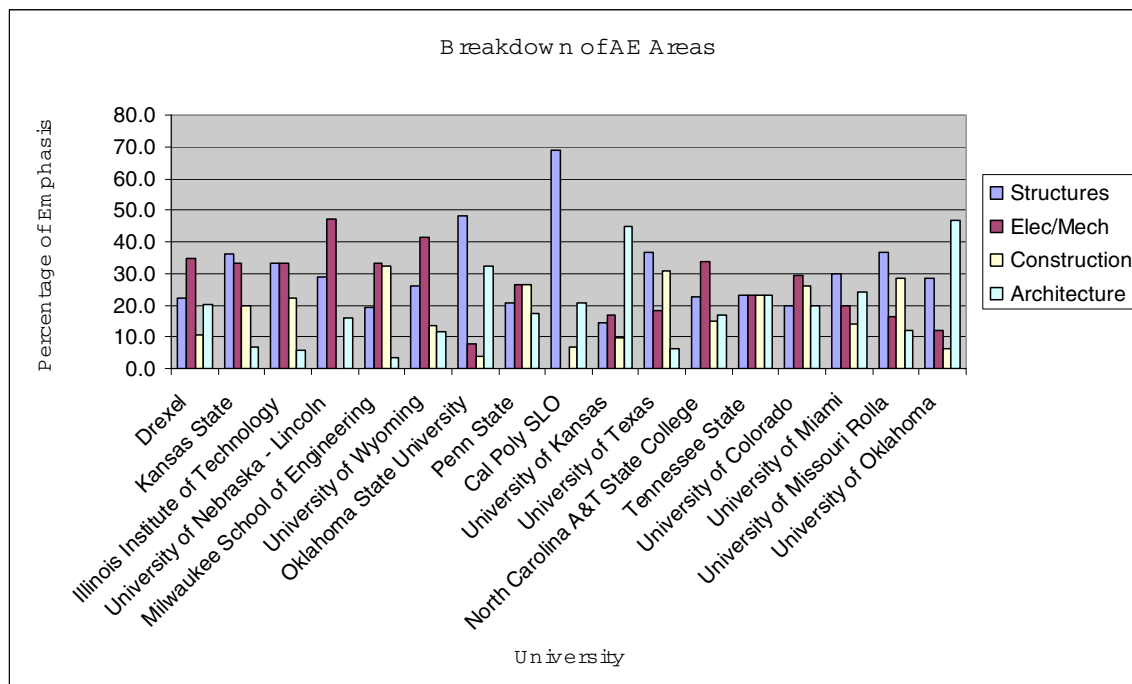


Figure 4. An analysis of the relative emphasis placed on structures, electrical/mechanical systems, construction, and architecture in the accredited AE programs

Figure 5 shows the relative emphasis as a percentage of the total curriculum that each AE program devotes toward communications, humanities, math and basic science, and engineering science. Most programs devote 20-30% of the curriculum to math and basic science and 10-20% of the curriculum to engineering science. Tennessee State stands out as offering a greater percentage of its curriculum to communications courses (12%) than the remainder of the programs.

Why is this important?

When the AE programs are so diverse, it can be difficult to find common ground and collaborate in an effective manner. One might ask why this is important. Some diversity in

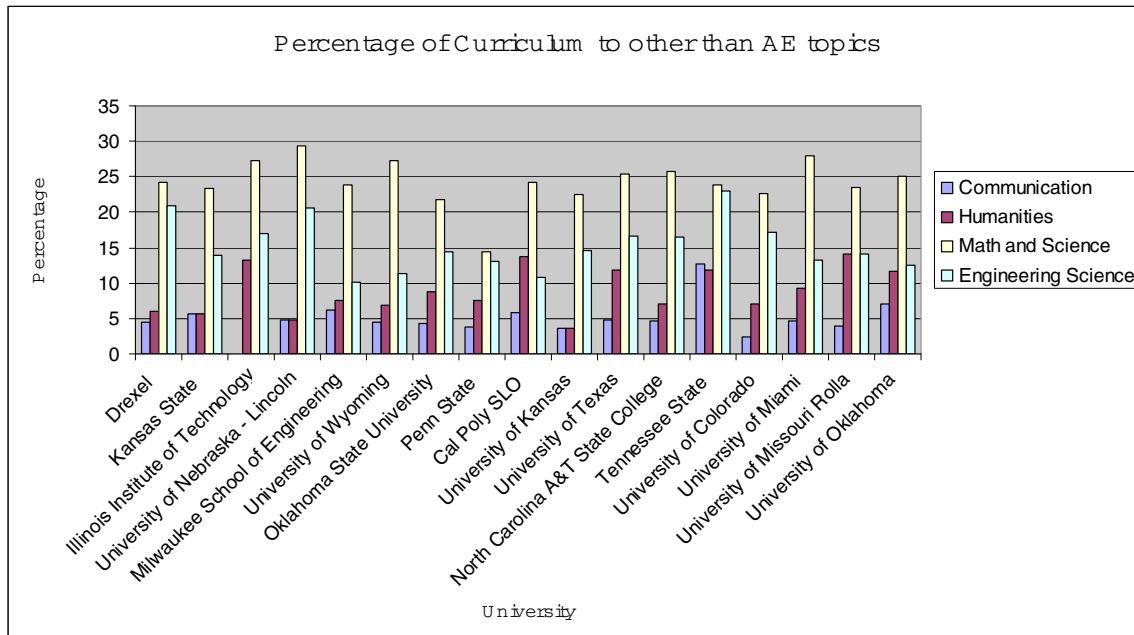


Figure 5. An analysis of the relative emphasis on communications, humanities, math and science, and engineering science in the accredited AE programs.

programs should be embraced and one would not want all of the programs to be the same. While these differences in emphasis in the AE specialties allows more choice for prospective students, there is some need for commonality, collaboration, and consensus among the nation’s AE programs for some of the following reasons.

- Because there are only 17 accredited AE programs, it is easy for their needs to be overlooked. If the programs can develop a common position on many issues, it will be easier to communicate this perspective forcefully to the rest of the academic community.
- The AE programs need to provide input and attempt to reach consensus toward what the AE program accreditation criteria should be. In many ways, the accreditation criteria are the biggest contributor to the degree of commonality that currently exists among AE programs. All seventeen programs offer the history of architecture probably because the accreditation criteria require it. The common list of engineering science courses is at least partially attributable to the accreditation requirements. The differences in the structural, electrical/mechanical, and construction offerings are present because the accreditation criteria are flexible enough to require proficiency in only two out of three of these sub-disciplines. Because the accreditation criteria are so important to the content of AE programs, the various programs all need to be providing input to both the criteria and the commentary that supports it.
- The American Society of Civil Engineers (ASCE) is the lead professional society for writing the program accreditation criteria for both civil engineering and architectural engineering programs. In 2005, there were 213 colleges and universities offering civil engineering degrees (ASEE 2005), a number that brings overpowering influence and dominance of effort toward civil engineering programs over architectural programs. The Academic Council of the ASCE Architectural Engineering Institute (AEI) is the primary means by which the various AE programs confer and discuss issues that affect their

respective programs. There are many other professional society venues where the AE programs do not appear to be well represented, but that is starting to change. The ASCE Educational Activities (EdAc) division has recently welcomed the AE programs onto their committee by adding a liaison that directly represents the AEI Academic Council. The AE programs have also been granted membership onto three of the critical EdAc subcommittees: the Committee on Student Activities (CSA), the Committee on Curricula and Accreditation (CC&A), and the Department Head Council Executive Committee (DHCEC). CC&A writes the accreditation program criteria and previously focused exclusively on civil engineering programs. The DHCEC is a committee of civil engineering department heads that provides oversight and advice to the ASCE Educational Activities division. The AE programs will benefit greatly through this greater participation in the various ASCE educational activities.

- The civil engineers are leading ground-breaking initiatives that will ultimately affect AE programs. ASCE Policy 465, which makes the masters degree the first professional degree in civil engineering (ASCE 2001), is making great strides toward completion. The National Council of Examiners in Engineering and Surveying (NCEES) recently voted to amend the model law to require thirty hours of coursework above the bachelor degree for professional licensure (Barrett 2006). Graduates of AE programs will inevitably be affected by this as many states will certainly adopt the model law into their own professional licensure requirements. Up to this point, AE programs have provided little input or participation in this effort.
- The civil engineers have published a body of knowledge (BOK) (ASCE 2004) that lists the skills and attitudes that civil engineers are expected to attain. These include additional skills beyond those required by ABET, such as leadership, project management, business policy and ethics. The new ABET civil engineering program criteria (ASCE 2005) have been approved by the Engineering Accreditation Commission and will go into effect in the 2008-2009 accreditation cycle. The AE program criteria will likely be affected by these initiatives in the future.
- Many prospective students do not understand what architectural engineering is and are confused as to how it differs from architecture. When the available programs have widely different areas of emphasis, it becomes more difficult to communicate what constitutes architectural engineering.
- Many engineering majors and their professional societies sponsor student chapters and conduct student regional conferences that hold student competitions. With the exception of the Midwest, it is hard for AE programs to conduct a regional conference due to the small number of programs. A standardized student competition has not been developed because the AE content varies so widely among programs that it is difficult to devise a competition in which all programs could fairly compete. Nevertheless, the AE representation on the ASCE CSA will allow AE programs to take advantage of the training and regional student conference structure that is already in place for CE programs.

Conclusions

It is to the advantage of the seventeen accredited AE programs to continue to meet, discuss and develop unified positions on the various issues that affect their programs. The AE programs

should become even more involved in the ASCE educational activities committees and with the civil engineering programs. Because there are so many more civil engineering programs, the AE programs should join with the civil engineering programs whenever it is to their advantage and receive the increased exposure and influence that larger disciplines provide. Still, the AE programs should retain their individual identity as a separate discipline, also when it is to their advantage. This paper has shown that the various AE programs are very similar in some regards and very different in others. The ABET program accreditation criteria seem to be the mechanism for enforcing standards of commonality and for allowing the flexibility to be different. The various AE programs should all be actively participating in the drafting of these program criteria.

Disclaimer: The opinions expressed in this article are those of the author and do not necessarily reflect the viewpoints of the AEI Academic Council, ASCE, or the AE programs listed herein.

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March 18, 2008.

From CAD to BIM: Constructing Opportunities in Architectural Education

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Abstract

Building Information Modeling (BIM) continues to transform the discipline of architecture, both in concept and practice, with the promise of streamlining the design and construction process. As opposed to 2-D CAD, 3-D BIM allows for almost limitless opportunities with respect to design, coordination and the sharing of information by architects and their consultants. The abstract nature of 2-D slicing is increasingly being replaced by full 3-D representations, allowing greater participation from stakeholders previously alienated from the building process. Similarly, BIM in architectural education continues to gain acceptance as a tool for demonstrating the comprehensive design abilities of students. 3-D representations of studio work, as opposed to traditional slicing in the form of plan section and elevation, allows for an increased depiction of the various systems employed and in turn a greater understanding of building tectonics and systems integration.

While the application of BIM as a comprehensive design tool continues to be utilized, this paper examines where BIM may also serve as a transformative technology and methodology within the architectural curriculum at Montana State University School of Architecture, primarily as a tool for a greater understanding of building materials, assemblies and systems as well as for building systems integration and technical documentation. Through a review of the technical requirements outlined in the NAAB criteria for accredited schools of architecture, of the current curriculum and of the student outcomes over the past 4 semesters, this paper documents the successes and shortcomings of BIM integration in the technical course sequence at MSU. The goal of these investigations is to better understand the pedagogic value of BIM in architectural education specifically in the areas of materials, assemblies and systems integration and better prepare students of architecture to influence the inevitable paradigm shift from 2-D slicing to 3-D integration.

From CAD to BIM: Constructing Opportunities in Architectural Education

‘Once it takes hold in architecture, this technology will accord unprecedented authority and advantage to all who can project it.’¹

It would appear from the above quote that the BIM revolution is finally upon us and gaining momentum throughout the entire building industry. After years of articles purporting the benefits of this technology ‘on the horizon’ to the AIA’s continued BIM awards since 2005 and the 2006 ‘*Report on integrated practice*’ to the most recent launching of the *Journal of Building Information Modeling*, BIM technology and the holistic context of integrated practice that it enables, will continue to increasingly dominate the industry.² The advantages of BIM, as both a digital tool and information management system, have been well documented in articles and books, appearing with increased frequency over the past three years. Initially lauded as a tool for the streamlining of construction³, BIM has quickly become the tool of choice for architectural practices, allowing possibly, for the developer driven maxim ‘better, faster, cheaper’ to move closer to a reality. BIM as an information management system combined with virtual modeling has created opportunities for the ‘front loading’ of the design process, providing better decision making earlier in the process allowing for increased clarity of information as well as increased collaboration within the design process to include all members of the project team.

Imagine a world where all communications throughout the process are clear, concise, open, transparent, and trusting; where designers have full understanding of the ramifications of their decisions at the time the decisions are made; where facilities managers, end users, contractors and suppliers are all involved at the start of the design process; where processes are outcome driven and decisions are not made solely on a first cost basis; where risk and reward are value-based, appropriately balanced among all team members over the life of a project; and where the profession delivers higher quality design that is sustainable and responsive. This is the future perfect vision of Integrated Practice.⁴

This description of integrated practice illustrates the possible advantages of technology merging with the processes of the building industry. Fundamental to this discussion of integrated practice is the role of BIM and the paradigmatic changes it has catalyzed for firms of all sizes. Structural and mechanical programs allow for other trades to coordinate with the architectural programs and the front loading process means that large or small a/e firms can access current tools including ‘energy use simulations, quantity takeoff, cost estimating, construction planning and various types of engineering analysis.’⁵ Within this new frontier

of industry-driven technology, it is now appropriate to ask how education will be able to meaningfully participate in the future of BIM and integrated practice in any way other than merely teaching students the appropriate tools in the form of the computer program *du jour*.

This paper will examine one current role of BIM in architectural education, as a comprehensive design tool in design studio applications, and argue for an increased role of BIM in other areas of architectural education, including technical documentation and systems related courses, based on the escalating use of BIM technologies and methodologies in practice today. This paper will analyze first, the current use of BIM in the design studio sequence at the Montana State University School of Architecture, second, the technical requirements for accreditation outlined in the NAAB criteria to possibly determine other possible locations for BIM within Schools of Architecture and finally the student outcomes related to BIM applications taught outside of the design studio in technical documentation and systems related courses over the past two years. The intent is to identify the successes and shortcomings of these recent BIM implementation strategies in technical coursework, to illustrate the pedagogic value of BIM as a holistic design tool in architectural education and to prepare students of architecture for the inevitable use of BIM in practice.

BIM as a transformative technology within the architectural curricula

For several years now at the Montana State University School of Architecture, 3-D, BIM-type models have been constructed by students in upper division design studios to depict the overall intent and inner workings of their respective projects.⁶ (Fig. 1.) These models, created in a variety of programs including Revit, Rhinoceros and ArchiCAD, that may or may not include BIM-type building component identification, are complete 3-D constructs that illustrate all of the major building systems including the structure, mechanical, enclosure and interior systems as well as diagramming sustainable systems and programmatic relationships.

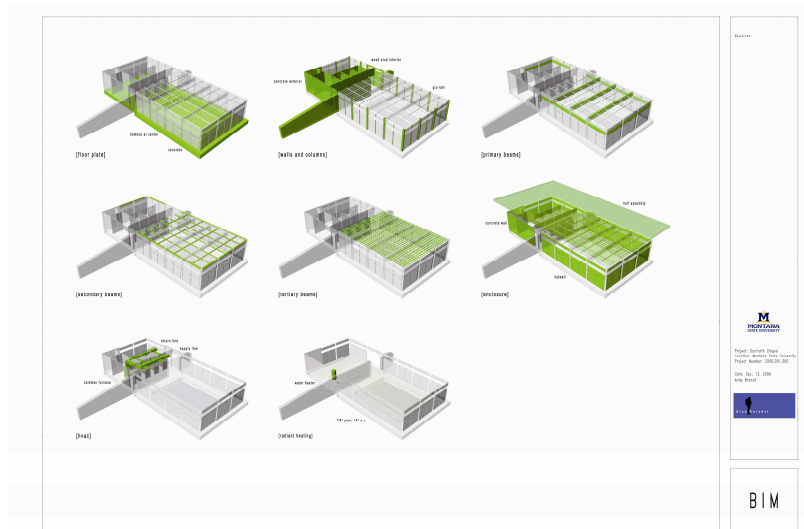


Figure 1. Building Information Model (separated by system), Student: Andy Brandt

Developed through the course of the semester, these building models can be an effective and powerful tool in final critiques to illustrate the ability to integrate various building

systems into a complete architectural project. In many respects, this use of BIM in the studio is similar to the use of BIM in practice as a tool for the front loading of the design process as described above. In general, many, if not all, student design presentations depict a project roughly at the schematic design or design development phase which is similar to the point in an actual project, outside of academia, where it would be presented to the client for design approval. In this way BIM resembles, in academia, the world of practice.⁷

With the construction of the project in virtual space, professors and stakeholders alike are able to understand the building as a complete whole and not a series of 2-D drawings that have to be mentally reassembled in order to understand to whole. In addition, the modeling of building systems, structure, mechanical ductwork and piping, can also be studied to assess the impact on the interior and exterior spatial configuration of the project. The developed program, spatial organization and relationship to the surrounding environment can also be critiqued by professors and stakeholders, again without the challenges of interpreting 2-D orthographic drawing. With the model complete, students can perform daylight studies, energy calculations and assess the fluid dynamic qualities of ventilation schemes similar to the way geospatial information gives practitioners a real world perspective for the development of actual buildings. In this aspect of BIM development, as a vehicle for visualization and communication of building information, the relationship of academia to the profession is strikingly similar. BIM, taught in this academic setting will ultimately impact the practice of architecture as graduating students with these abilities and the advancing computer technologies merge. While this has been an effective way to integrate BIM into the curriculum, we have been exploring other ways in which BIM can be integrated into our curriculum; specifically that of integration into the construction documents courses where issues of material, construction and configuration of building systems and details occur.

Accreditation Standards and BIM, the right fit?

With the escalating adoption and celebrated benefits of BIM in both architecture and construction practices, it is essential that BIM and the broader concepts of integrated practice be consolidated into university curricula. Naturally questions arise regarding how and where the best opportunities lie for integration. One form of integration, in an institution-wide manner, may appear to be the most logical choice of adoption (sprinkle small doses of BIM throughout a curriculum so that all students are affected) but, other, more strategic, alternatives may be realized by investigating the existing standards of accreditation that establish the performance criteria for accredited degree programs of architecture.⁸

The National Architectural Accrediting Board (NAAB) is the body which evaluates the performance of, and grants accredited status to, architectural programs in this country. Through an evaluation criterion that includes 37 knowledge and skill areas, the NAAB establishes a minimum performance standard to aid accredited programs prepare students for the profession. NAAB is also associated with the National Council of Architectural Registration Boards (NCARB) which, in addition to writing and administering the Architectural Registration Examination (ARE), 'establishes national standards for certifying qualified architects'.⁹ Simply put, the NAAB accreditation status of a program allows the graduates of that program to sit for the ARE licensing exam and is therefore essential to the

licensing of architects and the regulating of the architectural profession. While not essential with respect to licensure, the exposure to BIM can be seen as essential to the future health of the profession.¹⁰ It is within a review of the NAAB knowledge and skill areas that we may find opportunities for the deployment of BIM as a tool for greater understanding of building systems integration.

Currently BIM is used at the Montana State University School of Architecture as a tool to possibly demonstrate two of the NAAB criteria, '*building systems integration*' and '*comprehensive design*', where students are asked to demonstrate an ability to integrate various building systems within their design work.¹¹ This is primarily accomplished in the upper level design studios where the various service systems of a project are illustrated in relation to one another within the context of an architectural design. So where might BIM also be incorporated into the curriculum where it responds to the needs of industry in a way that might impact not only the future of integrated practice and the front loading of the design process but also impact a profession that is transitioning from 2-D to 3-D with regard to building representation? With regard to the more technical aspects of BIM including object properties and relationships, one logical choice may be in the areas of '*Building Materials and Assemblies*' and '*Technical Documentation*'.¹² In these areas, which stress the relationship between materials and system assemblies, we may see the opportunity for BIM utilization in the realm of 3-D detail drawings and construction documents.

The role of BIM in technology courses

Within current practice, and projecting how practice might be transformed by BIM, the placement of BIM-based investigations in technical courses such as construction documents addresses larger issues of architectural representation. We have to question how long the current utilization of BIM as a virtual 'test building', constructed before the actual building from 2-D drawings, or as a tool to construct a 3-D building and then represent in 2-D construction documents will continue with the rapid advances in technology.

These issues of representation and technology regarding BIM frame the basis for the course described below, Comprehensive Architectural Project, a four hundred level course taught as the construction documents component of our curriculum with an emphasis on BIM and material detailing. The overall goal of the course is to encourage students to consider the materials and systems of their design work and then document this work in the form of BIM-based construction documents with a focus on constructability and clarity of information. The course is structured using three forms of inquiry; the first being the creation of a schematic design and initial building information model that forms the basis for further development. Second, the development of details illustrating materials, connections and relationships derived and integrated from the initial building model and finally, these areas of investigation are used in the creation of an abbreviated construction document package, incorporating BIM to its fullest extent throughout. Where BIM forms a crucial role in this course, and possibly an expanded role in the profession, is through the formulation and construction of details that integrate into the larger information model, forming a critical relationship between the role of 2-D and 3-D information.

At the onset of the semester, the students are asked to create a schematic design for a small building. This varies from semester to semester but recently the project has been a small

campus structure including a non-denominational chapel and a materials testing laboratory for college of engineering research. In all cases the project, scaled at approximately 2,000 square feet, allows each student the opportunity to create a conceptual building design and then to thoroughly consider the materials, details and systems of construction over the course of the semester. The initial three week schematic design period culminates with a required building information model to illustrate the basic enclosure, mechanical and structural systems employed in each project. This initial BIM also allows a framework for speculation and consideration of additional detail and systems integration throughout the semester.

During the schematic design period, additional exercises concerning detail drawing and material identification are interwoven to continually emphasize the holistic, integrated nature of architectural design and construction. The first exercise, issued concurrent with the schematic design is the creation of a detail drawing, intended to investigate the potential of the ‘fertile detail’ as described by Marco Frascari in his essay *The tell-the-tale detail*.¹³ Utilizing the ‘fertile detail’ concept as a source of inspiration, it is hoped that the students will allow the implicit material choices and the construction of the detail to influence the partially completed schematic design. Detail in this way becomes a generative source for the overall design and not merely a collection of parts. The students are asked to create this detail in 3-D, illustrating both exploded and constructed views of the particular materials used and indicate connection points and strategies for the sequencing of construction. (Fig. 2) The detail is constructed as an isolated aspect of the overall design but is meant to relate to the overall model in the same way that details in a construction document set relate to the overall. The critical difference in this case is that the details are not in 2-D but in 3-D with in many cases, object properties to provide additional information for a future database.

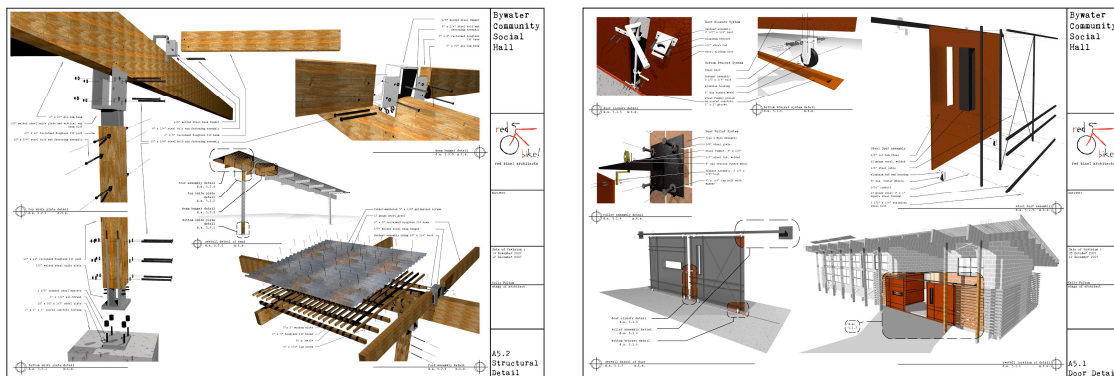


Figure 2. Construction Documents, Student: Kelly Fulton

An additional detail drawing is assigned at the end of the schematic design period to further emphasize the role of detail and material in their work. These details correspond directly with the larger BIM and indicate the potential of BIM to not be a supplement to the 2-D construction documents but to be the construction documents through a merging of 3-D drawing and construction information.

With the schematic design and detail drawings complete, the students are then introduced to specifications and are asked to create an outline specification for their project after first completing a list of all the materials in their project using the CSI format. With the

specifications complete the students continue on to complete their set of construction documents. In this process, BIM forms a central unifying element as the initial 3-D model is enhanced and presented, illustrating in isolation, the building systems of structure, mep, enclosure and interiors.¹⁴ Throughout the semester the students are encouraged to research the various building systems employed and integrate them into the building information model. This occurs during the schematic design phase, during the construction of the details as well as during the creation of the specifications and building section phase of the construction document production. As the class works towards the completion of an abbreviated construction document package due at the end of the semester, they are also encouraged to take advantage of the building information model for further integration. Select views of the model, rendered with material mesh are encouraged for exterior elevations, building sections, wall sections and additional detail drawings in the package.

Realized outcomes

Over the past 4 semesters, we have seen an increased reliance in the use of BIM and BIM-related drawing conventions in the Comprehensive Architectural Project course. While there are shortcomings related to this endeavor there are also many positive results. Firstly, students continue to explore new ways of illustrating construction details and methods for incorporating these details into the overall construction documents. Exploded details referenced off of the larger BIM illustrate a 3-D vision of what is currently expressed in the industry with 2-D slicing. In addition, the students continue to develop an understanding of the configuration and construction assembly of various building materials through 3-D representation. This has allowed the students to explore systems and assemblies as a whole and not as a series of sectional slices. This allows for increased knowledge of building construction and the ability to actually construct systems and assemblies out of elements and not as a series of lines on the screen. The production of details in 3-D has also allowed the students to see opportunities for greater communication, making the case for an increased utilization of BIM both in academia and in practice. Finally, the explorations of BIM as a platform for construction documents allows the students to mentally engineer a vision for the eventual paradigm shift from 2-D documents to full 3-D digitally based construction documents. While current technologies to make this possible may be in the future, these studies reveal the opportunities offered by BIM technology.

With regard to the shortcomings of the course, in general, software applications are a significant problem as we look to BIM and the creation of details and integration into construction documents. While BIM programs such as Revit exist, here doesn't seem to be a way for the detail to become a part of the larger project which means that there is a disjunctive relationship between the detail and the overall project. In this respect many details drawn by students are constructed in programs without the ability to create object types of the elements modeled. An additional shortcoming is related to the current curriculum which doesn't teach the comprehensive tools needed for students to be competent with BIM applications or the necessary building construction courses for the students to be conversant with building materials and systems. The one computer graphics course offered in the program covers many programs over the course of the semester with a limited time spent on each application. While three dimensional programs are part of this course they can't be taught in a comprehensive manner and essentially become a primer for

additional development in the studio or on the students 'free' time. The lack of significant preparatory courses places pressure on the course to not only teach the principles of construction documents and detailing but to also provide resources for students to accelerate their knowledge of construction and BIM applications. It is safe to say that the positive results are currently outweighing the shortcomings, which reflects a building model ready for the future.

Conclusions

It is unknown at this point how far away the industry is from a fully integrated BIM construction document packages where the BIM forms the primary tool of investigation and dissemination of all construction information. This may be unrealistic as well with the potential fallout from an entire industry wide paradigm shift. Clearly BIM is not being used as an integrated tool for construction documents. As we look to this possible shift, we can begin to educate students in the possibilities of this world by illustrating how detailing and material and system integration can come together in a 3-D environment. We can look to the current manifestations of BIM as both a tool and an information management system to provide a structure for students of architecture to investigate and understand building materials and systems integration and prepare students for the inevitable shift from 2-D slicing to full 3-D integration.

¹ Daniel S. Friedman, 'Architectural education and practice on the verge', Daniel Friedman, ed., *Report on integrated practice*, (AIA, 2006). Friedman's comments, from his introduction to the Report on integrated practice speak to the power of BIM technology in practice.

² The AIA's Technology in Architectural Practice (TAP) Knowledge Community has been holding the BIM awards since 2005, celebrating the best in BIM technology related projects; the 2006 *Report on integrated practice* was an anthology providing multiple viewpoints related to the concept of Integrated Practice, a concept largely centered around BIM; and the *Journal of Building Information Modeling* is the official publication of the National BIM Standard (NBIMS) and the National Institute of Building Sciences (NIBS). These awards and publications represent the AIA's commitment to BIM and the architects continued role in the building industry and society at large.

³ Larry Flynn, 'Getting on Board with Building Information Modeling', *Architectural Record*, (April, 2006): 163-7. This article provides a clear description of the advantages of BIM to the construction process, specifically focusing on the steel industry.

⁴ Norman Strong, 'Introduction', in Daniel Friedman, ed., *Report on integrated practice*, (AIA, 2006).

⁵ Richard See, 'Building Information Models and Model Views', *Journal of Building Information Modeling*, (Fall, 2007)21.

⁶ It should be noted that the student's use of BIM, both in theory and practice, is based on computer tools, i.e. Revit, ArchiCad, Bentley or other programs, and not on the concept of managing information. It should also be noted that BIM in this paper also references computer applications as utilized by students unless noted otherwise. This common misunderstanding is best described in Finith Jernigan's, *Big BIM little bim*, (MD: 4Site Press, 2007)23.

⁷ The major difference being that in academia the project at the schematic design phase is effectively complete and in practice the project moves into the construction document phase and is hopefully built.

⁸ The argument being that in smaller programs of architecture, with limited faculty and resources, there is little room in which to teach courses or content not specific to the accreditation standards.

⁹ NCARB, *The Regulation of Architecture in the United States*, (Wash. D.C.: NCARB, 1999) 2.

¹⁰ This has been noted, most emphatically by Thom Mayne in his remarks on BIM at the 2005 AIA convention in Las Vegas. See Thom Mayne, 'Change or perish', *Report on integrated practice*, (AIA, 2006).

¹¹ The NAAB Criteria states: "23. Building Systems Integration - *The ability to assess, select, and conceptually integrate structural systems, building envelope systems, environmental systems, life-safety systems and building service systems into building design*" and "28. Comprehensive Design - *The ability to produce a comprehensive architectural project based on a building program and site that includes development of programmed spaces demonstrating an understanding of structural and environmental systems, building envelope systems, life-safety provisions, wall sections and building assemblies and the principles of sustainability.*"

¹² The NAAB Criteria states: “24. Building Materials and Assemblies – *Understanding of the basic principles and appropriate application and performance of construction materials, products, components, and assemblies, including their environmental impact and reuse.*” and “26. Technical Documentation – *Ability to make technically precise drawings and write outline specifications for a proposed design.*”

¹³ Marco Frascari, “The Tell-The-Tale Detail” from Kate Nesbitt, editor, *Theorizing a New Agenda for Architecture*, (NY, Princeton Architectural Press, 1996):500-514.

¹⁴ This is taken from the criteria laid out in Richard Rush’s book *The Building Systems Integration Handbook* and Leonard Bachman’s book *Integrated Buildings*. These also serve as readings throughout the semester, looking at the ideas of integrated building, stressing the importance of integration in the success of architecture.

From CAD to BIM: Educational Strategies for the Coming Paradigm Shift

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Abstract

The rapid movement from Computer-Aided Drafting (CAD) to Building Information Modeling (BIM) by professional architects and engineers creates several challenges and opportunities for Architecture and Architectural Engineering programs. The BIM methodology offers dramatic new benefits to students that university programs have just begun to explore, but it also will likely trigger trade-off considerations for traditional skills that might be lost. Some educators worry that new BIM activities pose a threat to design thinking.

This paper surveys some of the major challenges and opportunities that BIM presents in educational settings, with several suggestions for future directions for exploration. These findings are discussed in the context of several key conclusions that have been developed based on six semesters using BIM in junior- and senior-level architectural design studios in an undergraduate program in Architectural Engineering. The paper remarks on accreditation issues, and it seeks to develop potential 'best practice' hallmarks with the goal of stimulating future discussion.

Keywords: *Building Information Modeling, BIM, engineering education, architecture education.*

1 Introduction

By now it is widely accepted that Building Information Modeling (BIM) will radically transform the discipline of Architectural Engineering and its allied fields [NBIMS, 2007]. The activity of parametric modeling is fundamentally different from drawing, because the product is a database of information and relationships (new paradigm), rather than a set of abstract representations to be interpreted (old paradigm). Thus the move from

Computer-Aided Drafting (CAD) to BIM constitutes a new methodology, rather than the simple introduction of a new tool.

BIM therefore threatens to disrupt traditional design education, both in terms of large curricular issues and specific teaching methods. Is BIM “inherently answer-driven,” threatening critical (design) thinking, or does it in fact promote more sophisticated design via its ability to run simulations? How will the accrediting bodies address the paradigm shift from 2D drawing to 3D modeling? How should Architecture and Architectural Engineering programs begin to grapple with these questions?

2 Previously Published Conclusions

Following are some key conclusions we have developed based on six semesters using BIM in junior- and senior-level architectural design studios in an undergraduate program in Architectural Engineering.

2.1 Integrated Design

BIM prompts students to think about architecture, structure, and mechanical systems in an integrated manner, and to consider issues of materiality and construction at an earlier stage of the design (compared to traditional 2D design). Prior to BIM, we observed that students had difficulty appreciating the architectural consequences of structural decisions, and vice-versa. In the old paradigm, students would develop an architectural plan and then overlay a structural framing plan where individual members were represented by single lines, but relationships between systems were under-developed three-dimensionally. In the new paradigm, students gain a clear appreciation for the advantages of an integrated design process because they, in effect, *build* the building rather than representing it abstractly, which requires them to consider how systems interact in space [Denzer & Hedges, 2007].

2.2 Shifting the Curve

Students using BIM experience an accelerated design process compared to those using CAD or other ‘traditional’ methods. BIM allows them to develop their projects to a greater degree of resolution over the course of a semester. In professional practice, BIM has transformed the proportions of time dedicated to the schematic design (SD), design development (DD), and construction documents (CD) phases, giving more time to SD. This is known as ‘shifting the curve to the left’. In the classroom, we have experienced a different shift. Architectural design studios commonly work exclusively in SD. But for students BIM actually shifts the curve to the right, by allowing them to address design questions that are generally associated with the DD phase [Hedges & Denzer, 2007c].

2.3 Conceptual Complexity

Students using BIM often choose to pursue designs that are more complex compared to designs composed exclusively with CAD, such as exploring eccentric geometries. One representative team reported: “We were able to solve many problems by being able to see our project in 3D right away. Drawing in 2D limits you on how well you are able to visualize the project.” These students reflected that they needed 3D representation to

develop their idea and would never have attempted such a challenging design without BIM [Hedges & Denzer, 2007d].

2.4 'In-Process' Visualizations

BIM allows for a more robust exploration of design alternatives 'in process', permitting students to simulate the effects of design alternatives in order to make more intelligent and persuasive decisions. The BIM environment is more conducive to identifying new opportunities and remedying unintended effects. 97% of the students surveyed (n=30) thought that BIM improved their understanding of form and space by working in one 3D model database. One student noted that "it allows you to constantly assess your work" [Hedges & Denzer, 2007b].

While it is important to use freehand sketching for the earliest conceptual work, there is little danger in introducing BIM very early in process. BIM's ability to rapidly generate photorealistic renderings naturally raises anxiety for instructors; will students be 'seduced' and overly attentive to the production of images at the expense of good design? We concluded that, with proper guidance, BIM does not lead students to see a preliminary design as 'finished'. Instead they are comfortable producing variant massing models and then studying and refining the alternatives [Denzer & Hedges, 2007].

2.5 New Models of Collaboration

The BIM methodology includes a powerful capacity to facilitate new models of collaboration for student teamwork. We have found that, when students chose a collaborative architecture approach using real-time simultaneous engagement, they experienced a greater conceptual complexity and an advanced level of refinement. Higher intellectual cognitive activity is essential in an open-ended design course where problems require team collaboration. For Architectural Engineering programs, accredited by ABET, BIM responds to the demands for multidisciplinary team activities and continuous improvement by providing a platform for exploring new team structures and realizing improved student outcomes [Hedges & Denzer, 2008].

3 Pedagogical Issues

3.1 Challenges

Because BIM introduces more sophisticated questions (particularly regarding construction) earlier in the design process, students without an adequate grounding in the fundamentals may be ill-served by the software's parametric capabilities. As Cheng [2006a] argued: "Never has a representation tool been so demanding of its user. The competent BIM operator must have an understanding of the tool, knowledge of materials and construction methods, and appreciation for professional practice." In surveys of our students, we found that only fifty percent responded that BIM improved their understanding of how construction materials are assembled [Hedges & Denzer, 2007c]. In the worst-case, students may be tempted to make decisions rapidly from a limited palette of default options.

The positive notion that BIM introduces a greater degree of conceptual complexity (see 2.3 above) derived from the experience of upper-level undergraduates. At a lower level, by contrast, increased complexity may hinder student learning rather than help. Scheer [2006] noted that BIM software programs are significantly more complex than CAD tools, and thus “BIM requires space in the curriculum that CAD does not. What do we sacrifice to make room for teaching BIM?” The authors have also observed that novice students tend to be overwhelmed by the software if it is not introduced gradually and systematically. Therefore, the insertion of BIM in an educational sequence should be carefully calibrated with prerequisite courses and student intellectual maturity.

Perhaps the biggest challenge for design instructors is that BIM demands new teaching methods. “The transformation of the traditional linear architectural education process into one more elliptical—by incorporating BIM as a process and not a tool—is the single most difficult challenge (or exciting opportunity, depending upon how you look at it) at hand for today's educators” [Seletsky, 2006]. Our experience at the University of Wyoming has prompted two major findings in this area. First, BIM can ‘disguise’ an underdeveloped design by giving it an appearance of resolution; instructors must probe the students’ process in order to distinguish between purposeful decisions and default selections. Second, BIM prompts students to ask advanced questions about structures, material assemblies, and detailing (even if they have had prerequisite coursework in these areas), requiring instructors to be relatively more agile in their ability to respond.

3.2 Barriers

Although this research presumes that Architecture and Architectural Engineering programs should (and will) introduce BIM in order to be responsive to their constituents (students and employers), there are barriers at the institutional level. Simple uncertainty is the first hurdle. Will BIM endure, or is it another fleeting disruption? What happens to subjects such as descriptive geometry and 2D construction documents? Is the new paradigm implicitly critical of the old paradigm, raising territorial issues among faculty? What time commitments are required of instructors to learn the software? We do not suggest that these questions have prescriptive answers, nor that recalcitrant faculty will simply submit to BIM after a proper indoctrination. Some of the uncertainty should be eased as early adopters publish student work and reflect about their experiences.

A second barrier, which might be called ‘cultural’ resistance, will require more thought and discussion at the level of institutional decision-making. One stance is based on the precept that BIM constitutes a “threat” by crowding out critical thinking. Cheng [2006a] argued: “There are two competing philosophies: BIM is inherently answer-driven, design thinking is question-driven.... If BIM is introduced in the curriculum without respecting its considerable liabilities, design thinking will not survive.” (Seletsky [2006] responded that BIM promotes more sophisticated ‘design thinking’ because it allows students “to then simulate their decisions in validating—and not just positing—what they're proposing,” thus acting intuitively *and* analytically. This view is consistent with our conclusions summarized above.) Structural engineering programs might justifiably worry, for example, that automatic sizing of columns and beams will render a fundamental disciplinary skill obsolete. Each academic program should consider these arguments vis-à-vis their objectives, their student needs, and the reports by early adopters.

Thirdly, institutional resistance to BIM arises because the software is expensive and may have operability problems in some cases. “Current BIM technology lacks uniform standards and is changing very rapidly,” according to Scheer [2006]. “The skills students acquire today on a particular platform are useless on another, and in any case will be obsolete in a few years. How can a curriculum prepare students in BIM so that their knowledge continues to serve them in such an environment?” Furthermore, we have noticed that students expect to have access to the newest applications and most current platforms, which requires more work, more deliberation and, potentially, more resources.

3.3 Future Opportunities

BIM’s challenges and barriers for educators are dramatically offset by potential benefits to students that university programs have just begun to explore. Following are some educational topics that can be addressed in a new manner as a result of BIM:

- Distance collaboration. In professional practice, members of the design team are able to collaborate remotely in real-time by working on a single BIM file. A similar methodology could be explored in academia by organizing cross-institutional design teams. Collaboration between Architecture and Architectural Engineering students would be especially fruitful. Any such activity should take note of Cheng’s [2006b] observation for architecture education: “Collaboration in its professional sense is hard to simulate in an academic setting. Professional collaboration forms among participants who have clearly defined (and complementary) roles, responsibilities and expertise. Collaborators come to the table with experience and maturity gained over many years of practice.” (See section 4.2 for how engineering accreditation already embraces collaboration.)
- Code compliance detection. BIM programs include an automatic detection feature where code-violations, such as insufficient clearance around a door swing, are automatically highlighted. Instructors can use this feature as a teaching tool, as well as for assessment.
- Structural analysis. BIM programs can interface with third party structural analysis software so that a design can be analyzed, given the appropriate parameters (member sizes, loads, soil conditions, etc.). Ideally this capability will allow structural designers to make more critical and creative decisions by rapidly testing alternatives with interoperable software as opposed to the historical recreation of an additional 3D computer model for design and analysis.
- Advanced energy use simulations, including life-cycle performance. The BIM methodology can improve the design process for MEP engineers by helping them test equipment alternatives through simulation, leading to better solutions (“right-sizing”). It will help students develop longitudinal thinking from a facilities-management point of view, prompting solutions that address the expected life-cycle of the project.
- Economics. Students may employ the software’s ability to compute material quantities, and correlate them with suppliers’ pricing (possibly real-time via the

web), which would provide opportunities for comparative economic studies between various design alternatives. (The old paradigm certainly considered economics, but BIM now allows greater specificity earlier in the process.)

- Construction staging and scheduling. Many in industry believe that BIM's true power is to streamline construction by simulating building processes in advance, thus anticipating conflicts and reducing errors. In the classroom, BIM certainly has the power to help students explore alternative methods of building their design, leading to a critical evaluation feedback loop that might not have been possible in the old paradigm.
- Digital fabrication. As academic programs pursue design-build studios, some will surely explore how BIM can be used to interact directly with fabricators and other subcontractors to remove the well-known inefficiencies of 2D documentation and interpretation.
- Programming. As Seletsky [2006] notes, students already have “the moxie and creative proclivity to write their own scripts, to combine or ‘mash up’ their own variety of pre-existing tools, to even go so far as to modify existing application interfaces to suit their own particular needs.” Our students, for example, have begun to explore ‘Excel-based model generation’. Some academic programs will wish to formalize the presence of programming skills in their curricula.

In any given studio or curriculum, time is a finite resource, and so the pursuit of any of these activities will trigger trade-off considerations for what might be lost. For an architectural student, is energy modeling more important than figure drawing? For a structural engineering student, is programming a BIM subroutine more important than surveying? These are the debates that will occur quite soon. Whether new activities like those listed above might crowd out ‘design thinking’ (as Cheng would argue) or enhance it (Seletsky) is, again, a question of institutional philosophy ripe for discussion.

4 Accreditation Issues

Architecture programs are accredited by the National Architectural Accrediting Board (NAAB), while Architectural Engineering programs are governed by the Accreditation Board for Engineering and Technology (ABET). We suggest that curricula evolve independent of the accreditation criteria as to avoid the stifling of creative education, but programs will be challenged to meet the upcoming criteria without acknowledging and employing to some degree the concepts surrounding BIM.

4.1 A Mandate?

Historically, NAAB and ABET have never imposed prescriptive requirements with regard to specific software platforms, nor have they set clear expectations about computing skills. (CAD, for example, is used in a majority of architectural and engineering firms but it is not required to be part of either curriculum, strictly speaking.) The governing approach is that an accredited program should demonstrate responsiveness to the feedback from its advisors in industry. Therefore if BIM is seen

simply as a 'new tool', it can be expected that the accrediting bodies will have little interest in mandating its use. There must be a higher purpose.

The industry term 'Integrated Project Delivery' (IPD) has recently received significant attention. The AIA/AIACC [2007] defines IPD as "project delivery approach that integrates people, systems, business structures and practices into a process that collaboratively harnesses the talents and insights of all participants to optimize project results, increase value to the owner, reduce waste, and maximize efficiency through all phases of design, fabrication, and construction." The association between IPD and BIM is very strong where BIM facilitates IPD. An argument exists in terms of what happened first and what is more important, a 'chicken or the egg' argument. One may argue that IPD evolved as a result to validate BIM as opposed to the greater significance of enhanced collaboration. Regardless of any criticism, BIM has shifted into the IPD domain where it currently has a covert influence in architectural accreditation.

4.2 Multidisciplinary Teams

For architecture programs, NAAB criterion 7 identifies the current requirement for Collaborative Skills: "Ability to recognize the varied talent found in interdisciplinary design project teams in professional practice and work in collaboration with other students as members of a design team" [NAAB, 2004]. Scheer [2006] acknowledges this as a weakness, "While architecture has always been collaborative, current architectural education downplays this fact." NAAB constituents are currently reviewing accreditation standards. The American Collegiate Schools of Architecture (ACSA) provided a commentary to this criterion: "Collaborative Skills should evolve to address the ability of students to both recognize the value of interdisciplinary collaboration and to work collaboratively with students in multidisciplinary design teams" [ACSA, 2007]. The National Council of Architectural Registration Boards (NCARB) similarly commented: "Architectural design, at its very core, is a collaborative process.... Education should...help students understand that collaboration as an intrinsic part of the design process" [NCARB, 2007]. If NAAB follows suit and formulates a collaborative design requirement, more architecture programs are likely to pursue BIM implementation.

The accreditation standards for Architectural Engineering already embrace collaboration as opposed to the individual nature of architectural education through multidisciplinary team learning. The third criterion of outcomes and assessments specifically identifies that engineering programs must demonstrate that students attain an ability to function on multidisciplinary teams [ABET, 2007b]. The ABET Commentary further provides a more succinct example of the individual characteristics within this multidisciplinary setting where, '(1) each team member serves in a well-defined role in the team; (2) each team member brings a particular expertise to bear in solving the problem; and (3) the scope of the problem is sufficiently broad that no one team member could successfully solve the problem alone' [ASCE CCA, 2007]. When students chose an interdisciplinary collaborative approach using real-time simultaneous engagement, they experienced a greater conceptual complexity and an advanced level of refinement [Hedges and Denzer, 2007d]. BIM's strongest attributes are not addressed by the current accreditation standards, suggesting that ABET requirements should be refined for Architectural

Engineering students working on multidisciplinary teams in the new paradigm. This may encourage architecture programs to explore interdisciplinary distance collaborations.

4.3 Sustainability

BIM can also help educators respond to a growing emphasis on issues of sustainability by the accreditation bodies. ABET's current requirement is relatively weak: that all engineering graduates must demonstrate the "ability to design a system, component, or process to meet desired needs within realistic constraints such as economic, environmental, social, political, ethical, health and safety, manufacturability, and sustainability" [ABET, 2007]. It seems reasonable to expect that ABET standards in this area will become more explicit, and that programs will place a stronger emphasis on issues of sustainability in the meantime in order to demonstrate continuous improvement. For Architecture programs, ACSA recently recommended the following (abridged) requirement to NAAB: "The program must demonstrate that it equips students with an informed understanding of ecological and environmental problems in the built environment and develops their capacity to address these problems with environmentally-responsive architecture and urban design decisions" [ACSA, 2007].

Clearly BIM can enhance students' understanding of the environmental consequences of design decisions compared to the old paradigm. For example, BIM applications typically support the ability to export building model data to an energy analysis application using gbXML protocol. Our students have used third-party programs such as Green Building Studio to simulate the performance of designs produced in BIM. The results showed energy use in terms of kWh/sf/yr, which then were compared to 'typical' buildings of the same type as well as case studies of high-performance buildings. Students then modified their designs, ran the simulation again, and studied the consequences. They also computed what 'green power' resources would be required to make the building carbon-neutral. None of this was even remotely possible prior to BIM, and thus it constitutes an advance that will be highlighted in the accrediting process.

5 Discussion: Moving towards Best Practices

Certainly the role of BIM in Architecture and Architectural Engineering education beckons for a robust set of 'best practices' recommendations. Such a project will require many more participants bringing their own conclusions from their own classroom practices. We have developed hallmarks of a 'best practices' educational strategy for the coming paradigm shift with the goal of stimulating discussion and working collectively in the direction of proven practices that benefit students.

Based on our experiences and research, giving particular consideration to the challenges and barriers discussed above, a few suggestions can be put forward modestly:

- Develop an appropriate prerequisite sequence. Educators should carefully consider what prerequisite courses should precede the introduction of BIM in an educational sequence. We would suggest that subjects such as Design Fundamentals (including orthographic drawing), Building Technology/Building Science, and Professional Practice should be considered prerequisite to

comprehensive building design with BIM. This sense of deliberation, however, will be undermined by the use of BIM in secondary education (which is rapidly advancing). Hedges [2007] discusses prerequisite requirements in more detail.

- Continue to use other media. BIM is *one* media that may be used to develop and communicate a design, but students should not rely on it exclusively. Hand sketching, physical modeling, collage, photomontage, orthographic drawing, etc., all remain important parts of the design toolkit. It is essential to prompt students to think critically about the appropriate media for expressing the design idea.
- Manage the complexity. Some students may recoil from the daunting complexity posed by the software itself. These types of students undoubtedly benefit from an incremental introduction in a pressure-free environment. A good introductory exercise is to model a simple existing building (say, a garage or house) so that learning the software is decoupled from design.
- Encourage invention. BIM programs currently have some limitations that may constrain students' creativity: the 'default' palette of materials or the capability to handle NURBS surfaces, for example. There are solutions, but they require students to be adventuresome. Instructors should foster an environment which encourages such exploration.

Despite the challenges and barriers, we expect the paradigm shift from CAD to BIM will trigger transformative changes for Architecture and Architectural Engineering programs in the immediate term. Indeed, the potential exists for one of the most disruptive (positive or negative) episodes in the history of architectural education. Faculty should be prepared to reexamine curricular priorities and discuss new teaching methods. Early adopters in academia should reduce uncertainty by sharing their experiences. Accrediting bodies will play an unpredictable role. Institutional anxieties will remain high.

The conclusions presented here are limited by experience and the relatively small body of literature describing others' experiences. Many of our suggestions are tentative and intended to promote discussion. As more Architecture and Architectural Engineering programs begin to introduce BIM, a robust discourse about BIM's challenges and opportunities should be nurtured, eventually leading to formal 'best practices' models.

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From CAD to BIM: The Engineering Student Perspective

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Abstract

This paper investigates the computer aided drafting (CAD) to building information modeling (BIM) transition from the student perspective in an architectural engineering (AE) degree program at the University of Wyoming (UW). A sophomore-level architectural engineering graphics course introduces CAD as the primary means of design communications. The following two courses are introductory and terminal architectural design studios where BIM first intervenes. Then the senior capstone course in systems design reverts back to CAD. Several problems occurred along the way such as the finer details of working with the BIM software to bigger pictures issues of being introduced to BIM at different points in the curriculum. I performed a narrative research study in the auto- and biographical traditions to recount and construct our BIM experiences. We recommend to UW faculty that CAD should be kept in the curriculum, and BIM should be introduced alongside CAD in the introductory graphics course.

Introduction

In the University of Wyoming (UW) College of Engineering, we have multiple degree programs that lead to the same licensure disciplines. If one wants to become a structural engineer, we either enroll in the civil engineering (CE) or the architectural engineering (AE) degree program. If one wants to become a mechanical engineer, we either enroll in the mechanical engineering (ME) or the AE degree program. The AE degree has structural and mechanical areas of emphasis and is accredited by ABET, Inc. The distinction between ME, CE, and AE is that AE has a specific focus on buildings which also includes a sequence of courses requiring graphic responses. Our AE degree program has four required courses consisting of graphics and design: Architectural Engineering Graphics (AEG), Architectural Design I (introductory studio), Architectural Design II (terminal studio), and a Systems Design Course (structural or mechanical option capstone). The architectural sequence highlighted in

Figure 1 totals thirteen credits of a total of 132 credits for the structural option. The method of communicating design ideas in this curricula sequence is through computer aided drafting (CAD) and building information modeling (BIM). Many of the students are already familiar with CAD since this was available in our secondary educations, but a vast majority of us initially heard of BIM through innuendo from the juniors and seniors. The National BIM Standard Project Committee defines BIM as “a digital representation of physical and functional characteristics of a facility. BIM is a shared knowledge resource for information about a facility forming a reliable basis for decisions during its life-cycle” [1]. The student definition is more of a powerful three-dimensional modeling system.

The course description for AEG read “Introduction to architectural drafting basics, including computer-aided drafting, architectural presentation drawings, freehand sketching, essentials of architectural design and building code compliance” [2]. CAD is the primary means of communications in the sophomore AEG class. ABET identifies that the purpose architectural design is for graduates to have “an understanding of architectural design . . . that will permit communication, and interaction, with the other design professionals in the execution of building projects” [3]. Since ABET does not specifically identify any graphics requirements, this is left as the instructor’s discretion. The architectural design course descriptions do not list any graphics techniques and simply require learning the fundamental principles of design and increasing the project complexity.

Since BIM is not formally identified in the course descriptions, this leads to several questions regarding *how BIM should be introduced into the curriculum?* We performed a narrative research study in the auto- and biographical traditions with the support of unobtrusive artifacts such as drawings, informal interviews, participant journaling and dialogues, and raw field notes collected from the students and faculty. The purpose is to recount and reconstruct our BIM experiences for the purpose of providing recommendations to our own AE degree program in addition to the common CE and ME programs.

	Freshman		Sophomore		Junior		Senior	
	Course	Credits	Course	Credits	Course	Credits	Course	Credits
Fall	Orientation to Engr Study	1	Architectural Engr Graphics	3	Architectural History	3	Structural Steel Design	3
	Intro to Engr Computing	3	Engineering Surveying	3	Structural Analysis I	3	Architectural Design II	3
	General Chemistry I	4	Dynamics	3	HVAC of Buildings	3	Structural Option Elective	3
	English Composition	3	Mechanics of Materials	3	Architectural Design I	3	Architectural Engineering Practice	3
	Calculus I	4	Calculus III	4	Fluid Dynamics	3	Basic Engineering Statistics	3
	Physical Activity Elective	1			Math / Science Elective	3		
Spring	Statics	3	Building Mat'ls & Const Mtds	3	Structural Analysis II	3	Structural Sys Design Project	4
	Calculus II	4	Plumbing & Electrical Systems	3	Structural Concrete Design	3	Structural Option Elective	3
	Engineering Physics I	4	Thermodynamics	3	Soil Mechanics	3	Structural Option Elective	3
	University Studies	3	Applied Differential Equations I	3	Electrical Circuit Analysis	3	Scientific & Technical Writing	3
	U.S. and State Constitutions	3	Physical Geology	4	Engr Econ & Prof Ethics	3	University Studies	3
					Civil Engineering Materials	3		

Figure 1. Architectural Engineering curriculum – structural option.

Findings

The findings represent our CAD and BIM experiences. Several themes emerged: the amount of CAD experience in secondary school varied from student to student; the variance in how firms at Career Day use graphics; and whether we first engaged BIM in the introductory or terminal studio.

Previous CAD Experiences

I completed technical drafting and computer aided drafting courses during my secondary education where I learned the concepts and skills of drafting in addition to an introduction to CAD. Approximately one-third of the AEG students did not have any graphic experiences in freehand sketching or CAD. Due to my previous drafting knowledge and my classmate's lack of experience, I felt ahead of the game in the university's introductory graphics course. This gave me the opportunity to build upon my skills and in turn improved the overall design of the assigned project which was a four unit apartment complex.

The entire class used CAD in the introductory studio. This allowed me to extend my CAD skills while designing a large aquatic center over 35,000 ft². Even though this was a very daunting task, I felt my CAD skills were adequate to handle the scope of the project. The terminal design studio was another story since the instructor required BIM for all of the projects. My previous knowledge of CAD initially proved to be of little use for this course. Some of the basic drafting tools in the BIM program (Autodesk Revit Architecture) were similar, yet the programs and their capabilities are very different. It was very difficult to be posed a semester's worth of work at the outset and not be able to begin due to unfamiliarity with the new design program. The learning curve in this class was very steep at the beginning. We were challenged with the project magnitude and complexity, as well as learning an entirely new computer software program. Since I did not have any previous BIM experience, I was no longer ahead of the curve which was somewhat discouraging. In fact, none of the other students had the experience either which made for a level playing field.

Introducing BIM in the Terminal Studio

To say that being presented a semester long design project and being required to use a new program is overwhelming, is saying the least. Our design challenge was a 5000 ft² addition to an existing building with specific client's needs. In addition, we considered various 'green' aspects in the design to meet minimum LEED standards. The scope and complexity was a pretty substantial project that we were presented when the semester began. It was frustrating to be restrained in the early stages of the design process by not knowing anything about the Revit software program. Having already completed the other graphics course and the introductory studio it seemed

crazy to flounder around with such elementary principles of the program such as drawing a wall or adding a door.

Due to the newness of Revit, our instructor had little knowledge of the program. This left us the only tool at our disposal, self-teaching. Having knowledge of CAD eventually proved beneficial as some of the basic drafting tools are the same. At the same time, it also felt discouraging because we could all easily draw our basic floor plans in CAD without a problem. The simplest tasks proved to be a challenge. How was it going to be possible for us to complete this project in the allotted time when we couldn't even outline a starting floor plan?

Once the initial anxiety wore off and some time was spent with the program, the power of this program became rapidly apparent. One floor plan in Revit is much more than a floor plan. It is a building elevation, a cross section, isometric and interior views. When a wall in plan view is shown in CAD, it is just one line offset from another. When a wall in plan view is shown in Revit, it has all the assembly components. The Revit user defines the wall as interior or exterior, and selects the composition of concrete masonry units, wood, etc. When a specific type of wall is selected, the user can see a view of all of the components and modify any or all of their properties. Not only does this expedite the design process, BIM can also be used as a learning tool.

Some students, such as myself, do not have any prior construction experience. This makes it hard to draw and visualize wall sections and other various details. What are the typical components of a five inch interior wall? This question is easily answered by clicking on the details of the wall in Revit. Not only is the user shown how a conventional wall is built, but also how each of the components is modeled in the design plans. The same concept can be applied to standard door and window sizes, electrical components, façade elements, etc. This capability really allows the student to get a better feel for how the building will be constructed. The default settings of the components can be altered; therefore designs are not limited by the program. This feature allows for explorations and creativity.

Utilizing such a powerful program really enhanced our designs. We were able to add components and features that would have been very tedious to implement in CAD. Being able to position furniture and other components allowed us to portray room usages in a clear fashion. Our design was also enhanced by the quality of images we were able to produce. Having rendered interior and exterior views of our project allowed us to clearly illustrate our project in an oral presentation. It was incredible to create such a realistic design, and this was only possible with Revit.

Using each of the different programs, CAD and Revit, in the design studios gave me a unique perspective. I was able to see how different the two design projects were in their respective finished stage (see Figure 2). In the introductory studio, it took the majority of the time to produce the required drawings. Our team had a difficult time completing the necessary components of the design; floor plans, elevations, wall

sections, etc. Also, if a part of our design needed to be changed, we had to alter the other drawings that coincided with this change. In the terminal studio, I was surprised to find how much more time we had once we overcame the initial struggle of learning the new computer program. The extra time was spent refining our final design as well as researching information about LEED requirements and how we could integrate them into our design. Using Revit allowed us to evaluate exactly how the building would fit into the proposed site. We were able to create realistic snapshots of our project with internal and external rendered views. In addition, we could focus on details such as the amount of natural daylight allowed in the building. Having this extra time allowed us to consider other aspects of the design that were not possible in the previous studio.

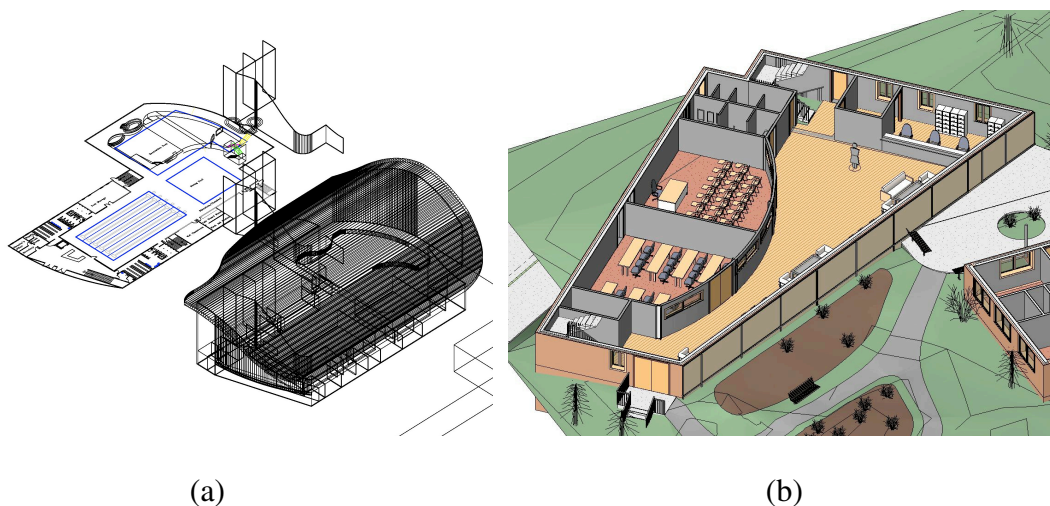


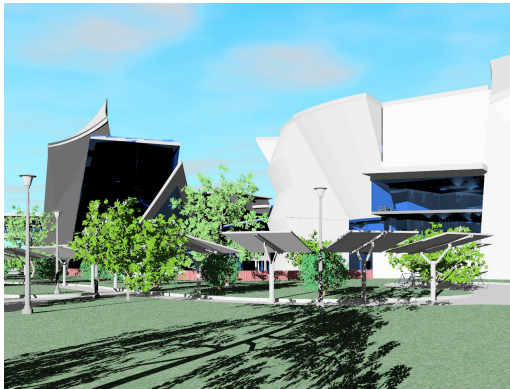
Figure 2. CAD to BIM transition: (a) introductory studio; and (b) terminal studio.

Introducing BIM in the Introductory Studio

After I completed the introductory studio, the instructor has since introduced BIM. He has also varied the sizes and complexities of the projects to gauge what the students can handle. We now have students that were introduced to BIM at different points in their education. This provides a perspective on when BIM should be instituted into the curriculum.

When I consulted students who were introduced to BIM in the introductory studio, I found that their experiences were similar to mine, just at an earlier stage. They endured the same struggles of learning a new software program with minimal instruction. After the initial challenges of familiarizing themselves with Revit, they all concluded that BIM is an excellent tool for architectural design. One group used the mass modeling feature of Revit and their final design was based off of this (see Figure 3(a)). Their design was so complex that they would not have been able to have the same finished project if they had used CAD.

These students agreed that already having used BIM made the terminal studio a pleasant experience. They could concentrate on the design challenge even with fewer members on their design teams. Time was available for them to focus on more aspects of the design besides just the minimal components. Fire codes and LEED elements were among the extras they were able to incorporate into their designs. The design in Figure 3(b) used local materials in an effort to minimize the consumption of nonrenewable energy resources by reducing the transportation distances. The image is also not a rendering as in Figure 3(a). This is a three-dimensional isometric drawing that can be captured instantaneously at any point in the design process. Even though these students had more experience with BIM, they all agreed that CAD should still be taught and used at some point as it is still widely used in the industry.



(a)



(b)

Figure 3. BIM advancements: (a) introductory studio; and (b) terminal studio.

Experience with Industry

My industry experience is very limited, but I have interviewed with a few employers and had an opportunity to discuss BIM. I found that some employers do not have their engineers do any graphics at all, while others use CAD, but are beginning to deploy BIM.

Being an architectural engineer creates some interesting situations. Students should be taught both the architecture and engineering fundamentals; however their careers may not require such a dual expanse of knowledge. One firm that I interviewed specifically recruits UW graduates because the AE program produces students who fit well into their company. They are a civil and structural engineering company where their Engineers-In-Training and Professional Engineers do not perform any drafting. The Engineers carry out the calculations and then approve the plans from the drafting department. Applying to work for a firm like this requires background

knowledge of both the architecture and engineering fields, but nullifies the CAD and BIM advantage over other candidates competing for the same job.

A different company who recruits UW students first learned of BIM when they noticed that Revit software was listed on a student's resume. This structural engineering firm has since started to use Revit in some of their designs and projects. The extent to which the firm uses Revit is unknown, yet the fact that it is incorporated into a structural engineering firm is worth noting. I have stopped interviewing firms and have entered the graduate program at UW. When I restart my job search I hope to find a job that allows me to use my graphical knowledge.

Conclusions

By learning BIM are we better off? There are many challenges to overcome when trying to incorporate a new design program into a studio type course. The obstacles that the students face are difficult, but are very worthwhile in the end. In hindsight I would never say that I would have preferred to use CAD for my design project. The benefits of learning Revit by far outweigh the beginning struggles. In today's society students in the engineering field are very computer literate. Learning a new program may not be a walk in the park, but is a very probable task to undertake. A course dedicated solely to teaching a BIM program may be overkill and the structured curriculum does not have excess room.

How and when should a BIM program be introduced? I believe that the earlier it is introduced the more students will be able to utilize its capabilities and better their designs. I do believe that CAD should still be a part of the curriculum as it is widely used in the industry. Integrating the two programs in introductory graphics courses would help students to be prepared for whichever course their career or the industry takes. Since the AE program teaches both the architectural and engineering aspects, the students should be familiar with both software packages.

Acknowledgements

I would like to thank the following students for providing their experiences of BIM being introduced into the introductory studio or providing the graphic images: Christina Abel, Daniel Charbonneau, Jeremy Chuhralya, David Clark, Sam Finkle, Westin Joy, Cristen Leavitt, and Jill Nichols.

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From CAD to BIM: The Experience of Architectural Education with Building Information Modeling

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Abstract

In the spring of 2006, Montana State University began to include a Building Information Modeling (BIM) program in its digital graphics and design curriculum. Since then, an increasing number of students have had the experience of designing with BIM programs and comparing their processes and final products with other students who are using traditional Computer Aided Design (CAD) programs. While 2-dimensional CAD and 3-dimensional BIM models may simply be tools to accomplish the same end, they provide two radically different approaches to learning architectural design.

This paper discusses the implications of BIM with respect to the student experience. Within this topic, the paper describes the classes in which students are using BIM software and exactly how the experience of designing with BIM is different than the experience of designing in two dimensions. The paper also outlines the ways in which we can expect BIM to alter architectural education in the near term as well as in the future. Lastly, the paper addresses the concern that many have that Building Information Modeling hinders creativity. The paper concludes with some recommendations as to how BIM can be used to the greatest advantage in the classroom.

Opportunities in the Curriculum

The architectural curriculum at Montana State University currently offers students the opportunity to learn Building Information Modeling programs in two separate classes. The first is a digital graphics course for sophomore-level students. In this class, a variety of software programs are introduced in the hopes that students will get a sense for which of these tools they would like to become more familiar with and use during the course of their design education. Drawing programs, such as AutoCAD, free-form 3-d modeling programs such as 3D Rhinoceros and BIM programs like ArchiCAD and Revit Building are introduced along with rendering programs and several components of the adobe creative suite such as Photoshop and InDesign. The second course, offered in the final year of the undergraduate program, is a required construction documents class emphasizing BIM modeling programs and providing students with another opportunity to learn and use BIM programs. This course involves the creation of a small set of construction documents and also covers cost estimating and specification writing. In this course, the time saved by using a BIM is significant and therefore it is at this time that many students genuinely learn to use a BIM program. In the design studio however, students at Montana State use a broad array of rendering and drawing methods. Some create 2-dimensional and 3-dimensional presentation materials by hand. Others hand-draw or use AutoCAD for 2-dimensional drawings and render free-form digital models for perspective views of the building. A growing number use BIM modeling tools in the studio. In the final years of the program these tools are especially useful in the design studio; by this time, we are required to integrate electrical, mechanical and plumbing

components into our studio projects. Building Information Modeling can facilitate this by allowing students to create a model of the building using these system components.

Changing use of BIM over Time

Some of the student opportunities associated with BIM software are relatively easy to take advantage of in the near term. For example, BIM allows students to conduct an energy analysis on a digital model that originated with a studio design. In the mechanical systems class at Montana State, students have already been taught to perform an energy analysis by importing a BIM into an energy-analysis program called AGI32. This was not difficult by any means, although the model had to be converted into a CAD format before it was imported into the program. This investigation was particularly useful because we could quantify how modifications to the model changed the average amount of energy that the building might use. A similar opportunity exists with the scheduling functions of many BIM programs. For example, we could easily use the detailed material quantity information generated from a digital model to find out how much it might actually cost to construct something we've designed. Opportunities now exist to subject a BIM model to other analyses like thermal simulation, daylighting and electric lighting studies as well as studies of airflow movement and occupant evacuation. Integrated Environmental Solutions is one software company that has developed these programs and in their case, the platform can be directly linked to Revit MEP. Opportunities for the analysis of a BIM already exist and are easier for students and professors to take advantage of in the near term, as they present fewer issues of coordination. I believe that as our professors become more familiar with BIM software and its capabilities, model-based analyses can be incorporated at the very least into our professional practice course.

While some of the benefits of designing with a BIM are now available to students, other opportunities will require more coordination than can be implemented in the course of a year or two. Many of the changes in education that we could see in the near term as a result of BIM modeling involve the integration of materials and systems from multiple courses into a single digital model or project. This seems to present some interesting challenges. For example, if we were to design a mechanical system for use in a project that is being designed simultaneously in a studio course, it would require that professors coordinate both their schedules and their educational agendas. Although this is possible and even likely in the long-term, it is difficult to implement this integration without a certain degree of curriculum-wide coordination.

Building Information Modeling creates the opportunity for students to introduce the element of time into their designs. This occurs through the phasing and planning options which are included in most BIM software. These features afford students the opportunity to understand the construction sequence and to classify different elements of the building as being part of a certain phase. This process would be most applicable in our construction documents course. It is ultimately the aim of that course to create a set of documents which describe the phases and details of construction; this BIM feature is intended to facilitate communication along these lines and to formalize issues of sequence and timing.

Yet another change that BIM will eventually lead us to involves the way in which we informally present design work to our professors. Currently, professors are more comfortable critiquing physical models and two dimensional drawings such as plans, sections and elevations than a 3-dimensional digital model. While this is understandable given the tools that existed at the time of their training, it can be problematic. For example, many students wish to give their professors a virtual tour of what they are creating by “walking” them through the structure as it can be seen on-screen. Professors seem to be unsure as to how to evaluate the design and they often respond by asking the student to print out a few perspective views of the building because they are able to evaluate these static images more easily. This is confusing to the student who knows that buildings are experienced by moving people and that a virtual tour would best mimic the human experience in their project. Professors are accustomed to demonstrating the aesthetic principles of architecture in two dimensions. Over time, instructors will be better able to translate the principles they were trained with into the 3-dimensional milieu. However, this represents a significant change in the pedagogy and will therefore take time to implement. All of the changes described above, the coordination of teaching agendas, the inclusion of phasing in classroom instruction and the challenge of critiquing in three dimensions will take more than a year or two to implement. However, it is critical that we do in fact implement these changes; they present an opportunity to take full advantage of the educational opportunities and efficiencies that BIM affords us.

Issues of Creativity

As was mentioned above, students at Montana State use a broad array of rendering and drawing methods in the design studio. The diversity of methods used indicates not only that different students are most comfortable with different methods of designing, but that students are having a variety of experiences in the same classroom environment. Generally speaking, students using AutoCAD are drawing lines of varying thicknesses, while students using BIM programs are adding components and integrating them with one another. These are two entirely different ways of creating architecture and in this sense, BIM changes the way people work by altering the very process we use to create representational drawings. It is highly significant from a pedagogical standpoint that while instructors might be giving the same assignments to all, the methods that students use to complete those assignments vary widely. In this sense, architectural education has changed dramatically. While the mechanics of drawing used to be taught as part of the curriculum, now design instructors can only advise students on how their drawings should appear. Methods of creating documentation are not emphasized as they used to be because such a variety of means – old and new – are in use in the design studio.

There is significant concern, among faculty members and students alike, that Building Information Modeling hinders creativity in the design studio by providing convenience in material and assembly choices. This is understandable considering that creating a new object or altering an existing one in a BIM requires a set of skills slightly beyond those needed to simply operate the program using the components that are immediately available. For example, creating a new object in Revit is a process unto itself: it requires that one know how to use the programs’ reference planes to create shapes from scratch and then classify these objects appropriately. Although this is not very difficult, it is

certainly more difficult than simply using the components that come with the program. This is not the case with 2-dimensional drawing programs. Creating new objects or drawing entirely new design segments in those programs requires *relatively* fewer skills. In AutoCAD for example, to represent a new detail, one would simply draw the connections using the same or similar processes with which they had created the rest of the drawing.

In the context of this discussion, it is important to note that when students were exclusively using AutoCAD or hand drawing, the repetitive use of standard details, wall assemblies and specific design features was common. Segments of drawings have always been used and reused in the architectural studio. Drawing new details, wall assemblies or design features has always been more difficult than using a replicated standard. Even so, drawing components anew in a 2-dimensional program does require relatively fewer skills than it does in a BIM program. This problem is compounded by the fact that there are many agendas to respond to in the architectural studio. Students are responsible for finding precedents for their concept and design, creating a theoretical framework in which their design can be understood, documenting all aspects of the building site (including its history and its future), creating a plan for landscaping around their project, designing a (preferably) unique structural system, designing every aspect of the structure itself while at the same time using environmentally sustainable technologies and materials, making photo-realistic renderings or drawings of the building on the landscape, and finally arranging all of these elements in a graphically sophisticated presentation. Given all of these tasks, architecture students are often highly interested in ways in which they can save time. Ready-made components often provide students the opportunity to spend less time on a piece of the project while perhaps achieving a similar result. A partial solution to this hindrance of creativity is the training of students – perhaps in an introductory course such as the sophomore level graphics course described above – not only to create new BIM components outside of those provided, but to emphasize the importance of their use in the design studio.

BIM software also has the potential to hinder creativity because in many cases, each of its components has to be shaped or structured in a certain way in order to maintain their properties within the program. For example, in Revit Building, if a wall curves in two directions simultaneously, it cannot retain the parameters or properties that identify it as a wall. This problem, when it infrequently occurs, may indeed hinder creativity. However, in my own experience, students will not simply change their design, (thereby allowing the software to control their project) but will instead work beyond the confines of the program. In the case described above, this might be done by creating the wall object in a free-form digital modeling program, importing it into the BIM program and then assigning wall properties to it.

BIM may marginally hinder creativity for the reasons described above. However, this does not indicate that BIM should be taken out of the classroom. Rather, we need to be aware of its weaknesses as a creative tool and our use of it should be altered accordingly. This has already happened at some architecture firms; I was told recently that a 100-person Seattle firm has banned their BIM program from the conceptual design phase in

an effort to combat the program's effect on creativity. For similar reasons, it may benefit professors to be aware of the use of BIM in the classroom. Although BIM users may be able to create a digital model more quickly than others, they can perhaps use the time that they have saved to create their own components or give their models a higher level of detail.

Integrated Practice as Studio Model

It has become clear that BIM will eventually change the classroom experience as much as it has changed the mechanics of the professional design process. This is quite inevitable; the educational environment must maintain some likeness to the real-world practice of architecture. However, some student benefits of designing with a BIM may not be realized for quite some time. For example, much has been made recently of the efficiencies offered by Integrated Practice. Integrated practice is a project delivery approach that many believe could increase overall efficiency through tighter collaboration between all participants in the building process. At a minimum, this approach includes collaboration between the owner, the architect, and the general contractor from early design through project handover. BIM fits perfectly into this project delivery concept because it facilitates this collaboration: each party works from and adds to the same model. There is a great opportunity in the classroom to mirror the growing degree of collaboration that BIM affords us. Architecture, Engineering and Construction Management students might all benefit from working as a team in an educational environment and using the technology that allows them to all work on the same project. This endeavor would require students from these departments to meet with an interest in collaborating in a design-build process. All of their strengths and skills could be used in every phase of the design, refinement and construction of a small-scale project. BIM has paved the way for us to actually move Integrated Practice into the classroom. This sort of change would present university-wide coordination issues and would therefore not be realized for quite some time, if it was indeed ever realized.

Conclusions

Bringing Building Information Modeling into the classroom genuinely helps students to become better architects. We have a greater opportunity to visualize a building as a series of systems, to visualize the construction sequence, to identify co-location problems and to work with other students in related fields. Building Information Modeling has changed the practice of building design and construction quite dramatically in a short period of time. Schools of Architecture have the responsibility to follow suit. As the situation stands however, many of the educational opportunities described above are lost when we don't use BIM to its full potential. The transition from one design process, drawing lines, to another, adding components, represents a fundamental change in how we design. If this is the way that professional work is done, schools of architecture have to change their pedagogy accordingly. This implies greater coordination on several levels: coordination between professors, coordination within the architecture curriculum and university-wide coordination as well.

EXPERIMENTAL VERIFICATION OF THE STATIC METHOD FOR SEISMIC DESIGN OF VIBRATION-ISOLATED HVAC EQUIPMENT

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ABSTRACT

Motivated by the repeated damage to vibration-isolated equipment in almost all strong earthquakes that occurred in the past four decades, an experimental study was conducted to verify the adequacy of the current static method used for the seismic design of this type of nonstructural components. The study included earthquake-simulator tests of two HVAC equipment items supported by restrained vibration-isolators. The test results confirmed that seismic protection of vibration-isolated equipment by snubbers comes at the expense of amplified acceleration responses and dynamic loads, which are often much larger than those predicted by the static method. The differences between the predictions of the static method and actual dynamic responses of the equipment were attributed to four aspects not considered in the static method: Rotational responses of the equipment, nonlinear relationship between the gap size and seismic demands, effect of thickness and hardness of the snubbers, and effect of flexibility of the equipment.

INTRODUCTION

Rigidly mounted Heat Ventilation and Air-Conditioning (HVAC) equipment can be the source of noise and mechanical vibration, which are annoying to the occupants and disturbing to sensitive equipment inside a building. To reduce the transferred noise and vibration, HVAC equipment items are often mounted on vibration isolators such as coil springs. The efficiency of vibration isolation is inversely proportional to the stiffness and damping property of the isolators (Petrušewicz and Longmore 1974). Therefore, vibration-isolated equipment items are technically lumped masses supported by flexible links without damping. On a building roof where HVAC equipment items are typically installed, this condition could lead to large seismic displacement responses. Large seismic displacement is problematic because typical vibration isolators have limited displacement capacity. Failure of the isolators, which support the equipment, might result in damage to the equipment or breakage of the lifelines (e.g. pipes and ducts) connected to the equipment. Therefore, it can be said that desirable vibration-isolation efficiency is achieved at the expense of high seismic vulnerability.

The application of elastomeric snubbers has been the predominant method in seismic protection of vibration-isolated equipment for more than 40 years. Snubbers are essentially bumpers, installed with a practical air gap from the equipment to limit the displacements of the equipment. The air gap is necessary to keep the snubbers out of contact during the normal operation of the equipment, and to maintain the vibration-isolation efficiency. When the displacement response of the equipment exceeds the gap size, an impact occurs between the equipment and the snubber, and the equipment bounces back to move within the accepted range of displacements.

The snubber contact-surface is often made from a flexible material such as neoprene to prevent the potential destructive impact between two hard surfaces. Snubbers can be installed separate from the vibration isolators or they can be integrated into vibration isolators and form so-called Isolation/Restraint (I/R) systems. Snubbers are classified also based on the number of directions along which they can restrain the motion; for instance, unidirectional or all-directional snubbers.

Compared to the equipment items freestanding on vibration isolators, the equipment protected by snubbers have performed much better during the earthquakes that occurred in past four decades. However, in several cases, the dynamic forces induced into snubbers were large enough to break or pull off the snubbers from their connections. Consequently, the equipment was damaged or dislodged, or lifelines connected to equipment were broken (Filiatrault et al. 2002; Gates and McGavin 1998). Figure 1 displays an example of such damages during the 1994 Northridge earthquake. The chiller shown in the photograph was dislodged after its I/R system supports failed. The excessive displacement of the chiller resulted in the rupture of the connected pipe.



Figure 1. Pipe rupture resulted from failure of vibration-isolator supports of a chiller, 1994 Northridge Earthquake (Photo Credits: Lloyd 2003).

A comprehensive experimental study was conducted at the University at Buffalo to verify the accuracy of the current seismic design procedure in predicting the actual seismic responses of vibration-isolated equipment. Earthquake-simulator tests with triaxial input motions were conducted on two HVAC equipment items, supported by I/R systems. This paper provides a brief introduction to the current seismic design procedure for vibration-isolated HVAC equipment, and discusses relevant details of the experiments, selected test results, and the conclusions drawn from the test results. More details about this experimental study can be found on two reports published by the authors (Fathali and Filiatrault 2007a, b).

CURRENT SEISMIC DESIGN PROCEDURE

The current static method for seismic design of vibration-isolated HVAC equipment simplifies the earthquake-induced forces to a pair of horizontal and vertical equivalent static forces applied to the approximate Center of Mass (CM) of the equipment. The support reactions due to the forces at the CM are calculated based on the equipment mass distribution. The static method inherently assumes that the equipment does not move during the earthquake, and thereby, the relative accelerations between its CM and the ground generate forces that must be balanced by reactions at the supports. The distribution of the equivalent static forces based on the equipment mass distribution, known as the lump mass method, is one of the many available methods to calculate the support reactions (Miesel 2001; Tauby et al. 1999).

According to Section 13.3 of ASCE 7-05 (2006), the horizontal static force is calculated as $F_p = \{0.4S_{DS}a_p(1 + 2z/h)I_p/R_p\}W_p$, where, S_{DS} , a_p , z , h , I_p , R_p , and W_p are respectively defined as: Design spectral response acceleration at short period for 5% damping ratio, component amplification factor, height of attachment of the equipment to the structure with respect to the base, average roof height of the structure with respect to the base, component importance factor, component response modification factor, and component operating weight. The horizontal static force (F_p) is not required to be greater than $1.6S_{DS}I_pW_p$, and should not be taken less than $0.3S_{DS}I_pW_p$.

The horizontal static force should be applied independently in at least two orthogonal horizontal directions in combination with service loads associated with the component, as appropriate. For vertically cantilevered systems, however, the horizontal static force shall be assumed to act in any horizontal direction. In addition, the component should be designed for a concurrent vertical force equal to $\pm 0.2S_{DS}W_p$.

The component amplification factor (a_p) is included in the computation to address flexibility issues. Per Table 13.6-1 of ASCE 7-05, a_p is 2.5 for all types of vibration-isolated equipment. The response modification factor (R_p) for equipment mounted on neoprene isolators and springs is 2.5 and 2.0, respectively.

Chapter 54 of the ASHRAE handbook (2003) defines minimum thickness requirements for the elastomeric contact surface of different types of snubbers. However, the only property of the snubbers included in the calculation of the equivalent static force is their gap size. In the current static method, it is assumed that the relationship between the snubber gap size and the seismic-force demand of the equipment can be simplified by a step function. ASCE 7-05 requires the design force be taken as $2F_p$ if the nominal snubber gap size is greater than 6 mm (0.25 in).

EXPERIMENTAL STUDY

Test Specimens

A heavy and rugged centrifugal chiller and a light and flexible Air-Handling Unit (AHU), shown in Figure 2, were the two test specimens considered in this study. The weight of the chiller when filled with water and refrigerants was about 117.7 kN. The operational weight of the AHU was about 19.3 kN. The chiller and AHU had four and six support locations, respectively. The horizontal eccentricities between the approximate CM and center of supports for the chiller and AHU were about 25 by 75 mm and 230 by 150 mm, respectively. The vertical distance between the approximate CM and support level was about 0.96 m for the chiller and 0.76 m for the AHU.

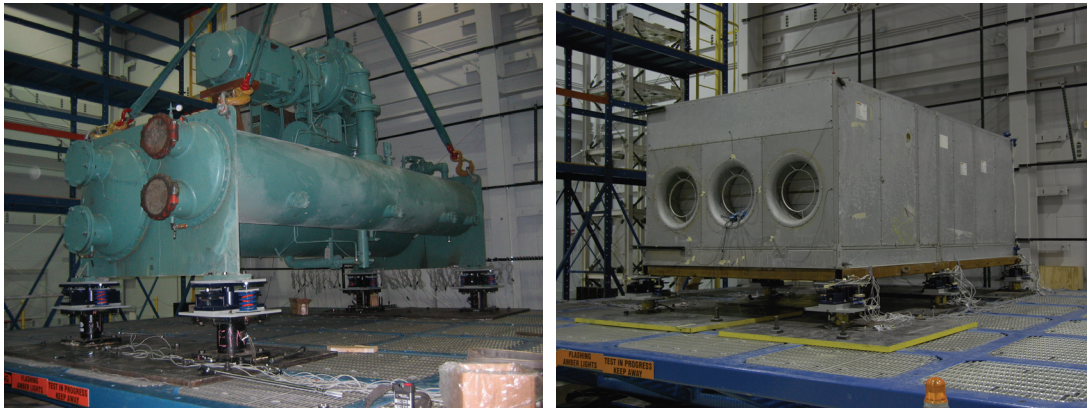


Figure 2. Test specimens mounted on I/R systems, light: Centrifugal liquid chiller, right: Air-Handling Unit (AHU).

Isolation/Restraint Systems

The I/R system used in this study consisted of an isolation and a restraint component oriented orthogonally with respect to each other. The isolation component consisted of only two nested coil springs embedded between two steel plates. The restraint component consisted of a top and a bottom part. The top part was bolted to the equipment and the bottom part to the base plate. The rubber snubbers were compartments of the bottom part. A rubber tube was fitted into the pipe on to the center, and two rubber grommets on the sides. The top part had a piece of pipe on the center and two threaded steel rods on the sides. When the threaded rods passed through the centers of the grommets, the top and bottom pipe were aligned coaxially with a cylindrical air gap left between the top pipe and the rubber tube. On each side of the grommets, the rods had a nut and a thick steel washer, which interfaced the nut and the grommet. Figure 3 shows one of the I/R systems supporting the chiller.

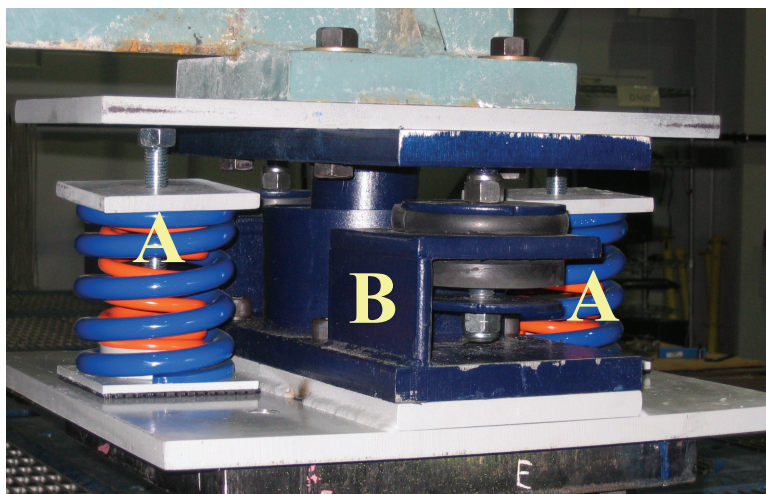


Figure 3. Assembled Isolation/Restraint (I/R) system installed under test specimen (chiller), A: isolation component, B: restraint component.

The restraint components of the I/R systems were designed with the assumption that the forces induced into the snubbers will be large enough to produce 3.0 g acceleration to the mass supported by the I/R system. The corresponding load was 88 and 15 kN for the I/R systems supporting the chiller and AHU, respectively.

Test Plan

The experimental study included several triaxial seismic and system-identification tests on the six degree-of-freedom earthquake-simulator at the University at Buffalo. Triaxial input motions for the seismic tests were generated to represent the roof motions of buildings located in areas with different levels of seismicity. As shown in Figure 4, the required horizontal and vertical roof response spectrum to generate the input motion were similar to those defined by the AC156 testing protocol (ICC-ES 2004). Different sets of triaxial input motions corresponding to different values of S_{DS} were used for the earthquake-simulator tests. Each seismic test was preceded and followed by a pulse-type system-identification test.

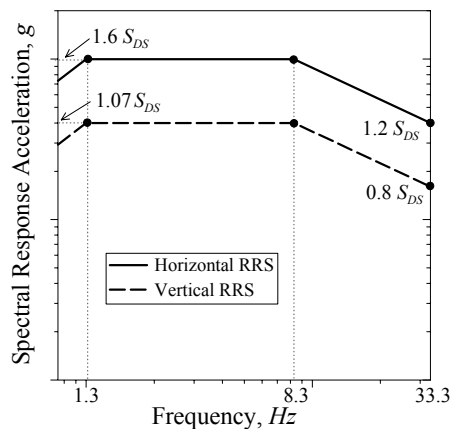


Figure 4. Horizontal and vertical Required Response Spectrum (RRS) used to generate triaxial roof motion for earthquake-simulator tests.

As listed in Table 1, the test plan included variation of snubber properties namely, the gap size, rubber snubber thickness and hardness. The horizontal gap was adjusted by inserting steel sleeves around the top pipe, and the vertical gap was adjusted by changing the location of the nuts along the steel rods.

Table 1. Variations of Snubber Properties throughout experiments.

Test Specimen	Test Series	Gap Size, mm	Horizontal Snubber, Rubber Tube		Vertical Snubber, Rubber Grommet	
			Thickness, mm	Hardness, Duro.	Thickness, mm	Hardness, Duro.
Chiller	C-1	13	6	60	19	60
	C-2	6	13	60	19	60
	C-3	6	3	70	19	60
	C-4	6	6	50	19	60
	C-5	6	6	60	19	60
	C-6	3	6	60	19	60
AHU	A-1	6	19	40	19	40
	A-2	6	19	60	19	60
	A-3	6	13	40	13	40
	A-4	6	13	60	13	60
	A-5	6	6	40	6	40
	A-6	6	6	60	6	60

SELECTED TEST RESULTS

Peak Shear and Uplift Forces

Variations of the peak shear and uplift forces induced into the supports with the S_{DS} value corresponding to the input motion during the tests conducted with the chiller and AHU are presented in Figures 5 and 6, respectively. For both test specimens, the largest mass supported by an I/R system was almost 25% of the total mass. Therefore, in Figures 5 and 6, the peak shear forces induced into the supports are compared to the 25% of the equivalent static force at the CM, which is calculated based on ASCE7-05 requirements for the rooftop equipment with high importance ($I_p=1.5$ and $R_p=2$). The test results show that the dynamic loads induced into the supports are very sensitive not only to the gap size but also to the thickness, and hardness of the elastomeric snubbers. Variation of the snubber properties in some cases could result in almost doubled seismic forces.

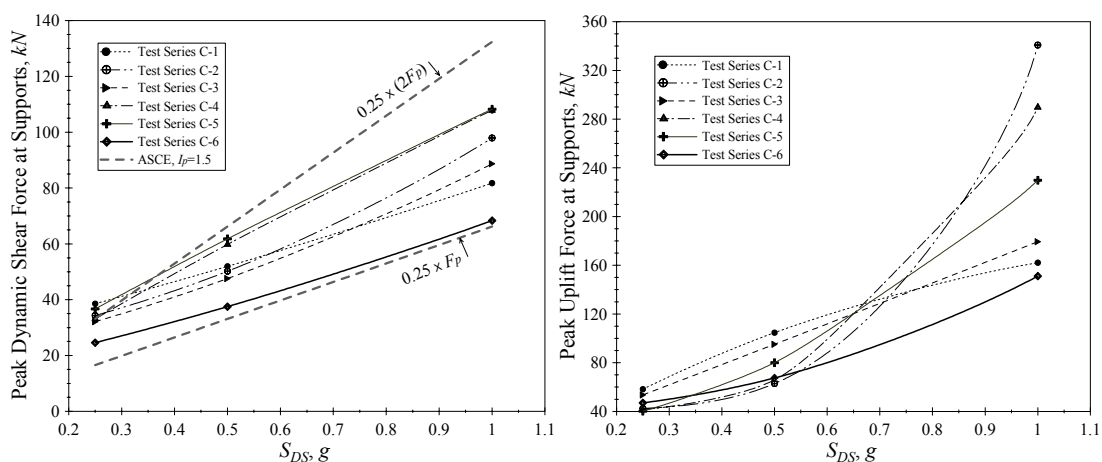


Figure 5. Variations of peak shear and uplift loads induced into chiller supports with S_{DS} .

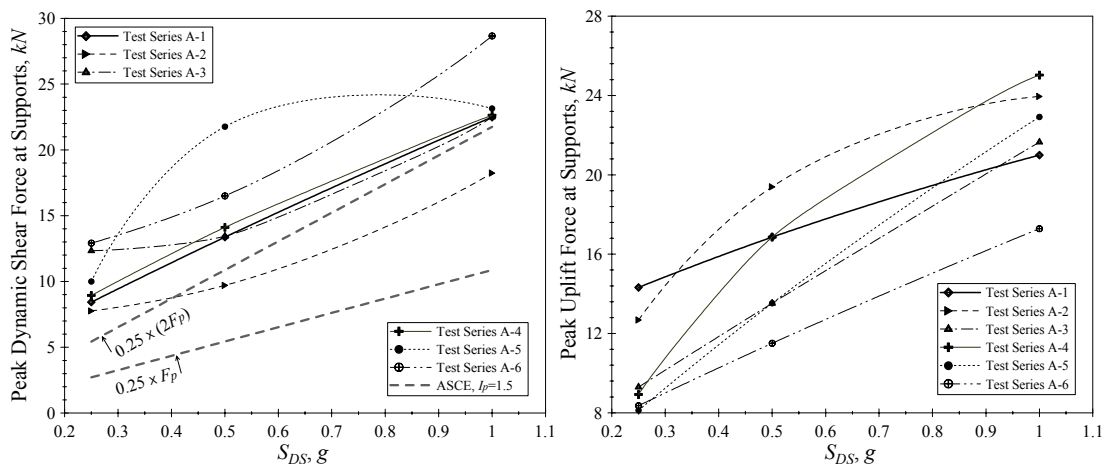


Figure 6. Variations of peak shear and uplift loads induced into AHU supports with S_{DS} .

According to the ASCE7-05 requirements, doubled dynamic loads ($2F_p$) were expected only during one test series (Test Series C-1), which was conducted with the gap size of the I/R systems larger than 6 mm (0.25 in.). However, the results show that in all cases the shear force at the supports of both test specimens have been larger than 25% of the equivalent static force at the CM. For the chiller, the doubled design

force ($2F_p$) seems to be a reasonable upper bound. However, for the AHU even the doubled design force is often less than the dynamic forces induced into the supports.

Acceleration Amplification Factors

The ratio between the peak acceleration response of the test specimen to the peak input acceleration during each test was defined as the Acceleration Amplification Factor (AAF). AAF is comparable to a_p (the component amplification factor), which is supposed to account for the flexibility of the equipment and/or its supports. AAF could be calculated for the response at any given point and in any given direction. Variations of the horizontal and vertical AAF at the CM of the chiller and AHU during the seismic tests are presented in Figures 7 and 8, respectively.

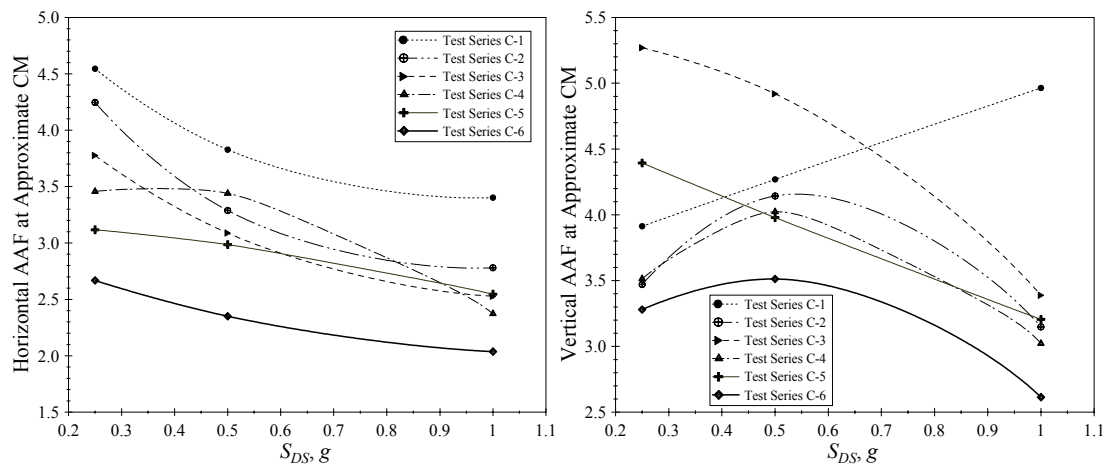


Figure 7. Variations of horizontal and vertical AAF at approximate CM of Chiller with S_{DS} .

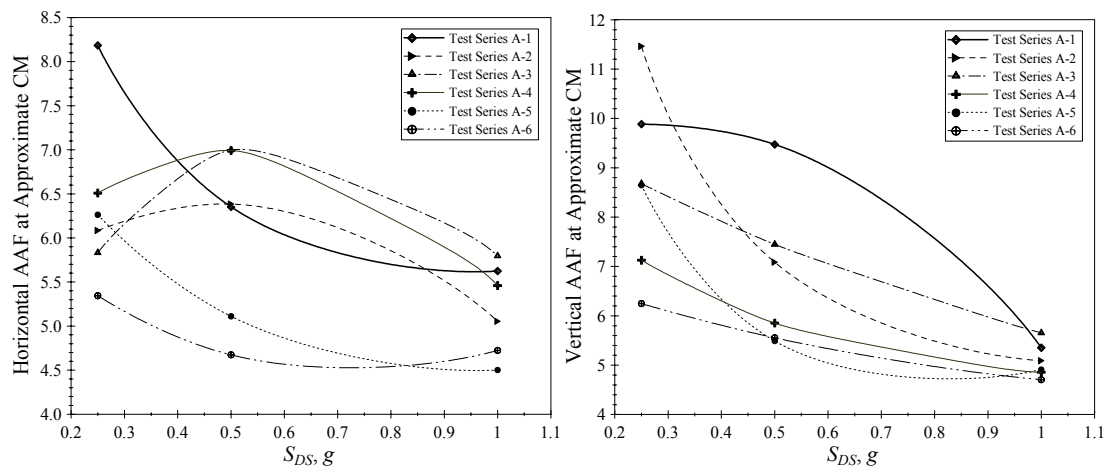


Figure 8. Variations of horizontal and vertical AAF at approximate CM of AHU with S_{DS} .

The results show that AAF is often much larger than the component amplification factor suggested by code ($a_p = 2.5$). Moreover, it is demonstrated that amplification of acceleration response depends both on the properties of the elastomeric snubbers and also on the flexibility of the equipment itself. This is an important observation that reveals one of the deficiencies of the current seismic design procedure to assign the same component amplification factor for all vibration-isolated equipment. From the standpoint of the current static method based on ASCE7-05 (2006), a vibration-isolated rugged chiller and a vibration-isolated flexible AHU should be designed for

the same seismic demand as long as they have equal mass and identical support conditions. The results of the experiments do not validate this assumption.

Peak Relative Displacement

Accurate assessment of the peak relative displacement response of the equipment is of great significance in designing the ducts and pipes connected to the equipment, which should accommodate the relative displacement experienced by the equipment. Variations of the peak relative displacement responses with S_{DS} during the tests conducted with chiller and AHU are presented in Figures 9 and 10, respectively. If the test specimens experienced pure translation, and the snubbers were uncompressible, the peak relative displacement response would be equal to the gap size. However, in the seismic tests and particularly in those with high-amplitude input motions and large gap sizes, the test specimens experienced a combination of translation and rotation. In addition, the rubber snubbers were compressed by the severe impacts occurring in the restraint components of the I/R systems. These two factors contributed in increasing the relative displacement responses of the test specimens sometime larger than ten times the gap size.

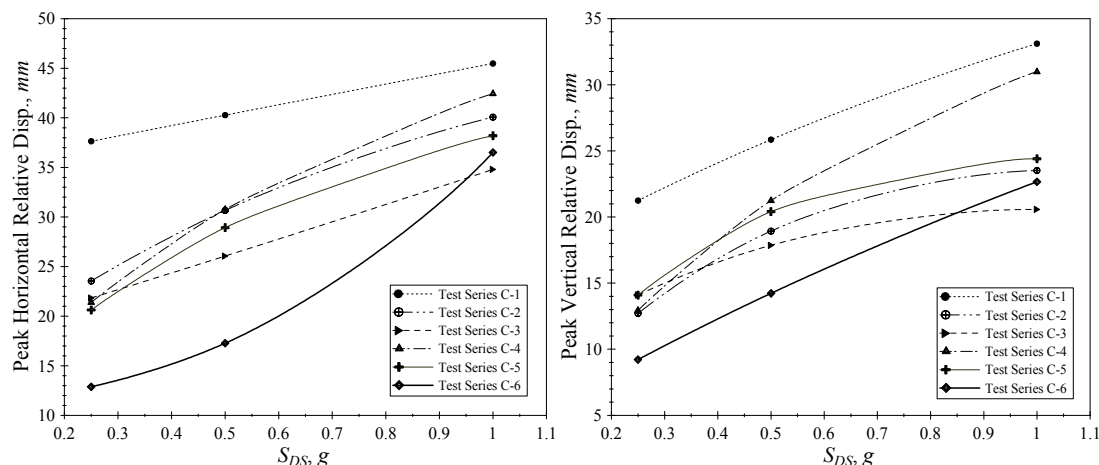


Figure 9. Variations of peak horizontal and vertical relative displacement responses of chiller with S_{DS} .

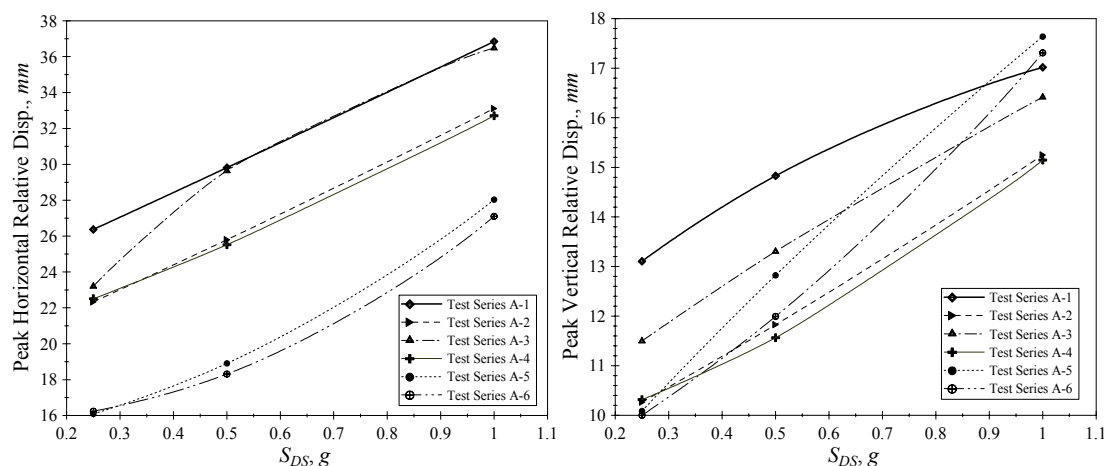


Figure 10. Variation of peak horizontal and vertical relative displacement responses of AHU with S_{DS} .

The displacement responses of the test specimens show that the rotational responses of vibration-isolated equipment are too significant to be ignored. With large rotational responses, the equipment engages the snubbers with both translational and rotational momentum, and therefore, the resultant forces will depend on both the mass and mass moment of inertia of the equipment.

CONCLUSIONS

The results of the earthquake-simulator tests on two HVAC equipment items mounted on isolation/restraint systems confirmed that the seismic protection of vibration-isolated equipment by rubber snubbers comes at the expense of amplified acceleration responses and large dynamic forces at support locations, which are often much larger than those predicted by the static method. The test results demonstrated that the differences between the predictions of the static method and actual dynamic responses of the equipment are attributed to the four main factors described below.

First, the rotational responses of the equipment, which are not considered in the static method, have large contributions to the total responses of the equipment. The equipment experiencing a combination of translational and rotational responses engages the snubbers by both translational and angular momentums. In addition, in presence of large rotational responses, the acceleration and displacement responses at the center of mass of the equipment can be significantly different from those at other locations of the equipment. These differences are not captured by the static method.

Second, the nonlinear relationship between the gap size of the snubbers and seismic responses of the equipment cannot be simplified by a step function per the current static design method according to ASCE 7-05. According to the current seismic design procedure, as long as the design forces are doubled, the air gaps can be adjusted to any size that would satisfy the vibration-isolation requirements. However, the test results showed that the seismic responses of the equipment keep on growing as the air gap increases. Large air gap would allow the equipment to accelerate within a large domain and engage the snubbers with high momentum. The test results suggest that the inclusion of an upper bound for the gap size in the design requirements of the vibration-isolated equipment is necessary.

Third, it was observed that other than the gap size, thickness and hardness of the elastomeric snubbers have significant influences on the seismic responses of the equipment, which are not considered in the static method. The compressibility of the elastomeric snubbers allows the equipment to move and accelerate within a displacement domain larger than that defined by the air gaps. Any requirement related to the gap size should account for the instantaneous increase of the gap size due to the compression of the snubbers.

Fourth, the test results showed that the amplification of acceleration response to a given input motion increases if the equipment itself is flexible. This suggests that the component amplification factor, included in the current seismic design procedure of vibration-isolated equipment, should be modified to account for both equipment flexibility and support condition.

ACKNOWLEDGEMENTS

This study was supported by the American Society of Heating, Refrigerating, and Air-Conditioning Engineers (ASHRAE) and by the Earthquake Engineering Research

Centers Program of the National Science Foundation (NSF) under Award Number EEC-9701471 to the Multidisciplinary Center for Earthquake Engineering Research (MCEER). The authors gratefully acknowledge the support and guidance received from the members of ASHRAE Technical Committee 2.7, Mason Industries, Kinetics Noise Control, York International, and the technical staff of the Structural Engineering and Earthquake Simulation Laboratory (SEESL) of the Department of Civil, Structural, and Environmental Engineering at University at Buffalo. Any opinions, findings, conclusions, and recommendations presented in this paper are those of the authors, and do not necessarily reflect the views of the sponsors.

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Using Building Information Modeling to Improve the Mechanical, Electrical, and Plumbing Coordination Process for Buildings

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ABSTRACT

The coordination of mechanical, electrical, and plumbing (MEP) systems has become a major challenge for building contractors. Specialty design firms and trade contractors, working independently, provide these systems. The coordination of these systems involves locating equipment and routing Heating, Ventilating, and Air-Conditioning (HVAC) duct, pipe, and electrical raceway in a manner that satisfies many different types of constraints. For the past several years MEP coordination has involved sequentially comparing and overlaying drawings from multiple trades, in which representatives from each MEP trade work together to detect, and eliminate spatial and functional interferences between MEP systems. This multi-discipline effort is time-consuming and expensive. With the recent development of Building Information Modeling (BIM) this process is now able to evolve with the software technology. Using examples from several case studies, this paper demonstrates how BIM can improve the MEP coordination process in buildings. The results of this research provide a foundation for a revised work process utilizing modeling software and information technology.

Key Words: Mechanical, Electrical, Plumbing (MEP) coordination, MEP interferences, Heating, Ventilating, and Air-Conditioning (HVAC), Building Information Modeling (BIM), Information Technology

Introduction and Need for MEP Coordination

Mechanical, electrical, and plumbing (MEP) systems are the active systems of a building that temper the building environment, distribute electric energy, allow communication, enable critical manufacturing process, provide water and dispose of waste [Barton 83]. MEP systems have increased in scope on many types of projects, due to the increased requirements by building users. With the need for increased functionality of these systems, projects now include much more than the traditional mechanical, electrical, and plumbing systems such as fire protection, controls, process piping, and telephone/datacom. Although many of these systems seem similar in nature, different specialty contractors often install them.

MEP coordination is defined as the arrangement of the building system components that must fit within the constraints of architecture and structure; furthermore, MEP coordination is a critical activity for efficient construction and acceptable system operation. The process involves defining the locations for components of building systems within the constraints of the envelope defined by the architectural and structural systems, and meet performance expectations for comfort and safety. The level of difficulty associated with this process directly relates to the complexity and number of building systems in a facility (Riley 1997). Ideally, the result of a coordination effort is an economical arrangement that meets critical design criteria and performance specifications.

While estimators can forecast the building systems cost, estimating the cost of coordination is more difficult. One general contractor estimated that MEP coordination costs as much as six percent of an individual building system (Sanchez 1998). An electrical contractor noted that the cost of coordination approximately equals the cost of electrical design. Each is about three percent of the total cost for electrical systems (Bergthold 1998). Many construction industry professionals have cited MEP coordination as one of the most challenging tasks encountered in the delivery process for construction projects (Tatum 2000).

The need for MEP coordination grows out of the fact that designers do not provide detailed designs for fabrication and installation of building systems. Contracts typically indicate that documents provided to building contractors only show a general arrangement of equipment and accessories to be located inside the building. In addition, construction contracts typically require the contractors who install these systems to work with each other to identify and resolve interferences that may occur during the installation of their work. The examples of contract language shown below come from both design-build and design-bid-build contracts:

The general contractor shall coordinate all equipment and accessories with all the trades and shall furnish any information necessary to permit installation with the least possible interference or delay. (PDL 1998)

It is the responsibility of the Heating and Ventilating Contractor to coordinate the work with the Plumbing Contractor and Electrical Contractor and other subcontractors. (Lam 1995)

The contractor shall cooperate with the other subcontractors in order to establish the responsibilities of each so that work can be completed without delay or interference. (Alza 1997)

The contract documents show the general arrangement of equipment, ductwork, piping, and accessories. Provide offsets, fittings, and accessories, which may be required but not shown on drawings. Investigate the site and review drawings of other trades to determine conditions affecting the work and provide such work and accessories as may be required to accommodate such conditions. (Stanford 1996)

In order to meet contract requirements, contractors must produce detailed design drawings, generally termed shop drawings, prior to construction. These drawings include actual dimensions and locations of system lines and components. The contractors must then internally review the design drawings and check them against all code and regulatory requirements. Upon completion of this task, the coordination process may begin. The following section of this paper describes the process in detail.

Description of current process using two-dimensional paper drawings

The MEP coordination process begins after all preliminary design drawings are completed, which entails all components (e.g., conduits, cable tray, pipe, HVAC duct) sized, engineering calculations completed, and diagrammatic drawings produced. Routing is the only aspect that is not defined. Representatives from each trade involved begin by holding a series of meetings using a light table to determine routing for their drawings.

During coordination meetings, the participating specialty contractors compare preliminary routing for their systems to identify and resolve conflicts. They typically overlay transparent design drawings on a light table. The most common method used today is the sequential comparison overlay process evaluation (SCOPE), which continues until all interferences are resolved. This process often requires preparing section views for highly congested areas to identify interferences. There is a considerable amount of negotiation to determine which contractor(s) will need to revise their design and/or submit requests for information (RFI) regarding problems that require an engineering resolution. The product of this process is a set of coordinated shop drawings that the participants submit to the design engineer for approval.

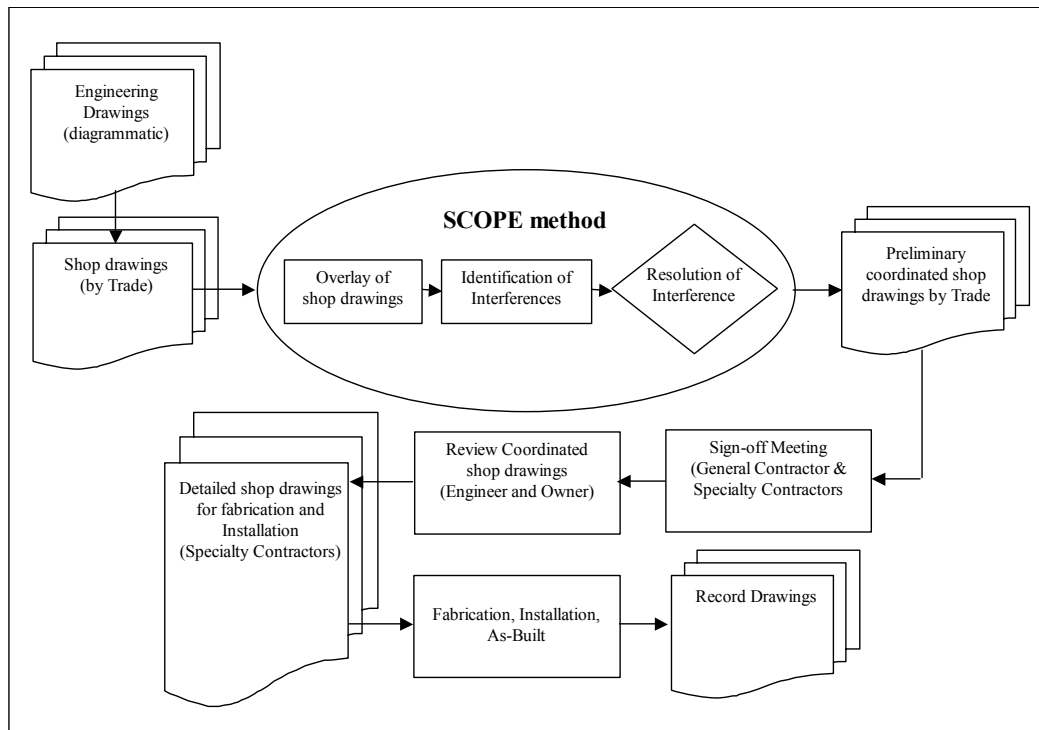


Figure 1 - Current practice (Korman 2001)

Common constraints for the group to consider in routing the MEP systems are corridors, openings in shear walls, and architectural requirements - such as ceiling type and interstitial space. Each trade initially routes their system to their own advantage. This includes decreasing overall length, routing close to support points, choosing prime locations for major components, and locating system runs to facilitate the construction needs of their own trade.

Upon completion of the overlay process, the MEP trades involved “sign-off” on each other’s drawings, indicating that they accept the coordinated design for the specific area of the facility. Specialty contractors then prepare cut sheets for duct fabrication and spool sheets for piping, based on the coordinated shop drawings. Once construction and installation of the systems is completed, the contractors prepare as-built or record drawings by marking and editing the shop drawings or by consolidating electronic files.

Detailed description of the Sequential Comparison Overlay Process Evaluation Method

The SCOPE method begins with the HVAC specialty contractor laying down a transparent drawing of the HVAC dry system on the light table for comparison with all other shop drawings. The drawing displays the preliminary routing for the system. Specialty contractors commonly use the HVAC dry system as a base because it has the largest components, primarily composed of large ductwork and variable-air-

volume (VAV) boxes. They are hardest to relocate. Large duct sizes restrict the routing to a few locations where adequate space is available.

The HVAC wet (hot and cold-water) system is the first system to overlay the HVAC dry system because it directly feeds into the HVAC dry system. The HVAC dry and HVAC wet systems work together and must be tightly coordinated. Routing of the HVAC wet system is based on the HVAC dry system routing and location.

Next, the plumbing system, including all graded lines, waste lines, and vent lines, enters into the coordination process. The requirement to slope all graded lines and waste lines to allow for gravity flow gives the plumbing system the next highest level of priority after the HVAC dry system. The gravity drain lines typically slope 1/8 inch for every foot (1cm per meter). This requirement forces the drain lines to compete with the large HVAC dry ducts at the higher elevations because they must start as high as possible to maintain the grade without falling below the ceiling tiles. Engineers route HVAC dry ducts at higher elevations because of their large volume.

Where process piping is included as a building system, it is coordinated following the plumbing system. Most process piping systems are pressure-driven and thus can yield to larger building system components and gravity-driven system lines that are more difficult to re-route due to the risk of affecting their functionality. In cases where a special routing is required for process piping to function at its optimal performance level, engineers assign priority to the process-piping system.

Next in the coordination process is fire protection. This is a pressure-driven system; however, the fire protection main lines must be slightly graded to allow scheduled draining as required by operations and maintenance. This complicates the coordination of the main lines. Engineers and contractors compare drawings individually with HVAC dry, HVAC wet, and plumbing systems.

Consideration of the electrical system follows the fire protection system. Engineers consider the electrical system to be one of the more flexible systems because the components are generally smaller and installers can easily route electrical conduit in the field. However, this is only true for branch conduits that can be as small as 1/2-inch (15 cm) in diameter. Larger electrical conduits receive a higher priority due to the greater the number of elbows and bends, and pull-points required to install cable.

Contractors coordinate the control systems and telephone/datacom systems last. The control system is the most flexible because of its smaller diameter tubing and conductors. Components in the control system run along other systems, such as HVAC dry and process piping. Therefore, specialty contractors usually coordinate the control system in the field. The primary problem with the telephone/datacom systems is induced electrical interference in routing these lines adjacent to electrical distribution cables. When required, engineers develop separation criteria and avoid this problem by segregating telephone/datacom lines from higher voltage lines by a minimum distance.

Problems with the current practice and how Building Information Modeling can improve the current process

Many problems exist with the current practice (see Figure 1). The coordination process is slow and expensive. MEP coordination often delays the project and increases the cost for all involved in the process. Coordination is often not budgeted in the construction cost - it is a hidden cost in the design category. The sequential and iterative process is very slow because only slight progress is made at each meeting. These coordination meetings consume valuable human resources with up to seven people at each meeting.

The coordination process is also highly fragmented. Design and coordination take place on an as-needed basis, and are often not performed sequentially. The desired scenario is for design changes to initiate coordination changes quickly and vice versa. Another complication is the lack of knowledge and understanding of the multiple disciplines involved, which often gives rise to systems that need redesign to meet coordination criteria. It is often the case that parts of the systems must be re-coordinated in different stages.

There are many challenges to improving the current work process for MEP coordination, but it also provides an opportunity to improve project performance. When performed manually, MEP coordination requires considerable time from scarce experts who have specialized knowledge about the design, construction, operation, and maintenance of the MEP systems. Building Information Modeling software is a relatively new tool which provides the potential to improve the current work process for MEP coordination.

Definition of Building Information Modeling

Building Information Modeling (BIM) can be defined as the process of creating an intelligent and computable three-dimensional (3-D) data set (model) that can be shared among the various professionals within the design and construction team. However, it is not only defined by simply creating a 3-D data set for internal analysis; but is defined by a digital 3-D data set that contains geographical representations and knowledge about the geographic representation. For example, when a project is modeled in 3-D, BIM can enable the designer, engineer and builder to visualize the entire scope of a building project in three dimensions, rather than the traditional two-dimensional (2-D) or flat drawings that are the most common form of construction documents. In addition, knowledge regarding the cost of specific building elements can be assigned to individual geographic representations as well as knowledge regarding the schedule for installation. Therefore, most professionals refer to BIM as the process of using a 3-D model to improve collaboration among all project participants.

Building Information Modeling software available for MEP coordination

Today, there are many examples of BIM software that designers and builders can use to model their project. ArchiCAD 11 is a software program that allows a user to draw in 3-D or import a 2-D drawing to create a 3-D model. In addition, it allows the user to toggle back and forth from 2-D to 3-D. AutoCAD, a 2-D drafting program, when coupled with Revit, also enables the user to create a 3-D model. Bentley is another powerful 3-D modeling program that is widely used on industrial projects to create 3-D models. Perhaps the most promising software tool available for MEP coordination is NavisWorks which is able to interpret geographic representations from all other software programs. In effect, NavisWorks is to software today what the Rosetta Stone was to interpreting languages in the past - it has the potential to unlock and/or interpret other 2-D and 3-D drawings.

Revised Work Process using Building Information Modeling

Using BIM, one can envision how designers and builders can plan out, in precise detail, the location and clearances needed for a complete and successful project. The use of 3-D modeling can enable the engineers and builders to visualize the size, routing, relative positioning among MEP systems as well with the structural system and key architectural features. In addition, it can be used to identify conflicts or clashes. Figure 2 shows a revised process for MEP coordination utilizing BIM. The revised process begins with separate CAD files containing each building trades system. Using a software integration tool, such as NavisWorks, CAD files are merged into a common 3-D model. A clash detection software application is then activated to identify physical interferences.

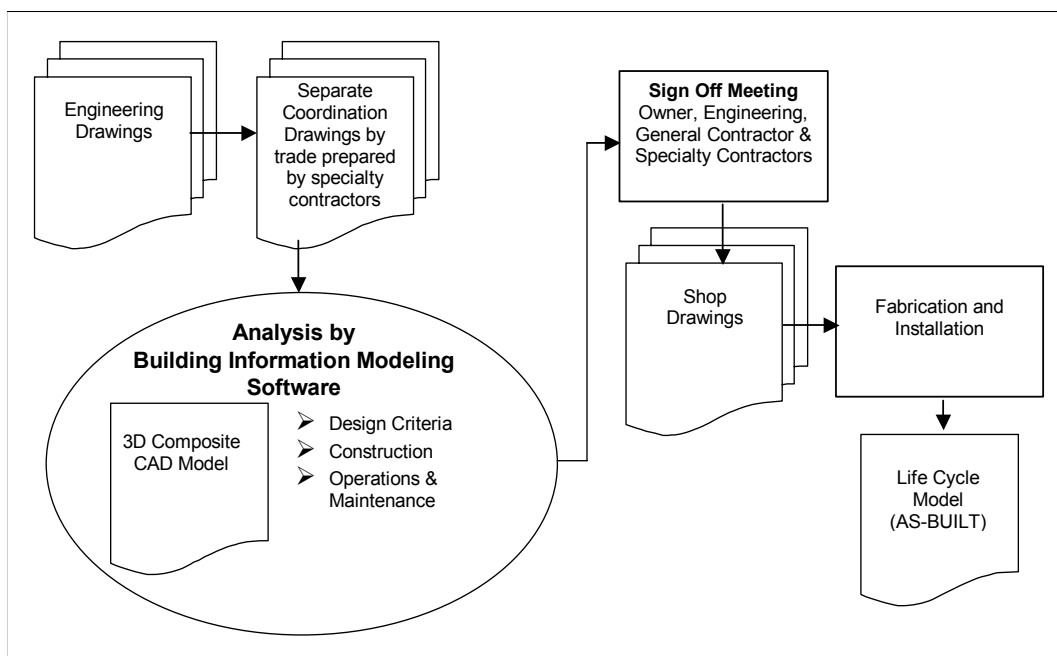


Figure 2 - MEP coordination using BIM

The process for identifying interferences must still be performed sequentially, due to the fact that the current software available today, such as NavisWorks, will not allow the geographically represented model to be modified – the model can only be used for visualization purposes to notify users of problems regarding clash locations. Building trades must modify their individual systems using the software program they originally used to produce the geographically represented models before trying to resolve of the interferences again with the clash detection software application. Once revised and modified, the building trades can resubmit for analysis and hopefully there are fewer or no clashes. Obviously, even with the most common building systems trades involved – mechanical, electrical, and plumbing, the process can still be challenging. For example, a model containing the standard MEP building systems may still involve four or five contractors: Sheet metal, HVAC process piping, water distribution piping, sanitary drainage piping, and electrical conduit and cable. Figure 3 displays a recommend sequential comparison method using BIM software.

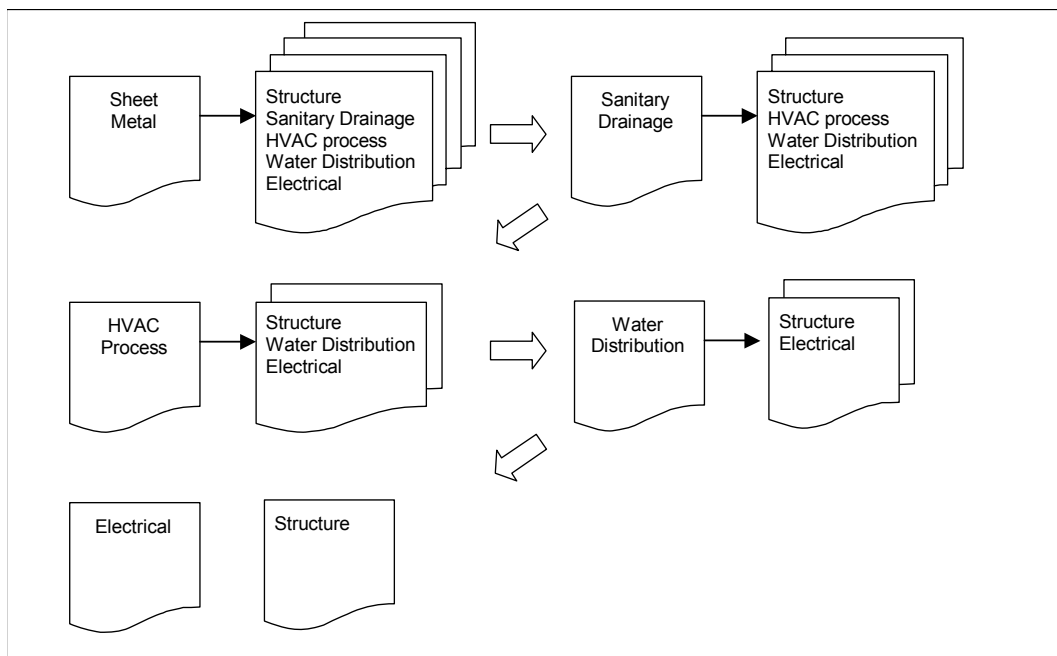


Figure 3 – Recommend sequential comparison method using BIM software

When one model is shared among the project team, physical conflicts between the structure, mechanical, electrical, and plumbing systems can be identified early in the design process and resolution is expedited and the building trades are not restricted to only relying on paper plans and written specifications. Upon resolution of all interferences, separate drawings for fabrication and installation could be produced for each system [Korman 01].

Conclusions and Recommendations for Future Research

MEP coordination requires considerable time from scarce experts who have specialized knowledge about the design, construction, operation, and maintenance of these systems. As mentioned above, effective MEP coordination requires recalling and integrating knowledge regarding design, construction, operations, and maintenance of each MEP system, not only resolving physical interferences and clashes. Missing from BIM models is that knowledge regarding each system. A revised work process utilizing BIM still requires individuals to meet and share knowledge regarding their system. Currently, BIM can only assist in resolving physical conflicts; however, coordination must satisfy critical design criteria, evaluate constructability issues, and address operations and maintenance concerns. During coordination, trades must consider all aspects from design, construction, and operations and maintenance. It is difficult to integrate construction knowledge, and operations and maintenance into the MEP coordination process. Often the parties involved do not take the opportunity to align goals and define requirements. In addition, constructability issues are not considered part of the MEP design consultants' scope of work, and designers must make assumptions about constructability or ignore the issue totally. Furthermore, there is a lack of understanding between the different MEP trades. Each discipline focuses on its own design and construction requirements. Failing to consider the big picture, many MEP contractors are unaware of unique installation requirements for other trades and are reluctant to learn more about or consider each other's systems. Implementing a revised work process that uses information technology that is able to integrate a number of knowledge bases – design criteria, construction, operations, and maintenance – into a BIM system could provide valuable insight to engineers and construction personnel assisting them in resolving coordination problems for multiple MEP systems.

In addition, in order to use BIM effectively to assist in the MEP coordination process, it is authors' opinion that indicators of good coordination results need to be defined. Resolving all physical interference during the MEP coordination process does not necessarily ensure that a facility has been well coordinated. This idea stretches the concept that there are multiple types of interferences beyond physical interferences. Follow-up research will include developing guidelines to assist in the resolution of multiple types of interferences.

Acknowledgements

The researchers would like to thank the many progressive design and construction firms that served as members of an industry advisory group for this research. Representatives from these firms provided access to construction sites and facilitated interviews with personnel actively engaged in MEP coordination. The researchers would also like to thank the Stanford University Center for Integrated Facility Engineering for funding the initial research phase. In addition, the authors would like

to thank Webcor Construction for the donation of NavisWorks software to the Construction Management Department at Cal Poly State University, San Luis Obispo.

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Innovative Products for Curtain Walls: Investigation of Anti-Reflective Glass Coating

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Abstract

Over the past few decades, glass coating technology has played a key role in the advancement of curtain wall design. From reflective to low-emittance (low-E) coatings, the available range of materials has provided the designer with greater flexibility in the areas of aesthetics and thermal performance of curtain walls. One of the more recent advances has been in the area of anti-reflective (AR) coatings for glass. Initially intended for items such as eyeglasses and museum display cases, anti-reflective coatings are now being applied to glass in commercial storefronts and curtain walls. This allows for clearer viewing through the glass in applications such as building lobbies, shopping stores, and stadium press and sky boxes by reducing the amount of light that is reflected back at the viewer.

This case study will focus on the technology and potential problems with AR coatings exposed to the exterior. The case study involves glass installed in a sloped curtain wall that clads luxury skyboxes at a large Midwestern stadium that was renovated within the past five years and included the replacement of all seating and construction of luxury skyboxes along the east side of the stadium.

The case study investigation involved staining of the coating during construction, the composition and placement of the coating, and remediation options considered. Basics of coating types and technology will be discussed as well as actual field surveys and lab testing performed as part of the case study.

Background

Coatings on glass are not a new technology. For example, Low-emissivity (Low-E) coatings have been applied to glass for years in order to improve the thermal capabilities of architectural glass. In recent years, specialty coatings, such as anti-reflective, have been used on architectural glass for storefront and curtain wall applications. The purpose of AR coatings is to reduce the reflected light from the surfaces of the glass and/or increase the amount of light transmitted through the glass. Clear insulated glass units typically have a transmission value of 85% with 15% of the light being transmitted back at the viewer. Depending on the number of coated surfaces, the application of an AR coating can reduce the amount reflected back to around 1%. Therefore, coated glass provides clearer viewing through the glass for such uses as retail storefronts and outdoor stadium press boxes and luxury suites.

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The coating is typically applied using one of the following methods:

- Sputter coating: Sputtering uses a vacuum chamber to place layers of coating on the glass after it has been formed and cut. The total thickness of the coatings is approximately 250 nanometers. Sputtered coatings are often referred to as 'soft coats' and can be more susceptible to environmental effects than pyrolytic coatings.
- Pyrolytic coating: Pyrolytic coating deposits a metallic oxide directly onto the glass surface while it is still hot. The coating is effectively 'baked-on' to the surface making it very hard and durable. The pyrolytic coatings are often referred to as 'hard coats'. Pyrolytic coatings can be up to 20 times thicker than sputtered coatings and the baking process makes them much harder and resistant to wear.

From January of 2002 to September of 2003, a large scale renovation was performed at a large, Midwestern outdoor stadium. The renovation included the retention of the original facades of the stadium but with an entire new seating bowl constructed inboard of the existing structure. Four levels of luxury suites were constructed above the new seating bowl on the east and south sides of the stadium. The suite levels are clad with a sloped aluminum and glass curtain wall system as shown in Figure 1. Fixed vision lites are located at each level with operable transom awning windows above. The system consists of captured horizontal members with pressure plates and snap covers and structural silicone glazed verticals. The vision lites consist of insulating glass (IG) units with a laminated, tempered outboard lite and single pane, tempered, inboard lite. The glass of the operable units is coated with a ceramic frit pattern while the vision lites are coated with an AR coating.



Figure 1 - Typical view of sloped curtain wall at suite level

The project specifications stated that the AR coating was to be applied “on surfaces needed to reduce glass reflectivity to a minimum obtainable level when viewing the playing field from interior of the skybox.” After discussions with the architect-of-record and general contractor, it was discovered that the coating was applied to surfaces 1, 4, 5, and 6 of the suite vision glass. See Figure 2 for illustration showing glass surface numbers. The literature for the proprietary coating that was chosen stated that the abrasion resistance of the coating was shown to perform comparably with uncoated float glass surfaces.

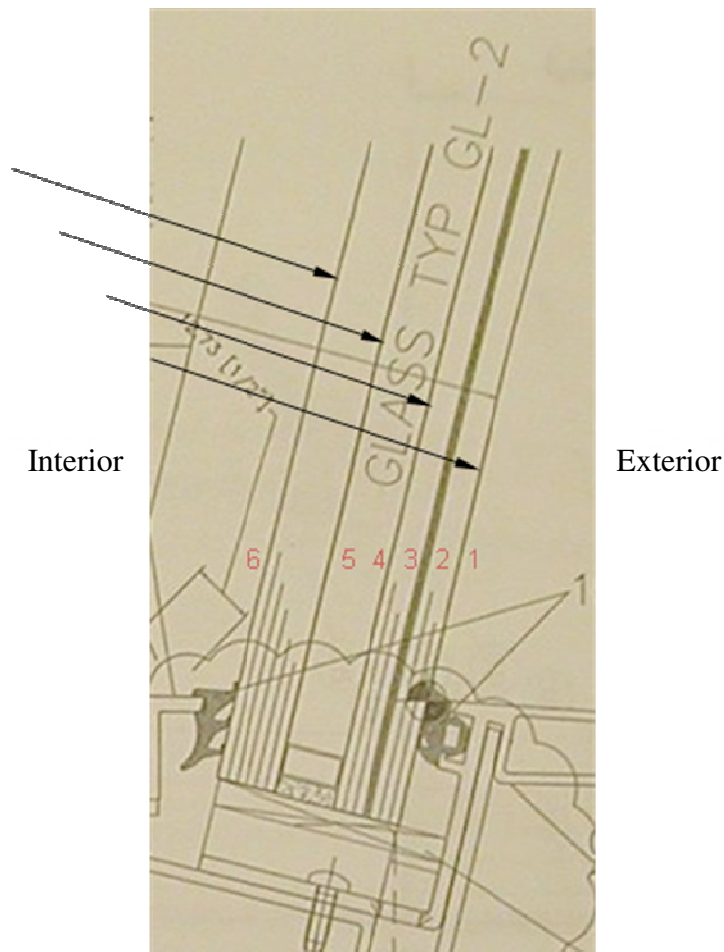


Figure 2 - Illustration showing the composition of suite vision lite. Arrows denote surfaces where AR coating is applied.

Problem

In early September 2003, less than three weeks before the scheduled re-opening of the stadium, streaking of the vision glass in the suites was observed by the general contractor. The streaking was observed on both the interior and exterior of over 100 vision lites at various locations along the suite facades and manifested itself as either an iridescent staining or deposit on the glass as the result of evaporation as shown in

Figures 3. All vision lites were visually inspected by various parties including the general contractor and forensic consultants. The inspections revealed that the severity and amount of the streaking varied with the worst areas affecting the view of the field from the suites. The two main questions that arose out of the preliminary inspection were:

1. What was the cause(s) of the streaking and staining?
2. Is there an effective method of removing the streaking and staining without damaging the AR coating and therefore compromising the performance of the coating?



Figure 3 - View of streaking at suite vision lite.

An initial series of cleaning trials was performed by the general contractor in an attempt to remediate the staining. The following products and or/processes were performed as part of the trial:

- Vinegar and non-abrasive cleansing pad
- Vinegar and terry cloth
- Proprietary silicon dioxide glass polish and non-abrasive cleaning pad
- Proprietary silicon dioxide glass polish and terry cloth

The cleaning trials were not effective in removing the staining or streaking and in some cases made the problem worse. In some locations, the stain was removed but the cleaning created a cloud on the surface of the glass as shown in Figure 4. The initial opinion of the forensic consultants, with input from the glass fabricator, was

that the coating may have been damaged by the streaking and exacerbated by the subsequent cleaning trials.

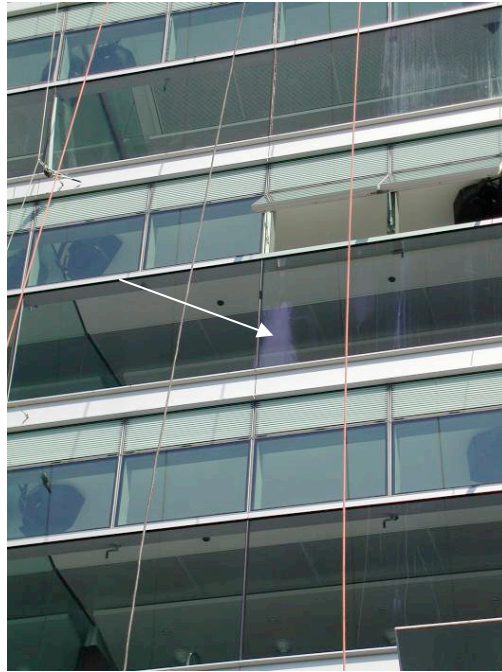


Figure 4 - Arrow indicates "clouded" trial cleaning area.

Based on the results of the trial cleaning, inquiries about the composition of the AR coating were made. Because it is a proprietary material, the glass fabricator would only say that the coating is made up of four layers and that the outer layer is silicon oxide. The glass fabricator recommended trying cerium oxide, scraping with alcohol and razor blades, or tri-chloro-ethylene (citrus cleaner) as methods of cleaning. The fabricator also stated that the coating may be damaged by any of these or other methods if not done with caution.

Application of cerium oxide slurry, scraping with alcohol and razor blades, and citrus cleaner were performed at separate locations. No further damage was observed after each method was performed. None of the products were able to completely remove the iridescent stains while the razor blades and alcohol were able to remove some of the deposits. The decision made by the owner's representative and general contractor was to mitigate the problem for first game was to scrape the worst areas with razor blades to remove the most visible deposits. This was performed as a temporary "stop gap" remediation while the investigation of the cause and remediation of the streaking was ongoing.

Investigation

To answer these questions, it was agreed by all parties that a vision lite, with some of the worst staining, would be removed for laboratory analysis. A vision lite was

removed for storage and analysis. Prior to the removal of the lite, a six inch high aluminum strip was bonded to bottom of unit on the number 6 surface to preserve “virgin” material for analysis. The following analysis protocol was used to determine what caused the damage and to determine if cleaning is possible without damaging the coating.

Phase I

Step #1 - Preparation of removed vision lite

A wood frame was built to set the unit in an upright and vertical position to allow inspection from both sides as shown in Figure 5. The frame was constructed with wheels in order to allow the unit to be more easily moved. The unit was then picked up by crane and set into the frame. The inboard and outboard lites were overlaid with a grid pattern to define coordinate locations for later sampling. The grid consisted of nylon line stretched taut at 6 in. o.c. horizontally and vertically.



Figure 5 - Vision lite placed in wood frame for further inspection and analysis.

Step #2 - Analysis

The unit was photo documented indoors and outdoors under various lighting conditions. The aluminum strip at the bottom portion of the interior side of the unit was removed and a 6 in. piece of the strip was retained for further analysis. Some of the remaining deposits under the aluminum were removed for analysis by scraping the samples dry from the glass as well as by applying a mist of de-ionized water, alcohol, and/or other solvent to the face of the glass and collecting the deposits by scraping them into sample vials.

Preliminary results of the analysis included:

- There are general similarities between all samples with respect to the inorganic materials that are present in the deposits on the glass under the aluminum strip.
- The deposits under the aluminum strip consisted of various forms and ratios of calcium oxides, calcium silicates, silicon oxides and other miscellaneous, occasionally seen particles (such as titanium oxides, iron oxides, sodium oxides, salts, zinc, manganese)
- The majority of the oxides present are more crystalline than amorphous (may indicate devitrification)
- Sulfur is sometimes present as a calcium, oxygen, and sulfur compound at least one of which was consistent with calcium sulfite -sulfur may also be present in other oxide compounds or as free sulfur
- Flakes of niobium-rich material were located in Areas A and B.
- Niobium appears to be component of one of the glass coating layers
- At least one area containing an acrylic-based polymer (sampled before application of the aluminum strip) has been identified and there may be more areas; unfortunately, the adhesive used on the aluminum strip is also acrylic-based.
- Silicone caulk containing calcium carbonate has been identified from Grid A10.

The analysis performed in Phase I of the investigation showed that the deposits on the glass contained various construction materials and debris consistent with materials used on the project. It also showed the presence of niobium which is not a component that is not typically found within usual construction or glass materials. Therefore, niobium was believed to be part of the AR coating.

Phase II

Step #3 - Trial cleaning

An agreed upon area of the inboard lite of glass above the aluminum strip was cleaned prior to glass cutting and sampling. Trial cleaning was performed using a cerium oxide slurry which consisted of approximately 2 parts cerium oxide powder to 1 part water. The slurry was applied in different locations using different methods including by hand and drill with a felt buffing pad as shown in Figure 6.



Figure 6 - View of trial cleaning with cerium oxide slurry

Step #4 - Separation of IG unit

The inboard and outboard glass lites of the vision unit were separated. The granular desiccant that fills the spacer bar was removed by drilling into the spacer and vacuuming out the desiccant. The lites were separated by cutting the sealant that bonded the metal spacer to the glass.

Step #5 - Preparation and sampling of the inboard lite

A clear plastic film was adhered to the airspace side inboard 1/2 in. thick tempered lite to retain glass pieces to the film after the lite is broken. In a small area the film was not adhered to the glass to provide an adhesive-free coated surface for analysis after the lite was broken. The glass was secured in a wooden frame to secure the samples in the original position. The inboard 1/2 in. thick tempered lite was broken. Samples of the glass pieces were taken in several areas. Microscopic analysis was performed on the samples. The analyses performed included:

- Light microscopy: stereomicroscopic study and polarized light microscopy (PLM) of particulate residues on the glass.
- Infrared microspectroscopy (IR) of organic residues on the glass.
- Low vacuum scanning electron microscopy (SEM) and energy dispersive x-ray spectrometry (EDS) of inorganic residues on the glass.
- Field emission high resolution scanning electron microscopy (SEM) at low voltages to study surface features on the glass.

- X-ray photoelectron spectroscopy (XPS), also known as electron spectrometry for chemical analysis (ESCA), for the identification of elements and chemical states (metals and oxides) of the surface coatings using a penetrating power on the order of 5 to 50 Å.

The testing revealed the following:

The layer structure of the coating was:

- | | |
|--------------------|--------------------|
| ➤ Silicon oxide | 60-80 nanometers |
| ➤ Niobium oxide | 130-150 nanometers |
| ➤ Silicon oxide | 30-50 nanometer |
| ➤ Titanium dioxide | 12-20 nanometers |
- Scratches consistent with razorblade scraping were present in the deposits and in the polished areas of the glass. A majority of the scratches appeared to be within the coating system) perhaps even within the exterior silicon oxide layer) but a few penetrated completely through the coating to the glass below.
 - Scratches from the cerium oxide polishing were not found; however, given the sub-micrometer size of the cerium oxide material, the scratches could be present and below the resolution of the equipment, as employed to date. This small scratch size would not preclude their influence creating a haze.
 - Removal of the exterior silicon oxide layer as a result of cerium oxide polishing varies. The maximum amount of removal that was noted was in the 40-50% range.
 - Polishing with cerium oxide did not remove all of the deposited stain or the iridescent portion of the stain.
 - The iridescent portion of the stain appeared to contain zinc oxide. The most logical origin of the zinc would be from galvanized metal used in the infrastructure of the building.

See Figure 7 for a chart showing the relative thickness of the layers at the cleaned and non-cleaned areas.

XPS Depth Profile Measurements of Top SiO_x Layer Thickness

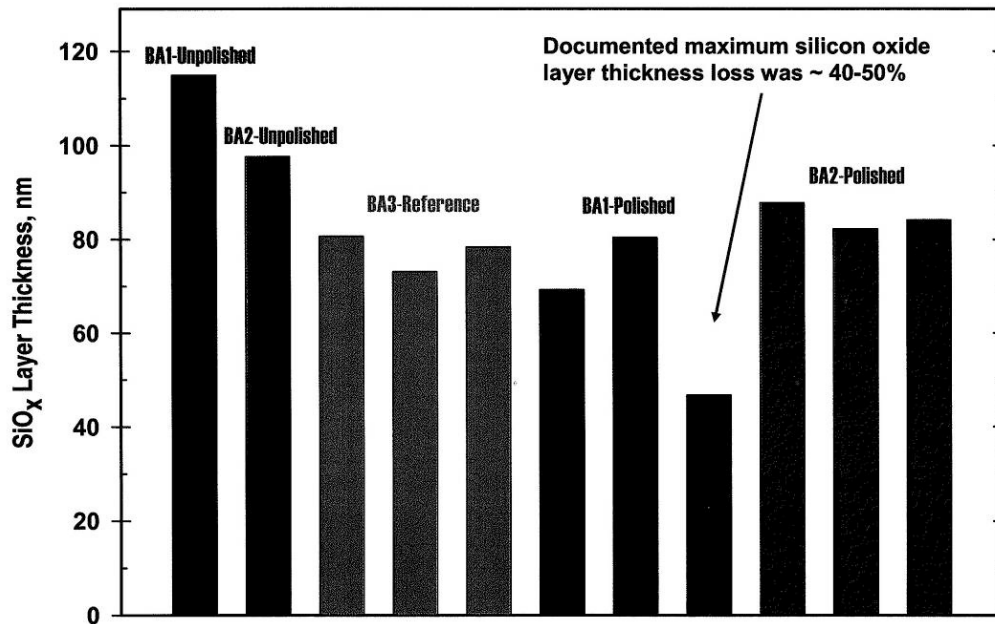


Figure 7 - Chart showing relative thickness of coating at cleaned areas

Conclusion

Based on the investigation, the general conclusions were:

- The streaking and staining of the vision glass in the suites was most likely caused by water containing construction materials and/or debris running over glass.
- Cleaning with a cerium oxide slurry may be effective on units with light streaking. However, the cleaning could remove a portion of the outer layer of the coating therefore slightly affecting the overall performance of the AR coating.

Renovation of a 1970s Glass Curtain Wall

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Abstract : A ten-story downtown Chicago office building, built in the early 1970s, had been experiencing insulating glass unit failures, water leakage, and problems with occupant comfort. Several investigations in the 1990s concluded that the entire curtain wall system was deficient and required replacement. However, the office building houses the offices for the Social Security Administration's Midwest Region, and each month serves over 9 million beneficiaries. Disruptions to this operation would be catastrophic. Based on careful study and analysis of the existing curtain wall aluminum framing elements and connections to the building structure, it was concluded that the framing could be reused and was capable of supporting a supplemental cladding system attached to its exterior face. A pre-glazed unitized curtain wall system was designed which was hung to the original framing after the original glass was removed. This solution reduced the disruption to occupants, enhanced the speed of installation, and reduced the time period between deglazing and new cladding installation to a matter of days. The work was largely performed from the exterior of the building starting in the fall of 2007 and is scheduled to be completed by the spring of 2008. The replacement cladding has enhanced insulating glass, condensation control and water management; providing a cost effective solution to the client's problems.

Introduction

The ten-story Harold Washington Social Security Center (HWSSC) building was designed in the brutalist style; the structure is clad in a combination of monolithic exposed-aggregate pre-cast concrete and reflective-glass facade reminiscent of vintage 1970's government architecture - solid without making too much of a splash. The term Brutalist Architecture originates from the French term *béton brut*, or "raw concrete", a term used by Le Corbusier to describe his choice of material. Now approaching its 35th anniversary, HWSSC was built for, and still remains, the central offices of the Social Security Administration (SSA)'s Midwest Region - a region that spans 6 states and serves 9 million beneficiaries of Social Security checks each month. The most noticeable feature of the building for most passerby's isn't the building at all, but rather the monumental 100 ft. tall metal-lattice sculpture of a baseball bat by the famous sculptor, Claes Oldenburg that stands in its entrance court.

When the building was built it stood relatively alone in a “skid row” of dilapidated brick structures and empty lots. Today, the building is surrounded in the ever expanding Chicago downtown, surrounded by modern metal and glass office towers and high-rise apartment blocks.

The Challenge

The building’s most prevalent problem, failing glass, is almost as old as the building itself. Records back to the 1980’s, within 10 years of the building’s commission, reveal a history of continued and repeated failure of the rather large insulating glass (IG) units that clad half the facade. The failure was noticed by employees inside as a distinctive, and rather disturbing, “fogging-up” of the glass as shown in Figure 1. A short time after this was first noticed on any glass panel, the fogging would advance until vision through the glass was impossible. In addition to the fogging glass, glass breakage was common and in a few instances, the air space between the glass would fill with water. Given a rather tepid response to this problem, tenants would endure long periods with visionless windows. From the outside, the effect rendered a “checker box” appearance to the façade as shown in Figure 2. The failed IG units were quite distinctive next to their reflective glass neighbors.



Figure 1: Fogging of IG unit observed from inside the building.



Figure 2: Fogging of IG units observed from the building exterior.

The government procured several studies of the problem which suggested a range of remediation; from simply replacing the glass with more modern technology, to the complete removal and replacement of the curtain wall. Glass replacement was later ruled out when replacement glass that was installed in the 1980s continued to fail as frequently as the original glass. The General Services Administration (GSA) turned to the design team, and told us to do what it took to fix the problem. Along with that they imposed only one non-negotiable condition - occupancy and operations of the building must continue unabated during any repair. The one concession, given the open office floor layout, was that occupants could be moved to temporary work stations on the same floor level for short durations. A repair was needed to solve the underlying problems that led to failed glass; be cost-effective; and be “transparent,” both figuratively and literally, to the employees of the building.

Background

The curtain wall system is a non-load-bearing wall that is supported on the structural steel frame. The curtain wall spans between floors and transfers lateral loads to the building structure. The IG units in the curtain wall system consist of two sheets of 1/4 in. thick glass with a 1/2 in. air space between the sheets of glass. A metal spacer with desiccant is placed between the two sheets of glass at the perimeter to maintain the 1/2 in. wide air space. The IG units are sealed with sealant around the perimeter to prevent moisture penetration into the sealed air space. The 1974 vintage IG units have a continuous stainless steel edge band around the perimeter of the units. IG units fabricated with this technology have a history of problems caused by water trapped between the glass and stainless steel edge band, leading to deterioration of the perimeter seal. Water or water vapor then can enter the air space between the sheets of

glass and fog the unit or corrode the metal spacer. Corrosion by-products can induce compressive forces on the glass and crack the glass as shown in Figure 3. Newer replacement IG units in the building do not have this stainless steel edge band.

The vision and spandrel each consist of IG units with similar construction. The spandrel IG unit has a reflective coating on the number 2 surface. Each IG unit is set on the outside of aluminum curtain wall mullions and rails and retained by an exterior applied, nominally 2 in. wide, aluminum pressure plate. The pressure plate is attached to the curtain wall with 1/4 in. diameter stainless steel bolts at approximately 12 in. spacing. A nylon spacer is specified at each bolt to hold constant the space between the pressure plate and the curtain wall. A stainless steel aesthetic snap-on trim is applied over the pressure plate. The glass is retained in place by rubber gaskets on the interior and exterior sides. A schematic section through the spandrel area reproduced from the original shop drawings shows the various curtain wall components in Figure 4.



Figure 3: Corrosion by-products can induce compressive forces on the glass and crack the glass.

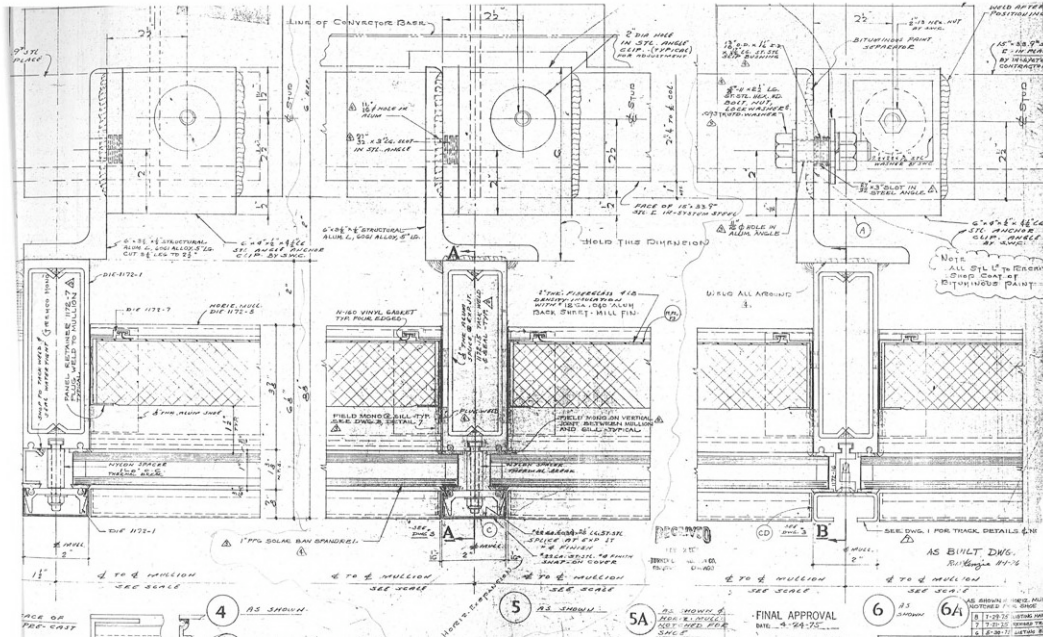


Figure 4: Schematic section through the spandrel area reproduced from the original shop drawings

The curtain wall connections to building’s structural steel frame are located at the intersection between each curtain wall vertical mullion and each building floor slab. Two types of connections are noted in the original shop drawings; a “gravity” connection which is intended to support the weight of the curtain wall as well as resist lateral wind loads; and a “wind” connection which is intended to resist lateral loads only. The gravity connections are specified at even floors, and wind connections at odd floors.

The curtain wall anchorage system for both the gravity and wind anchors as schematically shown in Figure 5 consists of the following:

- An aluminum angle welded to each vertical tube mullion at each floor level. The angle has a pre-punched vertical slotted connector hole.
- A steel angle welded to a “T” shaped steel section which in turn, is welded to a steel perimeter channel section.
- The aluminum angle is connected to the steel angle with a bolt, nut and washer. The gravity and wind anchors are detailed the same except that the washer at the gravity anchors is tack-welded to the steel angle, hence restricting vertical movement of the mullion.

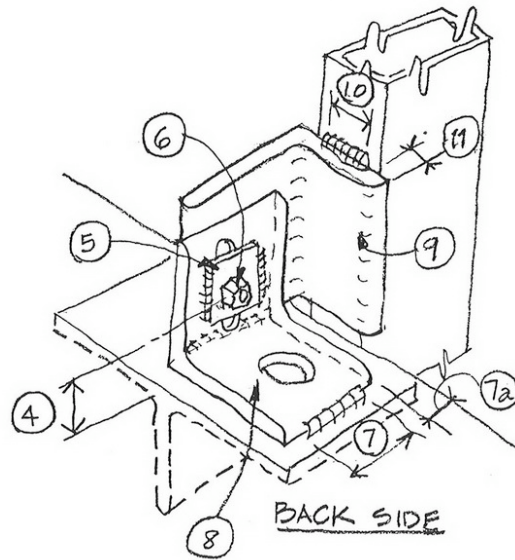


Figure 5: Schematic drawing of existing curtain wall framing connection.

The Challenge Refined

The design team considered two repair options to address glass failure on the building.

- **Remove and Replace** - remove the curtain wall materials including glass, all supporting metal and glazing materials and replace those with a new current standard glazed curtain wall
- **Over-clad** - identify curtain wall framing elements that are not defective and reuse those elements and then craft a new glass system that can clad over those existing elements.

Over-cladding quickly became the design team's choice, since it was less costly, less time consuming, and could allow compliance with the client's non-negotiable occupancy condition. Removal and replacement would have almost inevitably required massive tenant disruptions.

The next challenge was to determine what elements of the existing cladding could be salvaged. In-situ frost point testing performed on original IG units confirmed that even for original units that had not fogged, the glass had reached the end of its service life. The pressure plates, screws and glazing gaskets that retained the glass to aluminum framing were also in poor condition and not worth retaining. The aluminum framing behind the glass though looked acceptable. The new over-cladding would add more weight as compared to the original system. In addition, by introducing a water management system to the curtain wall that was not part of the original wall system, the new glass would have to be placed slightly outboard from its

original position. The curtain wall framing, and more critically the connections of the curtain wall framing back to the building structure would be subjected to additional dead load and eccentricity. To further compound matters, the design team wanted to verify that the new cladding would be compliant with present-day code-applied wind loads, which were 15-50% greater than the original design wind loads. In addition to an increased eccentric dead load, the existing anchors would be required to resist modern lateral loads.

A spot-check of the curtain-wall connections was deemed necessary to assess the integrity of the existing anchor connections and compare as-built conditions with information shown on original shop drawings. Spandrel glass panels were removed in the late 1990s by a glazing contractor concurrent with IG replacement work. Behind the IG spandrel was an insulation blanket and metal back pan which also was removed. Once these elements were removed as shown in Figure 6, the connections were clearly visible and carefully inspected for compliance with the original shop drawings and for the quality and length of field welds. At the same time that these inspections were on-going, the design team performed a spot check of the plumb and square of the framing and found them within reasonable tolerances. This provided additional confidence that the aluminum framing was a solid foundation to attach a new over-cladding glass curtain wall.



Figure 6: Existing curtain wall framing connection.

To assess the lateral load capacity of the existing curtain wall connections, the project team compared design pressures established by the City of Chicago, American Society of Civil Engineers (ASCE), and model building codes, which at the time was transitioning from Building Officials and Code Administrators (BOCA) to International Building Code (IBC). After review and comparing lateral load values obtained by each method, the design team established a lateral load force of 25 psf for the main cladding components and a lateral load force of 40 psf for the corner zone cladding components. Calculations were performed of the worst observed as-built condition at the spot-check locations. The calculations showed that reserve capacity exceed the new demands. This gave the design team confidence to pursue the over-clad concept.

Notwithstanding the design team's enthusiasm for the proposed solution, the GSA and SSA needed to procure adequate funding for this project; it took almost 8 years to move forward with implementation of the repair.

The Solution

In early 2006, the GSA re-contacted the design team and charted a plan to solve the long-lingering glass problem. The GSA as owner, along with the SSA as sole tenant, put together a team that now included a general contractor and a curtain wall specialty contractor as follows:

Engineer and Architect: Wiss Janney Elstner Associates, Inc.
General Contractor: Berglund Construction Company
Curtain Wall Contractor: Harmon, Inc.

Harmon further brought in Gardner Metal Systems, Inc. to fabricate the metal, and Viracon, Inc. to fabricate the glass for the new curtain wall.

Through 2006 plans progressed from the original design documents. Shop drawings, as shown in Figure 7, were prepared, reviewed and approved. The curtain wall was intended to be a shop-assembled glass and frame unit. Each typical unit was about 5 ft. wide and 13 ft. 0 in. high and contained one vision glass and one spandrel glass panel. A mock-up of the curtain wall system was built at an off-site test laboratory to verify key performance aspects of the curtain wall. Given the fact that the new curtain wall relied on the old framing, the lab set the stage for erecting the new curtain wall, by first building replicas of the original frames. Then the new curtain wall units were affixed in the same manner as intended on the project as shown in Figure 8.

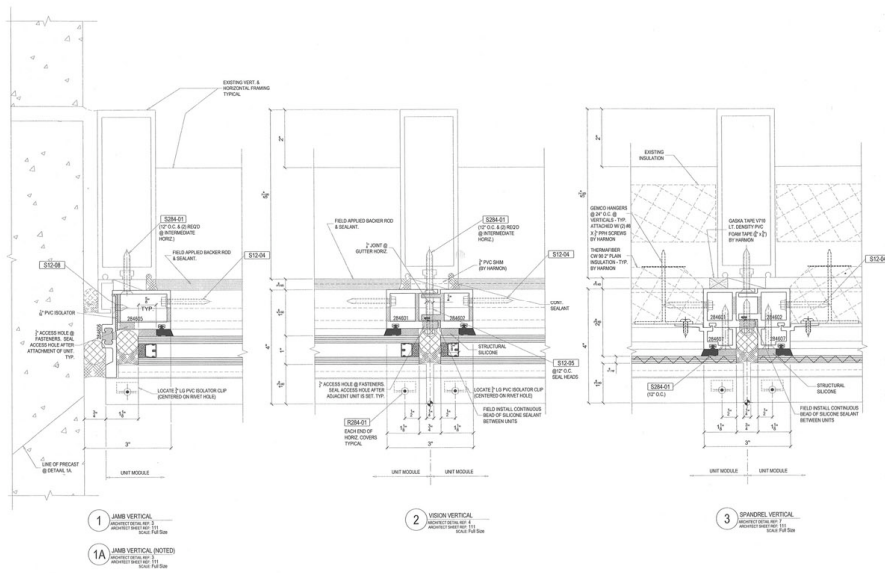


Figure 7: Overclad curtain wall shop drawings.



Figure 8: New curtain wall units attached to the laboratory mock-up frame.

In preparation for laboratory testing, the contractor performed an independent analysis of the existing curtain wall framing with superimposed design loads from the new system. Properties and profiles for the existing corner and typical aluminum mullions are shown in Figure 9. The contractor’s analysis performed in support of modeling the existing building for laboratory testing revealed that for simple span loading and a maximum moment equal to $wl^2/8$, the existing corner curtain wall mullions are overstressed. Furthermore, the midspan deflections exceed limitations and exceed stress limitations in the weld affected zones for the typical mullion. The design team pointed out that the curtain wall mullion length, including the corner and typical mullions, span two floors and have splices located at about 15-in. above alternating floor levels. The continuous support condition generates mid-span moments of approximately $wl^2/12$ and support moments of $wl^2/24$. Since the weld affected zone is not located in close proximity to the point of maximum moment, the actual stress in the weld affected zone is less than the allowable stress. In addition, the design team’s analysis pointed out that the existing corner mullions are braced against in-plane bending by existing horizontal rails located at approximately 6-ft. 6-in. on center. The existing horizontal rails effectively reduce the corner mullion mid-span moment and the maximum stress is less than the allowable stress. Laboratory testing confirmed that mid-span deflections for the corner mullions are within acceptable limitations.

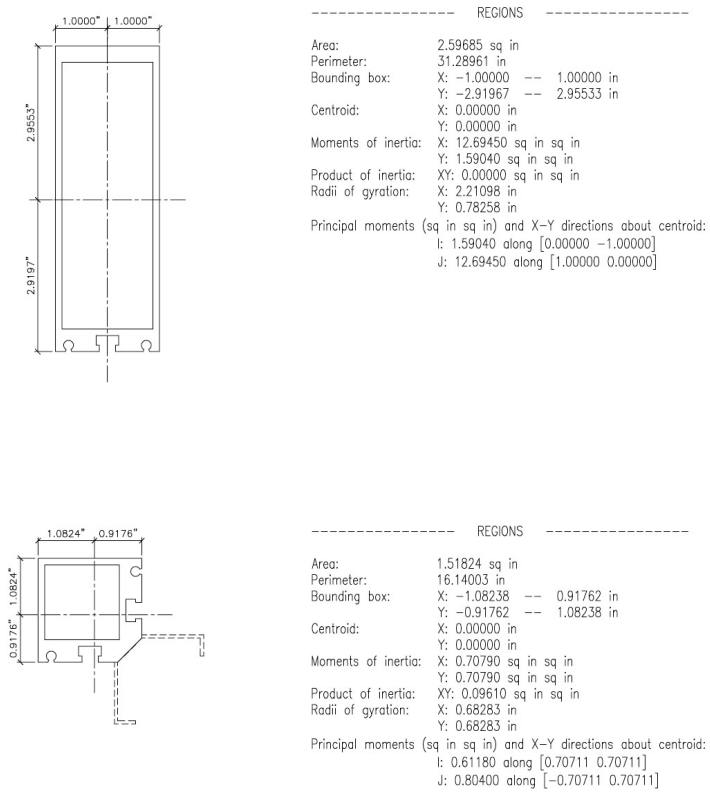


Figure 9: Profiles and section properties for existing curtain wall mullions.

Additional performance testing revealed several concerns with internal seals and gutters integral with the new curtain wall water management system. Modifications were made and re-tested at the laboratory and the mock-up passed each of the specified performance test standards, including air infiltration, static and dynamic water penetration, and condensation resistance.

While laboratory testing was on-going, site preparation was underway in Chicago. The contractor repeated the survey of the existing framing, this time through-out the building. This survey pointed to some out-of-tolerance conditions which required designing and detailing a supplemental anchorage connection in the event that the centerline of the new curtain wall unit did not align with the center of the existing curtain wall mullion. Site logistics raised additional concerns that had to be taken into consideration during the pre-planning process. A site perimeter security wall at the job site coupled with one way streets and a vaulted site paver system limited the installer's options for receiving and distributing materials. Given the tight physical constraints of the job site, a typical crane or forklift unload was not possible. The solution required a mobile piece of equipment capable of lifting a 13'-0" x 5'-0" x 6'-0" tall bunk weighing 4,000 pounds, to a height of six feet, all while squeezing through a 42" wide opening. An electric walk-behind pallet stacker proved to be the equipment of choice.

At the shop, individual frame units were fabricated as shown in Figure 10 and subsequently packaged in groups of seven on bunks made specifically for the project. The bunks were loaded onto flatbed trailers and tarped for storage. For ease of erecting the units at the job site, the units had to be loaded on edge and in reverse order. Bunks were unloaded and placed in the staging area according to the proper setting sequence.



Figure 10: Curtain wall unit shop fabrication.

Inclement weather was one of the biggest concerns for the Project Team. How do you remove the existing envelope of a building while assuring the occupants that the weather will not be an issue? We all know the difficulties associated with inclement weather on new construction projects, much less one that's filled with office furniture, carpeting, computers and maintaining its regular business hours.

To accomplish this, a combination of swing stages and mono rail systems were erected on the roof level over each of the 12 facade areas. Demolition and installation crews combined to remove and replace the existing covers, pressure plates, gaskets, window washing tracks and glass with a new unitized curtain wall system. Temporary walls were erected on the interior floors to protect the occupants and help reduce noise and wind. The old glass was first removed by workforce on suspended scaffold stages and passed through the now open wall to be carted and hauled out through the building.

Skinner lines were erected at each façade to help guide the new units during hoisting operations. The new units were lifted from the ground level using an electric hoist mounted to the mono rail. Once at the correct level, the units were disconnected from the skinner lines and "trolleyed" into position using the monorail system as shown in Figure 11 and Figure 12. Crews working off of swing stages fastened the new 5'-0" wide x 13'-0" tall units to the old framing system as shown in Figure 13 while interior crews helped guide and mate the upper portion of the units. Once all units in facade area were installed, it still was necessary for the contractor to wet-seal joints between units and at the perimeter of the curtain wall areas.



Figure 11: Using the monorail system to "trolley" curtain wall units into position.



Figure 12: Moving a new curtain wall unit into position.



Figure 13: Fastening a new curtain wall unit into existing aluminum mullion.

Prior to leaving for the day, crews filled all existing vacant glass opening of the facade with 1" thick insulated aluminum panels as shown in Figure 14. Glazing the temporary panels into the system's existing glass pockets proved to be a quick and lasting solution to the weather concerns.



Figure 14: Installing temporary panels at the end of a work day.

Conclusion

After years of deliberation and planning, in the fall of 2007, repairs commenced at the building. As envisioned, the over-clad and unitized panels rendered a quick repair method once actual installation began. The ease of installation allowed work to progress relatively unabated through the snowy Chicago winter of 2007-2008. By the end of March the work was completed as scheduled.



Role of Information Flow Management in the Design and Construction of Exterior Wall Systems

Andreas Phelps, P.E.,¹ and Michael Horman, Ph.D.²

Abstract

The design and construction of buildings requires an immense amount of information. Yet, little is really known of the information flow in projects. This qualitative analysis studies the information flows for the design and construction of exterior wall systems. By investigating these needs, it was found that a large portion of the information that existed within the team was not being used effectively. Additional information needs that did not already exist within the project team, needed to be provided by outside sources. The process for effectively incorporating information from inside and outside sources was heavily influenced by: 1) interpersonal relationships between team members; 2) understanding the information needs of the projects at the system and whole building levels; and 3) use of various mechanisms to effectively communicating problems and possible solutions. Using ethnographic research methods, this analysis provides a much deeper understanding of the conditions of projects concerning information flow and the performance of project teams on information aspects of a project. The results show the importance of information flow to completing design and construction activities with the implication that managers need to assign greater importance to effectively managing information flow on highly complex projects.

Introduction

The increasing prevalence of integrated design, sustainable design, new products and methods, and highly specialized systems has greatly increased the complexity of construction projects. Complex projects, such as hospitals and laboratory facilities, create extremely information-intensive and dynamic project environments. Project teams have to continuously evaluate new information and integrate it with old information to modify existing solutions, designs, and tasks or create new ones in order to accomplish their tasks. For any project, information flow plays an important role in project delivery. However, for complex projects, the effective management of information becomes critical. The processes and activities to identify, create, analyze, and disseminate information are essential for coordinated material and labor flows that enable a completed facility to be realized.

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Because the construction industry is socio-technical in nature, both social and technical processes need to be considered in understanding the behavior of project teams. This is especially true for understanding information flow which is even more nuanced. As projects become increasingly complex, a stronger emphasis on information systems will be required to effectively coordinate and deliver projects.

Related Literature

Information flow on building projects rely on understanding the processes and structures that occur within a project team. Regarding process, literature on lean process management from manufacturing (Womack and Jones 2003) has been adopted by the construction industry. (Howell 1999, Ballard and Howell 1994, Salem et al. 2006) Lean construction, coupled with techniques such as process mapping, provide a means to understand flows of materials, labor, and information and identify sources of waste and value.(Lapinski 2006) Other research, such as the work Austin et al (2002) have done in design structure matrices seeks to organize information flow in design. Building information modeling research also seeks to improve information flow in projects.(Yerrapathruni et al 2003) Unfortunately, many of these efforts overlook critical social factors that affect information flow.

In order to gain insight into the social issues affecting information flow in project teams, structural factors such as the organizational framework and group dynamics need to be explored. Recent research in virtual organizations and teams provides a useful context to understand project teams. Virtual organizations are defined as organizations that transcend legal identities, accounting systems, organizational charts, and physical assets. (Hedberg et al 1997) Child (2005) outlined the potential benefits of virtual organizations, but to realize these potential benefits requires a critical shift in management that focuses more on: 1) communicating a vision and strategy to guide and motivate the organization, 2) placing a strong focus on information processing and knowledge management, 3) emphasizing the coordination of others, and 4) constantly reinforcing the skills and cooperation among staff. As a result organizations will rely more heavily on managerial skills, such as communication, assessment, learning, and valuation.(Warner & Witzel 2004)

Interpersonal factors also play a role in information flow. Because project teams are made up of individuals from different parent companies and expertise backgrounds, each person tends to have idiosyncratic norms, values, time frames, and coding schemes. (March and Simon, 1958; Katz and Kahn, 1966) There is significant literature outlining the difficulties, inefficiencies, and hazards of transferring information across organizational interfaces.(Tushman 1977) High-performance teams and group development studies are also helpful in understanding group dynamics. (e.g. Katzenback and Smith 2003, Gersick 1991, Chang et al 2003) Research on teams in the organizational behavior field has provided insight into the role of collaboration and team social integration (Harrison et al. 2002), the effects that team expertise diversity and collective identification have on team performance (Van Der Vegt & Bunderson 2005), and the role of trust (Dirks and Ferrin 2001).

Conceptual Framework

The delivery of complex projects relies heavily on the ability of the team to effectively identify, analyze, and disseminate information into the project. This includes tacit and explicit information that already exists within the project team and new information that exists in the industry. Because of the socio-technical nature of the construction industry, both social and technical processes are major determinants of information flow effectiveness (see Figure 1). Many of the factors affecting the team's ability to identify and use existing and new information depends on interpersonal factors such as trust and respect among team members, establishment of common goals, individual knowledge and skills, and understanding of project information needs and possible information sources. There are also technical factors such as introducing the appropriate scopes of information at the optimal time, using certain techniques or technologies to pull information, and creating an information framework to assist in managing and prioritizing the large amounts of information present on complex project.

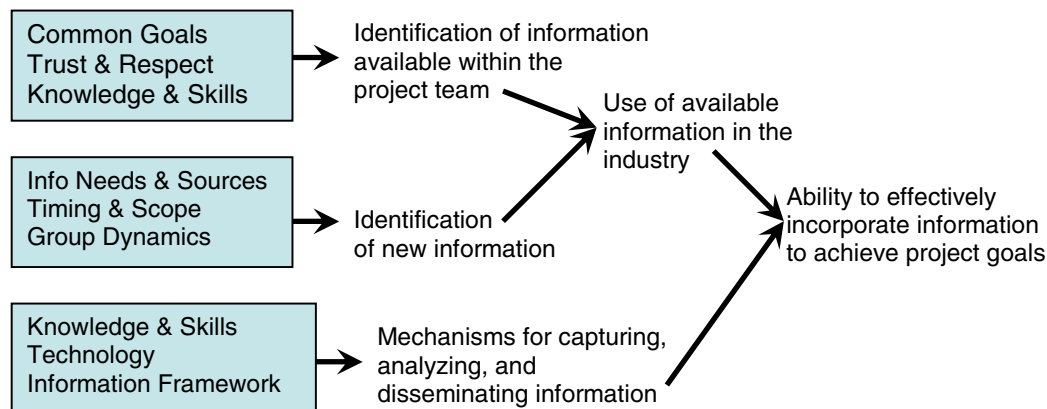


Figure 1: Conceptual Framework for Factors Affecting Information Use

Methodology

The authors' interest in complex projects stems from the opportunities and challenges of incorporating sustainable design features into healthcare facilities. The United States is currently going through the biggest health care construction boom in the last 40 years giving project teams opportunities to incorporate innovative design features that for better building and occupant performance. The benefits of design and construction innovations have been illustrated evidence-based design research from the Center for Health Design and pilot projects for the Green Guide for Health Care. Because of their complexity, one of the most significant problems in implementing innovations in healthcare projects is that additional information is difficult to manage and ends up being ignored, misused, or compromised.

Healthcare facilities are some of the most challenging and information intensive projects to deliver due to highly specialized user needs and the critical effect that design and construction have on long-term facility and staff performance. (Ulrich and Quan 2004) For these reasons, effective information management is essential to the success of these complex projects. The purpose of this research is to identify the

social and technical factors that affect information flow. In order to capture and analyze the complex interdependencies of information flow processes, ethnographic methods were used. Ethnography is best suited understanding complex settings where technical, organizational, and social factors intersect and has provided useful insights in the organizational sciences, social sciences, information science, as well as construction. (e.g. Mahalingam and Levitt 2004, Seymour and Rooke 2001, Barker 1993, Sutton 1991, and Reddy et al. 2001)

Data Collection

In order to understand the information flow structures on complex projects, this research consisted of a long-term field study of a project team working on major building projects at a medical center campus. The total cost of the projects was over \$350 million and included the new construction of a two major clinical and research facilities as well as supporting facilities. Over a two year period, researchers collected data through observations of meetings and other work processes, informal and semi-structured interviews, and analysis of project documents. The data collection focused on understanding the structures and processes for information flow and identifying factors that affect the design and construction of four interrelated scope areas. One scope area, the exterior wall systems, is the focus of this paper.

Support for Concepts

In any project, there are three main types of information needs on any project: Aesthetics, constructability, and performance. Aesthetic information needs include the qualitative and quantitative spatial requirements of the building, the materials, finishes, spatial ordering, and the overall architectural intent of the building and the spaces inside the building. Constructability and performance information need to be considered at both the system-level and the building-level. System-level constructability information includes their means and methods for sequencing work, coordination within their work crews, access and logistical issues, cost, and schedule. Building-level constructability includes information regarding spatial and temporal coordination between trades, clarifying responsibilities and sequencing required at interface details between systems, and managing the overall cost and schedule of the project. System-level performance information includes air and water tightness of the system, thermal performance, material compatibility, durability, maintenance, and estimated service life. Building-level performance information includes the energy efficiency of the entire building, sustainability issues, synergies and conflicts between various systems, and how well the building meets the overall project goals.

The structure of the project team is one major factor that determines how effectively information will flow on a project. Understanding how various information needs will be met is just as critical as determining the needs in the first place. In some cases, traditional project roles align very well with information needs. For example, the architect is primarily responsible for aesthetics, the subcontractor for system-level constructability, and the general contractor for building-level constructability. Performance information is often missing or split among various roles. Conflict may arise when various individuals on the project team have overlapping knowledge

regarding certain information areas. These conflicts result in differing opinions and can either compromise the team's ability to work together effectively or facilitate constructive collaboration and a better common understanding.

In the case of project teams, there is information that exists within the project team and information that exists outside the team but within the industry. Information needs that cannot be satisfied within the immediate project team either need to be addressed by the team's use of new information or by acquiring additional team members (e.g. consultants). The sections to follow outline observations from the field study regarding the use of information available within the team, outside the team, and the mechanisms used to pull, capture, and disseminate information.

Using Information Available within the Team

Information that already exists within the team consists of the tacit knowledge that each of the team members possesses due to their formal training and experience. There are several factors that affect the team's ability to make use of the information that already exists within the project team. Interaction between team members can be synergistic in the case of high-performance teams. Conversely, interactions can be detrimental in the case of poorly performing teams and reduce the effectiveness of individual contributions. In their typology of groups and teams, Katzenbach & Smith (2003) outline factors that affect a team performance. They found that teams with no interest in shaping common purpose or performance goals have the weakest performance, but performance improves as teams clarify their purpose, goals, and common work approaches and begin to hold themselves mutually accountable for project outcomes.

Common project goals can be created through several means. In some cases, a common purpose can result from a novel project goal. These unique and often challenging goals, such as delivering a LEED Platinum hospital, can serve as a unifier among team members by giving them a challenge that requires they work together to be successfully achieve these goals. In the absence of a novel goal, interpersonal skills within the team become even more critical. This can take the form of a single individual "integrator" with managerial and interpersonal expertise that mediates between individuals to help them understand each other's concerns, expertise, and interactions. The integrator also helps develop a sense of team and common purpose which results in a trusting, respectful, and open project team environment. (Srivastava et al 2006) The integration function can alternatively be provided by the entire team through their complimentary skill sets and mutual accountability for the overall project outcomes. Several studies have shown that trust within teams results in better communication, less conflict, better team and individual performance, satisfaction, acceptance of decisions, and higher perceived accuracy of information. (Dirks and Ferrin 2001)

Time pressures can result in development of common goals. Group development research has shown that groups tend to work under one mode until they become cognizant of the approaching deadline and then they shift into a second mode. (Gersick 1991) The first mode is characterized by dependency and inclusion

among members as they try to define the situation and counterdependency and fight as they clarify their roles and try to reconcile differing values. The second mode is characterized by trust that results from resolution of the previous conflict and more open communication as well as optimal work effectiveness stemming from shared goals due to the awareness of the deadline.(Chang et al 2003) Although approaching deadlines result in more cohesive team behavior, this “putting out fires” approach greatly reduces the teams ability to make the most effective use of all the available information and results in sub-optimal project outcomes.

In the field study, there were two situations that resulted in a shift in team dynamics. One situation was that the project manager (PM) for the owner changed after the bidding phase of the project. The first PM was very knowledgeable and progressive in terms of industry trends, construction research, and had both significant architectural and construction management experience. He had several innovative initiatives that he was trying to inject into the project and was also very diligent about controlling the cost of the project. In some cases, however, his seemingly beneficial knowledge and skills actually ended up being detrimental to the project team because of a failure to develop trust and respect and common goals among the project team. During design review meetings, there were priority conflicts between different members. Architectural details that the architect felt were essential to the architectural intent were perceived by the PM as costly, potentially problematic in terms of performance, and not critical enough to the architectural intent to warrant the risk. In this case, the PM’s architectural background and experience caused him to question the architect’s judgment which gradually resulted in lack of trust and respect within the team. Eventually team members would not bring up their concerns, offer suggestions, or even engage in discussion with each other regarding certain issues because they thought that their comments would be criticized or their competence questioned. In addition, the PM’s pursuit of personal goals, although potentially beneficial to the project, further accentuated the defensiveness of other because they were never asked for their “buy in” and felt that the goals were being imposed upon them at the expense of their own personal goals. Along similar lines, the architects were not very receptive to the owner’s in-house reviews of the drawings because they felt that these were further efforts to question their competence. This protective and defensive mentality resulted in a withholding of information and a reduced likelihood that individuals would acknowledge missing information which could have improved the outcome of the project, especially at the early stages of design.

Due in part to the information flow issues encountered in design, the initial bids for the building envelope systems were significantly over the anticipated budget. As a result, these scopes were re-bid and the owner switched the contract to a design-assist relationship between the subcontractors and the architect. Around this time, there was also a switch in the PM. The new PM, while seasoned in construction, had much less experience with healthcare project. He also had much more of an interest in improving the team dynamics. He was able to create a more open and trusting team environment through techniques such as asking team members what their specific concerns were, clarifying what his understanding of the critical issues were, asking team members for their recommendations, and explaining his own concerns. These

techniques allowed team members to gain a better understanding of the issues that were important to the others, prioritize the major issues, and feel that their input was valued and respected. The PM's lack of previous knowledge regarding these types of project allowed him focus on the major issues while leaving the details to the team members that were best suited to deal with them. This switch in leadership also coincided with more frequent meetings between the subcontractors and the designers due to the need to resolve critical issues. This further contributed to the development of common goals, common understanding, and cohesion within the team.

Using New Information

Improved team dynamics resulted in more effective use of the information already available within the team, but on complex projects there is often significant information that is missing and needs to be provided by outside sources. The ability of a project team to identify, analyze, and disseminate new information from outside sources depends on the team dynamics and having knowledge of what outside information sources exist and how they can address team information needs.

As mentioned earlier, there are several information needs within a project team. In high-performance teams, trust and open communication makes it easier to identify the knowledge and skills available within the team as well as those that are missing. In poorly performing teams, the closed and defensive mentality makes these distinctions more difficult. In addition, teams need members with broader, more holistic perspective of the project and the trends in the industry to identify interdisciplinary and building-level information needs and potential information sources. Without this perspective, individuals on project teams tend to limit their discussions to information that they already possess.

Once these needs for additional information are identified, they need to be matched to available sources outside the team, such as consultants, research, or references. The effectiveness of incorporating additional information depends on the timing and scope of the new information being provided and understanding the interpersonal team dynamics. Because the ability of individuals on the team to absorb new information depends on their ability to apply that information to their existing knowledge base (Cohen and Levinthal 1990), determining when and how to address certain information needs is just as critical understanding what those needs are. In terms of timing, information provided too early results in unnecessary time spent discussing and evaluating because the team does not have the proper context with which to apply that information. Conversely, information provided too late creates a lot of rework, opportunities for schedule and cost savings have diminished, and often there are changes that need to be made to many other parts of the project that have already been fully developed and coordinated. (Pulaski & Horman 2005) Providing the appropriate level of details is also a delicate balance. General, building-level information should be considered early on in the process to develop basis for design documents and establish goals and priorities for the project, whereas detailed, system-level information should come out of coordinated efforts later in the delivery process. In addition, detailed information should be prioritized based on complexity and importance to the project. Complex details that are essential to the project usually

require a lot of information and should be addressed at the earliest opportunity when the most options are available.

Another factor that affects the ability of new information to be brought into the team is the existing group dynamic. Depending on an individual's relationship within the team, their efforts to introduce new information may be viewed as genuine or as a means to promote their agenda or discredit others. In the case of consultants joining the team, there is a process of socialization that new members need to go through when joining a team in order to understand the project history, team dynamics, and critical issues. During this process, the rest of the team develops a sense for what the consultant knows, how that information affects the project and their specific scope, and whether there is a sense of mutual trust and respect. (Tsai and Ghosal 1998) Acceptance of new information depends on who in the existing team introduces that information or how new team members fit within the existing team dynamic.

In the field study, there were additional complexities due to the use of state funds for a percentage of the project. The inclusion of state funds required that the design be based on non-proprietary systems. As a result, much of the basic system-level constructability and performance information that is usually provided to the architect during design by product manufacturers was missing. To address the missing information needs, both the PM and the architect had outside consultants review the exterior cladding drawings. Because of the tensions between the PM and architect, neither set of the review comments was accepted by the other and incorporated into the project. During the early stages of design-assist, the owner hired a consultant to review the drawings and develop a performance testing protocol for the project. Because of the consulting firm's strong reputation in performance testing, the team accepted their recommendation regarding the performance testing protocol but their comments on the design still did not result in any changes. During the design-assist process, the general contractor hired a consultant to review the drawings, provide comments, and help facilitate the coordination process.

The information from the consultant that the general contractor hired was integrated into the project much more successfully. There are several potential reasons for this. First, the consultant was introduced by the general contractor, who was a neutral, non-biased party in the ongoing tensions between the owner's PM and the architect. Second, the consultant was regularly involved with coordination efforts and problem solving and was viewed as an integral part of the team. There were also several techniques that the consultant used to develop trust between members of the team such as explaining his technical background, acknowledging important issues to find the best all-around solutions, and using language and level of detail that was appropriate to each team member's expertise. Although the consultant was very effective in communicating information, some of the information overlapped and differed with information provided by the other performance testing consultant and subcontractors and resulted in some minor conflicts and defensiveness, respectively.

Another factor contributing to the effectiveness of the general contractor's consultant was the timing of his involvement. Earlier consultant comments regarding missing complex details were not addressed by the architects because they did not have a

sufficient information to begin to address these issues. Early in design, there was a tendency for individuals to avoid issues they were not sure about by shifting the focus to something that they did understand, underplaying its importance, or discouraging others from discussing it by becoming defensive. However, after several design-assist meetings with the subcontractors, the architects had a better understanding of the system and could begin to address the complicated details. Unfortunately, the majority of the design-assist process was spent working out a few complex details and left little time for developing innovative means of improving upon schedule, quality, or cost. Had constructability and performance information been provided earlier in the process, the team could have freely explored solutions to these complex details before the simpler details of the surrounding systems were firmly defined. By finalizing all the typical details before developing solutions for the more complex, important details, the team constrained themselves to options that either compromise the architectural intent or will require significant redesign of adjacent systems.

Mechanisms for Communication

In addition to tacit and explicit information that exists within a project team or within the industry, there is also information that can be created within a project team based on the knowledge and skills available. The ability to create new knowledge depends largely on the mechanisms that are used to “pull” information from individuals, order the information for analysis, and disseminate the information to others on the project team. These mechanisms can come in the form of information objects, facilitators, or information frameworks.

The use of certain objects in design and coordination processes tends to pull different types of information. Traditional two-dimensional drawings tend to perpetuate the problem of avoiding what is unknown by only acting as a depository for known information. More complex issues, such as system interfaces and intersections of details, are only addressed when more sophisticated objects or the actual construction process require additional information. These more sophisticated objects include isometrics, sequence sketches, coordinated shop drawings between multiple trades, energy modeling, condensation analysis, daylighting analysis, and visual and performance mock-ups. Different objects change the nature of coordination discussions. Isometrics of detail intersections show how several details come together and instantly outline the conflicts or missing information. Similarly, 3D sequence sketches identify conflicts in trade coordination, schedule issues, and lines of responsibility. Visual and performance mock-ups provide concrete information for resolving speculative discussions of how a system will perform or look. The challenge is to match the cost and complexity of the object with the importance of gaining the associated information.

Another useful mechanism is having team members that can act as facilitators during design and coordination. Facilitators need to have balanced relationships with the other team members, gain their respect and trust, and have the appropriate skills to pull the needed information. They need to ask the right questions of various team members, clarify information, serve as the collective memory for the team, and prioritize information in the context of project goals. Facilitators also need to have a

broad knowledge of all the scope areas so that they can speak knowledgeably to team members with a lot of expertise in one area and disseminate that information in terms that make sense and are relevant to other areas. (Tushman and Scanlan 1981) When needed, the facilitator also needs to take measures to improve working relationships between team members.

In lieu of or in addition to a skilled facilitator, an information framework can be used to formally capture the goals of the project, the concerns of each team member, and to identify critical issues and responsibilities. Goals serve as a basis for evaluating decisions and creating common focus. Listing team member's concerns aids in understanding their point of view and values. It also provides a list of issues that need to be addressed in the development of solutions. By identifying critical issues, the team can prioritize them, set forth a process for addressing them, and can secure commitments from team members regarding who is responsible for what.

In the field study, various mechanisms were used throughout the different stages of the project. Early on, process maps were used to outline the processes and goals for each phase of the project delivery. In addition, at the beginning of coordination meetings a facilitator would go around the room and ask everyone what their major concerns were and post the list on the wall during the meeting. Use of isometrics and mock-ups helped to put to rest contentious issues based on differing opinions by providing concrete objects from which to make educated decisions. One of the consultants commented that "the isometrics help smoke out the complexity." At various stages, facilitators helped the team understand some of the issues in a more holistic sense and improved relationships within the team by empowering individuals and creating a better collective understanding.

Conclusion and Future Research

As construction project become increasingly complex, the role of information management will become essential to the success of projects. Within the diversity of expertise already involved with projects, there is already substantial tacit and explicit information available to project teams. This is supplemented by a wealth of information in our industry and affiliated research institutions. The ability of project teams to effectively make use of that information depends heavily on socio-technical factors within the project team. This paper begins to develop a framework for understanding the effect of social and technical processes on information flow. Additional research is needed to support the various constructs, specifically in the areas of: 1) how various information objects pull information, 2) group dynamics in teams with dual identities (project and parent firm), 3) the role of trust, respect, and team cohesion on information behavior, and 4) whether traditional roles and delivery methods are adequate to address information needs on complex projects.

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HIGH-PERFORMANCE CEMENT PLASTER (STUCCO) SYSTEMS

INTRODUCTION

Traditional Stucco Cladding

Stucco cladding has been a common, cost effective cladding design for light commercial and residential construction for many years. The first exterior stucco systems on wood-framed buildings utilized wood lath and had no sheathing, water-resistive barriers (WRBs), or thermal insulation. Wood components were made of solid timbers, often from old-growth forests, and were generally less moisture-sensitive than the wood products being used in modern construction.

The traditional system has evolved significantly over the last century. Stucco is now often installed over both wood-framed and light-gage-steel-framed buildings. Wood lath has been replaced by sheathing and metal lath. Building codes now require WRBs and insulation; some codes (Canadian National Building Code and Commonwealth of Massachusetts' Building Code) even require continuous air barrier systems to prevent air movement across exterior walls.

Modern Trends in Stucco Walls

Although stucco has performed well historically, changes in wall construction have made new buildings more susceptible to damage in the event of a leak. In the past, small leaks may have gone unnoticed because walls were open to natural ventilation and diffusion; stucco assemblies and wall cavities were more porous and breathable so leaks could evaporate before they caused damage. Moreover, the popularization of moisture-sensitive, cellulose-based wall components such as paper-faced gypsum wallboard and oriented-strand board (OSB) has made modern walls more susceptible to water damage.

Today, insulation within wall cavities, dense wall sheathings, vapor-impermeable paints, and vapor-, water- and air-barriers slow down evaporation from wall cavities by limiting bulk air movement and vapor diffusion. Although each component serves an important, unique purpose (Gabby and Cammalleri 2006), the assembly of these components into modern wall systems slows drying in the event of a leak. Retarding

evaporation causes water to remain in contact with the moisture-sensitive materials longer, thereby accelerating deterioration and degrading wall cavities.

Many present-day designs include cellulose-based sheathing of OSB, paper-faced gypsum board, and plywood, which all absorb moisture. If they stay moist for extended periods of time and mold spores are present (as they always are), mold will grow.

The slowed evaporation and increased sensitivity to the presence of moisture work synergistically toward damaging building walls. Composite cellulosic materials such as OSB appear to absorb more water and deteriorate more rapidly than natural wood. The slow evaporation rate increases the probability that mold spores will germinate and grow. Slow evaporation rates can also exacerbate the effects of construction moisture; slabs dry more slowly, and precipitation during construction is trapped when insulation, sheathing, finishes and barriers are installed.

Since modern walls are more sensitive to moisture, exterior stucco systems must provide more protection and manage moisture more effectively to prevent damage. Though industry standards, building codes, and common practices already require watertight stucco assemblies, they do not address the details of how to do so. The following discussion presents common problems that continue to plague stucco systems and the industry-standard solutions currently used to address those problems. And, since we find that these solutions are often ineffective or insufficient, we will present high-performance solutions that are beginning to demonstrate their effectiveness.

COMMON STUCCO PROBLEMS

Weather-Resistant Barrier

Introduction

The weather-resistant barrier (WRB) system including flashings is the final moisture barrier in a stucco system and has the most potential for making the system “high performance.” Any water that bypasses the cementitious stucco, whether through cracks, capillaries, accessories, trim, penetrations, terminations or any other sources, should be halted by the WRB system and then directed out of the system. WRB selection and integration with other various wall components is critical to overall system performance.

The sheathing is the foundation of high performance; it provides support for the WRB and flashings, helps to maintain the stucco’s thickness by providing solid, consistent resistance during stucco application, and reduces stresses in the stucco by transferring loads (punching and shear) to the framing.

Problems

- Designers, installers and owners often underestimate the amount of moisture that migrates through or around the cementitious stucco cladding; their underestimation leads to inadequate attention to detail during design and installation. WRBs are too-often left with open gaps, reverse-shingled to the flow of water, and damaged during construction.
- Adjacent trades, such as mechanical, electrical and plumbing installers often do not realize the implications of penetrating waterproofing elements, and they do not seal or flash penetrations adequately to prevent leaks.
- Designers do not prioritize drainage within the system, so water collects within the system and follows the path of least resistance - through defects in the WRB and into the building.
- Open framing often does not provide the needed backing support for the stucco and WRB resulting in the stucco being too thin over the studs and too thick in between studs.
- Moisture-sensitive sheathings (OSB and paper-faced products) deteriorate and support mold growth in the event of a leak.
- Installation efficiency and material cost typically are the driving factors for selecting sheathing products. In this age of redesign, design assist, requests for information, substitutions and value engineering, the owners, contractors, vendors and subcontractors all weigh-in on the sheathing-product selection after the designs are finalized. Designers should be more concerned with stucco and WRB performance, moisture-sensitivity of system components, local climates and the lessons learned from failure investigations.
- Building-code authorities seem to regard sheathing strictly as a fire-resistive and/or structural element, and not as part of the exterior cladding system (stucco or otherwise); this is evident by the fact that many single-story or two-story structures need no sheathing to satisfy code requirements in low-hazard seismic zones. Other structures may only need partial sheathing at building corners per code. Moreover, sheathing and stucco designs are poorly integrated because the project's structural engineer designs the sheathing, and the architect designs the stucco.

Industry-Standard Solutions

- Designers often show typical flashing details, but largely rely on contractors to detail flashings at difficult building geometries as well as system terminations and transitions.
- Adjacent trades are left to fend for themselves when penetrating the WRB. Plumbers and electricians are asked to be stucco-flashing experts. In addition, in many instances that we see, the general contractor does not coordinate the efforts of the subcontractors.

- The stucco industry generally follows the code minimums for WRBs: one layer of building paper or approved equal over open-stud framing and gypsum sheathing, and two layers over wood-based sheathing.

High-Performance Solutions

- Use waterproofing design practices similar to those for roofing, plaza waterproofing, and below-grade waterproofing for WRBs: provide provisions for effective drainage, protect the membrane (WRB) with sacrificial layers and from adjacent trades, and seal and flash penetrations to shed water toward the exterior.
- Install a drainage plane product under the lath or between the two layers of WRB to provide an effective and efficient system that allows water to positively drain or weep out of the system.
- Educate designers on the importance of drainage within the wall cavity and of lapping the waterproofing elements relative to the gravitational flow of water. When designing stucco walls, consider the additional work and risk associated with each penetration and change in wall plane. Provide reliable detailing at penetrations and odd geometries to convey design intent to the contractors. Avoid geometries where the WRB bends through sharp angles and has a tendency to span across corners.
- Install additional horizontal through-wall flashings with weeping capabilities to provide an exit path for moisture at regular intervals (i.e. at each floor line) and at obstructions such as window and door heads, as opposed to the typical practice of weeping multiple stories of stucco only once at the foundation.
- Perform laboratory testing and field trials to determine the viability of cold- and hot-fluid-applied waterproofing systems as the WRB under stucco. Testing should be similar to testing used in other areas of the waterproofing industries.
- On the practical end of high performance, use compatible sealant to seal all fasteners and any tears or holes that penetrate WRB, or at least seal shiners, over-driven fasteners, and fasteners on horizontal and sloped surfaces.
- Improve on code minimums by considering the sheathing part of the cladding system. Use sheathing beneath all areas receiving stucco to increase the performance of the system by continuously supporting the WRB and cementitious stucco.

Cracking

Introduction

Stucco can crack for a variety of reasons, including building movement, natural drying shrinkage, thermal movement, stress concentrations at dimensional inconsistencies, and poor mixing. Cracking results when the tensile stresses in the stucco exceed the tensile strength. Stresses can be transferred from the structure to the stucco, or they can originate within the stucco itself. The best solutions will limit cracking by either improving

dimensional stability of the stucco or the building, increasing the stucco's tensile strength, or limiting stress transfer between the structure and the stucco.

Problem

Cracked stucco is much more than an aesthetic problem. Stucco with an excessive amount of cracking, or cracks of excessive width, allow larger quantities of water to come into contact with the WRB. The water can then find its way through fastener penetrations, laps or other WRB defects.

Industry-Standard Solutions

- For many years, various types of fiber additives have been used to strengthen stucco. The fibers, whether they be modern polypropylene, human hair or animal fur have more tensile strength than cement, so embedding them in the stucco helps to limit cracking and crack widths by increasing the tensile strength of the stucco in either the plastic state or the cured state or both.
- As with other cementitious materials like concrete, stucco should have control joints at regular intervals to allow for shrinkage and thermal movement and to control crack locations. The areas surrounded by control joints (stucco panels) should be limited to a maximum of 144 sq ft. Theoretically, the most effective panels are square, but the industry standard allows rectangles with aspect ratios up to 2-1/2:1 and lengths up to 18 feet (ASTM C1063).
- Expansion joints at substrate discontinuities such as floor lines allow the building to shift without loading the stucco. The most effective stucco expansion joints are made of two pieces, which are fastened separately to either side of an anticipated movement plane, such as a slip track in metal framing or a building expansion joint.
- Providing appropriate gaps at the perimeter of wood-based sheathing products allows them to swell and contract without interference when cycling through wet and dry periods, which would cause buckling and lead to cracking (ACI 524R).
- Wetting substrates prior to stucco installation can help prevent rapid, premature dehydration of the cement matrix during its initial cure by reducing suction of water out of the stucco mix. When stucco is installed on dry substrates, the moisture in the mix may be quickly desorbed into the substrate, and the cement cannot cure effectively without the necessary water content. Cement hydration during cure is necessary to produce a high-strength stucco that resists cracking.
- Moist-curing the various stucco coats (i.e. scratch, brown and finish) also helps maintain hydration during the cement-curing process. Although many standards do not prescribe a time period for moist-curing, 48 hours is the industry standard (Logue 2007). An occasional fog-spray during the moist-cure period is usually sufficient to maintain hydration, but high wind and hot sun conditions require protecting the

stucco with canvas or plastic sheet materials to obtain an effective cure.

High-Performance Solutions

- Polymeric admixtures (e.g. acrylic, polyurethane, acrylic-polyurethane copolymer) can increase the bond strength, tensile strength and flexibility of cured stucco.
- Kiln dried (KD 19) wood studs reduce stucco cracking by minimizing the structure's shrinkage movement after the stucco is installed. Drying the wood brings it closer to its final equilibrium condition thereby reducing the amount of potential shrinkage.
- 100% acrylic finish coats, such as those used on Exterior Insulating Finish Systems (EIFS) are more flexible than cementitious finishes, and are therefore more effective at bridging cracks.
- Many lime-substitute admixtures improve workability and increase the final stucco product's strength. Moreover, improved workability facilitates control of stucco thickness, which helps prevent stress concentrations and subsequent cracking at thin areas. The problem with such proprietary additives, however, is that their content is neither regulated nor disclosed.

Premixed Stucco

Introduction

Traditionally, workers mixed stucco components, including sand, cement, plasticizer (lime) and additives on site within the mixing ratio limits prescribed by building codes and standards. The standard proportions are based on proven performance history and represent consensus among industry organizations. Today's manufacturers produce premixed stucco that requires the addition of only water. These products include sand and cement, and they often have admixtures such as fiber, lime, and air-entraining additives pre-mixed in each bag. Some products do not have manufacturer's literature that clearly indicates whether they are in conformance with ASTM C 926 (Portland Cement-Based Plaster), ASTM C 1328 (Plastic Stucco Cement) or any other industry standard. The industry standards do not specifically address such premixed stucco products.

Problems

Premixed stucco products are problematic because the installer may not be familiar with each specific product, mixing time, standing time, water requirements, and climate effects that may affect performance. The manufacturers' technical ability and performance track record are important and a part of the final cladding system's performance. The lack of published technical information (typical of other exterior cladding systems, e.g. metal panels, precast concrete, glazed aluminum windows, and curtain walls) inhibits designers from selecting premixed products to work under distinct and varying project conditions.

Industry-Standard Solution

The current standard solution involves shifting design responsibilities onto the contractor by assigning quality control in the scope of work; if the installer suggests a change to the mix or products, they will be held responsible for adverse consequences. In essence, designers give up control of the stucco-material selection to the contractors and manufacturers' salespeople. This works in many cases, but sometimes unproven products and/or mixes are used that lead to problems. Building designers should be proactive rather than reactive and specify materials with a proven performance history and require the contractor to provide proof of successful performance when alternate products are proposed.

High-Performance Solutions

- Field-testing is a common and effective quality-control practice in the construction industry, and we suggest that physical testing such as prism testing be applied to the stucco mixes in the field. Industry organizations have developed standard test methods for concrete and mortar to ensure quality and verify appropriate physical characteristics. The same or similar tests should extend into the stucco practice for the same reasons.
- As is done with other exterior-envelope-waterproofing systems, designers should require onsite visits, printed technical product data, and installer certifications from the manufacturers of premixed stucco products. Manufacturers should be compelled to train installers, provide important technical information, and share responsibility for ensuring appropriate product selection.
- Have installers submit engineered mix designs similar to those required for concrete.

Delivery (Pumping)

Introduction

The days of small, hand-mixed and hand-carried stucco batches have almost disappeared. Pumping equipment is the industry norm for delivering stucco from the ground to the walls on commercial projects. Installers mix wetter and more-plastic stucco to reduce viscosity and drag in pumps, hoses and spray nozzles. The physical properties and limits of stucco are governed by the same restrictions as concrete applications, but building codes do not specifically address the pumping restrictions for stucco as they do for concrete.

As an industry leader in cement properties, American Concrete Institute (ACI) has analyzed and reported on the effects of mechanical pumping on cement-based materials. ACI has helped implement codes and procedures to ensure quality in concrete installations. Similar standards should be developed in the stucco industry.

Problems

- Field installers modify mix designs (i.e. cement-to-water ratio, air-entraining and plasticizing additives) from the code, industry-standard and manufacturer requirements as they see fit to produce a pumpable mix. Mix proportions and properties vary randomly from day to day on the same project. The lack of standardized pump mixes leads to unreliable physical properties in the finished product.
- Contractors rarely monitor the pumping or perform batch testing to check the pumped stucco's quality, so the final product's performance can be unpredictable.

Industry-Standard Solution

As stated above in the "Premixed Stucco" section, the current standard solution involves shifting design responsibilities onto the contractor by assigning quality control in the scope of work. In contrast, the concrete industry has built-in checks and balances to ensure quality by third-party inspection and testing. When varying mixes lead to failure of stucco systems (i.e. cracking, stucco falling off walls, unacceptable appearance), the project often ends up before a judge. Immense amounts of human effort and capital can be wasted on litigation and/or repairs. Building designs should be proactive rather than reactive to minimize problems.

High-Performance Solutions

- Require mix submittals showing all the mixing ratios and products, as well as the method the contractor will use to ensure consistent batch mixes daily.
- Prior to commencement of installation, require separate, different mix-ratio submittals for pumping and hand delivery systems.
- Require submission of test reports that demonstrate proven performance history of the submitted mix ratios.
- Regulate, monitor and customize pump distances based on the mix's performance history.
- Test the mix periodically to verify its composition, and coordinate mix proportions with project conditions such as pumping and climate.

SUMMARY

To obtain stucco that performs its intended function, go beyond the industry standard by implementing high-performance solutions.

- Follow the lead of other industries by working to standardize quality-control test procedures for the various stucco components.
- When designing the WRB, remember that it is essentially a waterproofing membrane. Pay close attention to sealing and flashing around penetrations, terminations, odd geometries, and transitions to other systems. Provide continuous solid sheathing to support the WRB, and include provisions that drain water out of the system.
- Stucco cracks when tensile stresses overcome tensile strength. Reduce stresses in the stucco by preventing load transfer from the

structure and maintaining hydration during cure. Increase the stucco's tensile strength with high-performance additives such as lime substitutes, polymer treatments, and fibers.

- Avoid using premixed stucco products unless you know what's in them, how they've performed in the past, and who to contact at the manufacturer's office in case you have questions or problems. Even then, implement testing procedures to ensure that the selected product is appropriate for the particular project.
- Gather information about the mix that the installer intends to pump at each project. Work in the industry to standardize pumpable mix designs, and test the mix onsite to check physical properties.

The costs of implementing our suggestions will seem small when compared to the value gained and liabilities avoided by improving your project's stucco system.

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Experimental Evaluation of the Effect of Two Different Types of Drywall Joint Compound on Shear Capacity of Wood-frame Walls

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Abstract

This paper summarizes the findings of a pilot study to determine whether the type of drywall joint compound will influence the shear strength of wood-frame stud walls sheathed with drywall. In this study, five 8 ft x 8 ft specimens were tested under in-plane cyclic racking loading following CUREE loading protocol for light-frame wall systems. Three specimens were finished using non-cement based joint compound and the other two using cement based joint compound. Based on the limited number of specimens tested, the results show that the use of cement based joint compound on drywall joints as in the specimens tested will result in higher wall shear capacity compared to similar specimens finished with conventional non-cement based joint compound. The failure modes for the specimens finished with the two different types of compound were different. The result of the study is particularly important for high seismic regions where interior stud walls in residential construction effectively take part in seismic resistance even though wood shear walls are normally used on exterior walls.

INTRODUCTION

In residential construction, wood stud walls with drywall on both sides are normally used as interior walls. Representative studies on seismic response of wood stud wall systems include Wolfe (1983), Lam et al. (1997), Dinehart and Shenton (1998), and Dolan and Heine (1998), McMullin and Derrick (2002), and Bersofsky (2004). If used as exterior shear walls (to resist wind and seismic loads), the wood-frame is normally sheathed with OSB or plywood on the exterior and drywall on the interior. In either application, drywall joint compound is used with tape (paper or mesh) to cover the drywall joints. Although interior walls are not considered shear walls, the effective in-plane shear capacity of these walls help the building perform better under wind and earthquake induced lateral loads.

In a recent pilot study at Penn State University (Memari et al. 2008), the effect of joint compound on the in-plane shear resistance of wood stud and steel stud walls was evaluated. Wall specimens with 8 ft by 8 ft dimensions were tested with and without joint finish to determine the effect of joint compound on shear strength of the wall panel. The test results showed that for specimens without joint finish, the drywall sheets slide past each other at the joint, while the specimen with joint finish would have much smaller sliding due to joint compound effect. The conclusion of the in-plane shear test was that

application of joint compound not only reduces sliding of the drywall panels, it also increases the in-plane shear capacity of the light-frame. On the basis of such a result, it was concluded that the use of drywall joint compound can enhance the shear strength of light-frame walls, and that the stronger the joint compound material is, the larger will be its contribution to shear strength. In particular, a joint compound with cement in its mix could have a larger contribution compared to conventional joint compounds without any cement in their mixes. To prove such a concept, it was decided to test wood-frame wall specimens and compare the difference in shear capacity of specimens finished with cement based joint compound type Rapid Set OnePass manufactured by CTS Cement Manufacturing Company and those finished with a conventional (non-cement based) joint compound such as that manufactured by UGL available at local hardware stores.

EXPERIMENTAL PROGRAM

For this pilot study, five 8 ft x 8 ft specimens of wood stud wall panels were constructed. Two specimens were finished with OnePass and three with UGL joint compound. Because Rapid Set OnePass joint compound is normally applied in one coat, it was of interest to evaluate the performance of a specimen with one coat after 24 hours of cure time (RPDST24HR1C) and another specimen with one coat after seven days of curing (RPDST7D1C). For the UGL compound, although it is normally applied in two coats, for comparison purposes, it was decided to evaluate the performance of three specimens as follows: one specimen finished with one coat and cured for 24 hours (UGL24HR1C), one specimen finished with one coat but cured for seven days (UGL7D1C), and finally one specimen finished with two coats and cured for seven days (UGL7D2C).

The test setup shown in Figure 1 consists of a horizontal floor steel frame designed to load test walls laid flat on the test frame and supported on rollers. The hold-down mechanism for this study is based on ASTM E 72 (ASTM 1998), but the loading protocol followed CUREE-Caltech wood-frame protocol (Krawinkler et al. 2002). ASTM E 72 requires the use of a tie-rod as hold-down mechanism to prevent uplift or overturning of the specimen during the test. The load was applied to the specimen using a pair of 20 kip capacity bottleneck jacks. Each loading jack had a stroke capacity of 3 1/2" and was equipped with a load cell to measure the applied load to the wall specimen.

The displacement-time history cyclic loading protocol (CUREE) is shown in Figure 2. The displacement amplitudes were obtained using the results of the monotonic test on unfinished wood-frame specimen from the previous study (Memari et al. 2008). Horizontal and vertical displacements were measured using a series of potentiometers. The wood stud framing was initially constructed on the laboratory floor using conventional 2"x4" studs. Each frame consisted of seven studs, one bottom plate and a double top plate. The two 2"x4" double plates were attached to each other using 10d common nails at 24 in. on centers. The studs were connected to top and bottom plates using two 16d common nails driven at an angle at each end. The wood-frame was then sheathed with two 1/2 in. thick 4 ft x 8 ft drywall sheets on each side oriented horizontally to create a horizontal joint at mid-height of the 8 ft x 8 ft panels. The drywall was



Figure 1. Details of test set-up

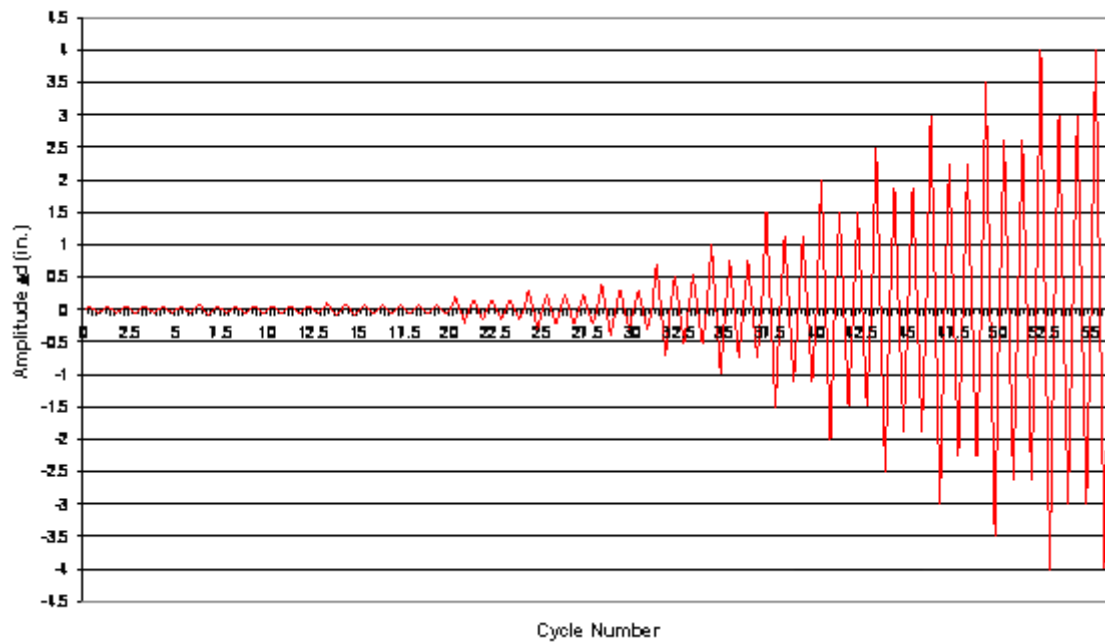


Figure 2. CUREE Loading Protocol, showing cycle number vs. target displacement

attached to the framing using No. 8, 1¼ in. coarse-threaded drywall screws at 4 in. o.c. for the perimeter and 8 in. o.c. for interior studs. The finish material consisted of CTS Cement's Rapid Set OnePass joint compound and UGL 222 joint compound. The OnePass product mix includes premium grade hydraulic cement, high performance polymers, and finely ground aggregates. Compressive tests of 2 in. x 2 in. cubes of the material have shown strengths of at least 1700 psi at 24 hours and 2700 psi at 28 days based on a mix of 40% (weight) water and 60% OnePass powder. The material properties for UGL compound were not available at the time of writing this paper. Figure 3 shows the photographs of the containers/box for both types of materials used. The former was applied to the drywall by a CTS representative, while the latter was applied by a professional drywall installer, as shown in Figure 4. The applied areas were sanded after the joint compound had sufficiently dried, as shown in Figure 5.

TEST RESULTS

The objective of this study was to evaluate the performance of the tested specimens under in-plane cyclic racking loading and to determine the effect of the difference of joint compound type on shear capacity of light-frame walls. For each specimen tested, load-displacement plots were developed using the data recorded for load by load cells and for displacement by potentiometers. The hysteresis curves were plotted for a selected number of primary cycles. The envelope curve generated by connecting peak points of primary cycles was also plotted. Figures 6-7 show a typical UGL specimen after the test with a crack through the entire horizontal drywall joint as well as details of failure mode. Such modes were similar for all three UGL specimens. As the figures show, the UGL joint compound cracked at the joint allowing the two drywall sheets slide pass each other. The shear in the panel resulted in failure of the drywall joint through cracking of the joint compound and tearing the mesh tape. On the other hand, there was no relative movement of top or bottom plates with respect to the drywall. This means that drywall screws at top and bottom plates and stud screws closer to top and bottom plates did not make enlarged holes on drywall since the compound at horizontal joint was weaker in resisting the shear.

Figure 8 shows the load-displacement hysteresis curves for one of the UGL specimens. The monotonic load-displacement curve for an unfinished specimen tested previously (Memari et al. 2008) is also plotted for comparison. The test results showed that the peak shear strength based on the envelope curve is approximately 3200 lb, 3650 lb, and 3350 lb, respectively, for UGL24HR1C, UGL7D1C, UGL7D2C specimens. The monotonic test peak for unfinished specimen is approximately 2800 lb.

These results show that longer cure time can increase the strength of UGL joint compound and that using two coats may not necessarily result in increase capacity. However, since these results are based on only one specimen of each configuration and cure time, obviously one cannot draw definitive conclusions, which should be based on statistically sound number of specimens. Nonetheless, we can compare the average peak value of the three tests, which is 3400 lb, with the monotonic peak value of the unfinished specimen (2800 lb) to obtain a ratio of 1.21, or 21% increase in shear capacity due to



Figure 3. UGL and Rapid Set OnePass joint compounds used



Figure 4. Application of joint compound over the horizontal drywall joint



Figure 5. Sanding the finishing compound after being sufficiently dry

finishing the joint with UGL compound. Although specimen UGL7D2C showed smaller shear capacity compared to specimen UGL7D1C, nonetheless, it is expected that a thicker coat will result in higher capacity. The peak value of the envelope curve can be considered as the ultimate shear strength for a given specimen. Using a factor of safety of 3.0, which is normally used for light-frame wall systems, we can find an estimate for the allowable shear value. On the basis of the average peak value, the allowable shear value for the UGL specimens is approximately 1130 lb.



Figure 6. Sliding of drywall sheets at the joint and tearing of drywall by edge stud screws



Figure 7. Condition of drywall near the loading jack and at end studs

The same type of test results were developed for the two specimens finished using cement based Rapid Set OnePass joint compound. Figures 9-10 show one of the specimens after the test including several details on the failure mode. The failure modes for the two specimens were practically the same. The figures show that the Rapid Set OnePass joint compound did not crack at the drywall joint and there was no sliding of the two drywall sheets. The figures show that because the joint compound did not crack at the horizontal drywall joint, the screws pushed through the drywall to accommodate the frame deformation.

Figure 11 shows the plot of load-displacement hysteresis curves for one of the two Rapid Set OnePass specimens along with the monotonic curve for the unfinished specimen. The test results show that the envelope peak value for shear strength is approximately 5000 lb for both specimens. This shows that for Rapid Set OnePass joint compound, longer (than 24 hours) cure time will not necessarily increase the capacity. Compared to the monotonic test on unfinished specimen, we have a peak strength ratio of 1.78, or 78% increase in shear capacity due to finishing the joint with Rapid Set OnePass joint compound. The allowable shear value for these specimens is approximately 1660 lb.

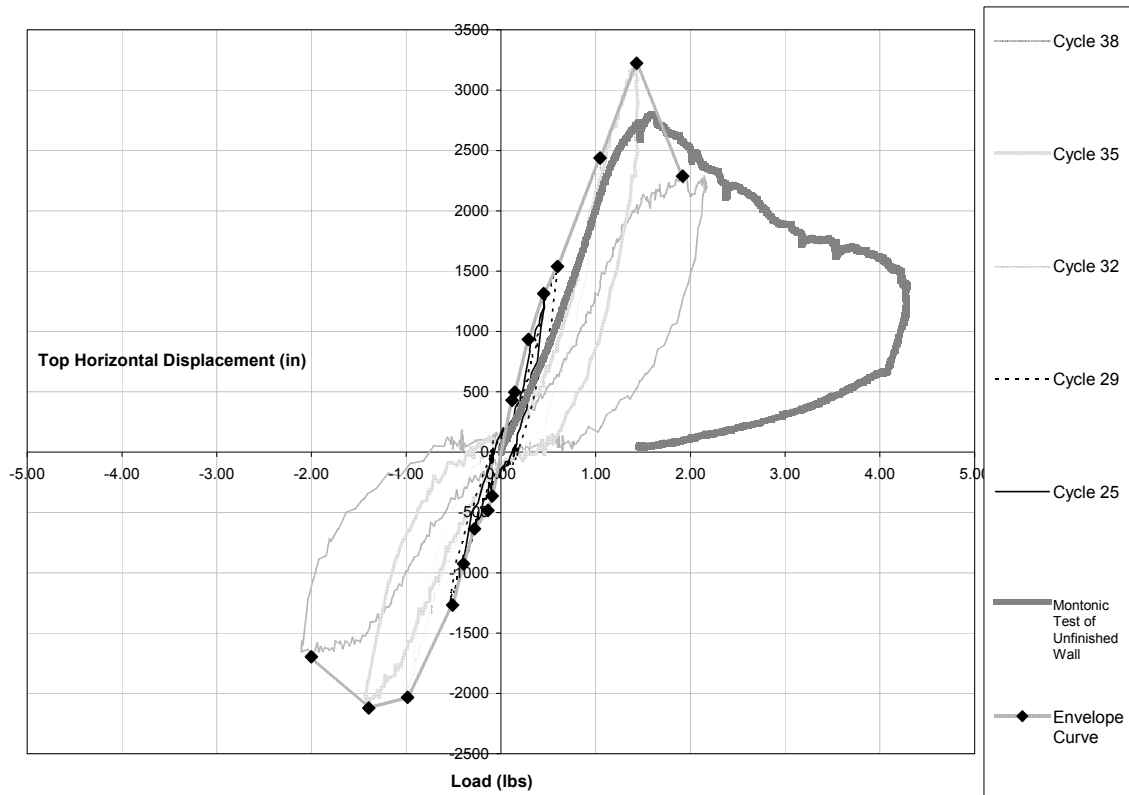


Figure 8. Load-displacement relationship for UGL specimen UGL24HR1C



Figure 9. No crack through any part of the horizontal joint finished with OnePass compound

The first aspect to compare the UGL and OnePass specimens is that for failure modes. Figures 6, 7, 9, and 10 show the difference in failure modes for the two different joint compounds. Close inspection of the OnePass specimen after the test showed that the OnePass joint compound had bonded to the drywall paper through the mesh tape, and that the tape could not be peeled off after the test. For the UGL compound, however, the tape could be peeled off, exposing the drywall paper. The second and more important comparison, however, is the peak load. As described earlier, the test results show that the average shear strength of the OnePass specimens is 5000 lb, while that for UGL

specimens is 3400 lb. This shows that the OnePass specimens had on the average 47% higher shear strength compared to the UGL specimens. To compare all five specimens, the envelop curves are plotted in Figure 12. The plotted test results clearly show the difference in peak points of shear resistance. Once past the peak point, the capacity drops and that is where the tests were terminated.



Figure 10. Screws on end studs tearing through drywall

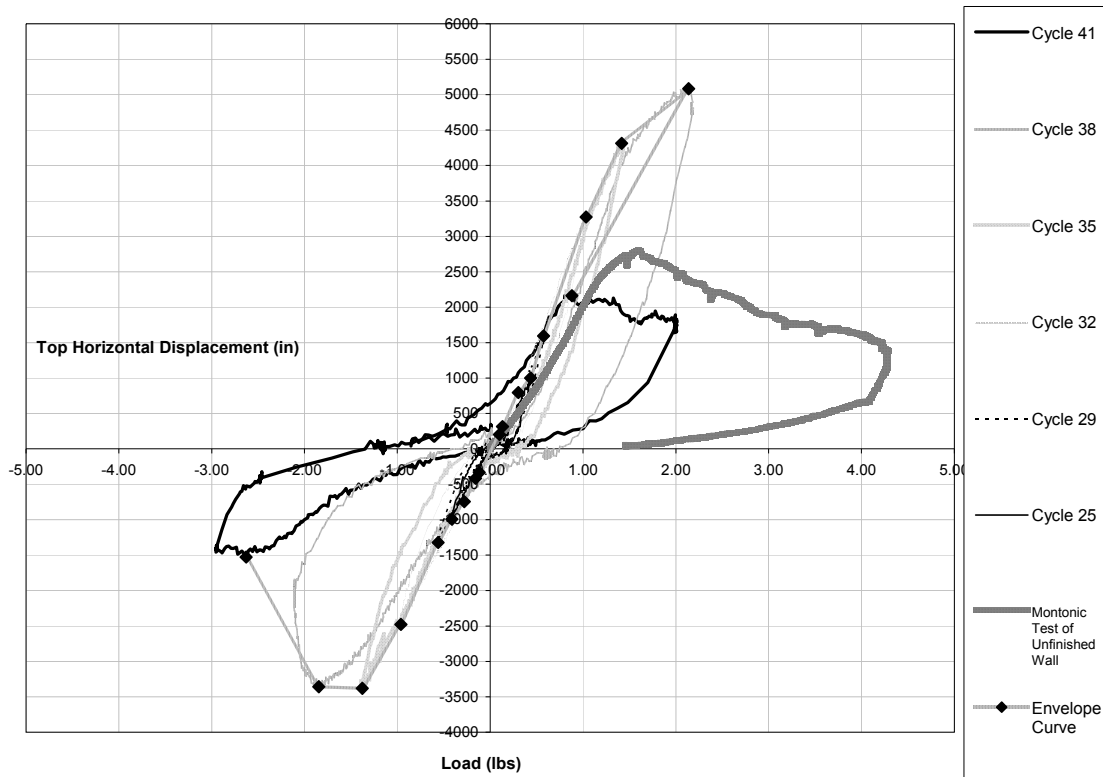


Figure 11. Load-displacement relationship for OnePass specimen RPDST24HR1C

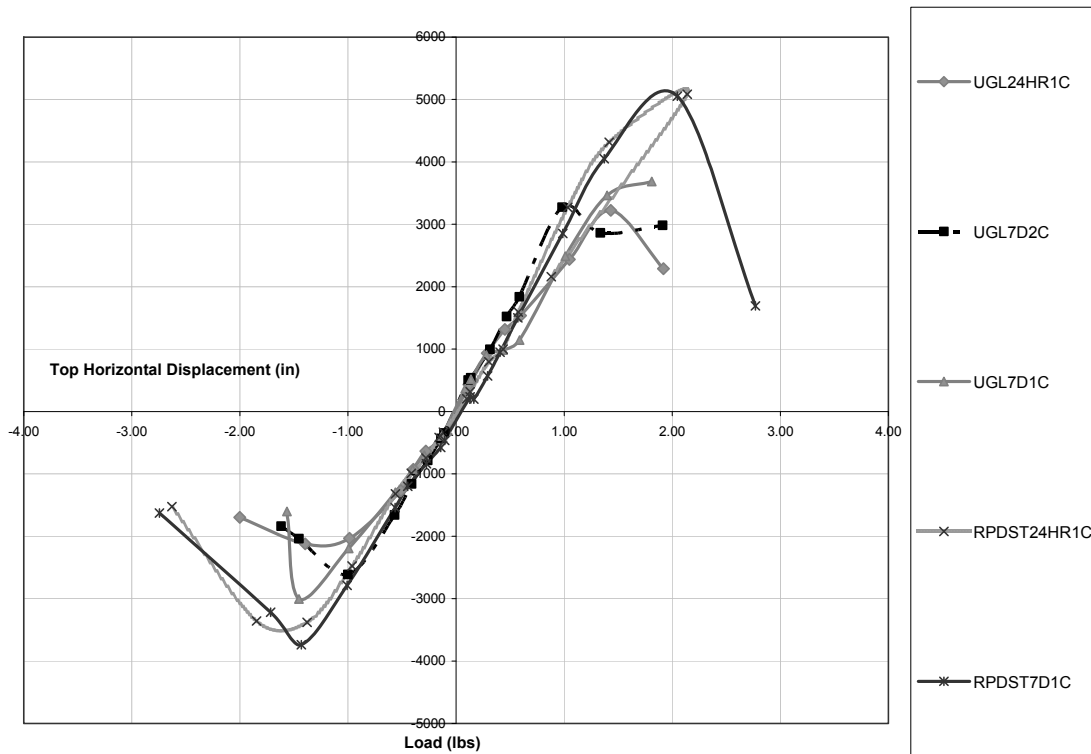


Figure 12. Comparison of the envelope curves for all five specimens

SUMMARY, CONCLUSIONS AND RECOMMENDATIONS

The results of this pilot study on five specimens consisting of three finished using a non-cement-based UGL joint compound and two finished using cement-based Rapid Set OnePass joint compound were presented. The cyclic racking test results showed that the non-cement based UGL finished specimens had on the average 21% higher shear capacity compared to an unfinished specimen, while cement based Rapid Set OnePass specimens resulted in 78% higher strength. Comparison of cement-based and non-cement-based joint compound specimens shows that specimens finished with Rapid Set OnePass compound had 47% higher shear capacity compared to the average of the specimens finished with UGL compound. Such a difference in capacity can be very important and attractive for high seismic regions where interior stud walls in residential construction effectively take part in seismic resistance even though wood shear walls are normally used on exterior walls. Curing time seems more important for the UGL compound compared to Rapid Set compound.

The results and conclusions presented are based on very few specimens and should only be viewed as providing a rough measure of the difference between the two types of joint compound and hinting the advantage that the cement-based joint compound can provide. Clearly, several tests of each type of configuration are necessary in order to develop statistically meaningful results. A comprehensive follow-up study is recommended to further compare the cyclic racking performance and energy dissipation capacity of wall

specimens with and without openings and finished using cement based joint compound and those finished using different types of non-cement based joint compounds. It is also of interest to compare the effect of cement based joint compound on both wood stud and steel stud specimens.

ACKNOWLEDGEMENT

The financial support of the CTS Cement Manufacturing Company for this study is gratefully acknowledged. The contribution of the following individuals is also acknowledged: William Splain, Andrew Adams, Thomas Kuczynski, and Joseph Standley. The opinions expressed in this report are those of the authors only and do not necessarily reflect those of the sponsor.

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Use of FRP Panels as Reinforced Segmental Retaining Walls Units

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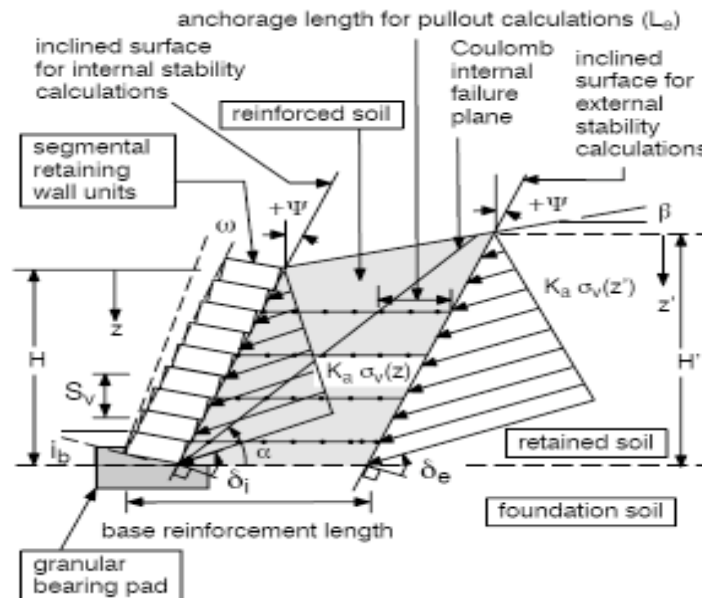
Abstract: Reinforced segmental retaining walls (SRW) have been shown to be a very effective earth retention system. Conventional reinforced segmental retaining walls use concrete blocks as their facial system, to which the earth reinforcing geogrids are attached. This binds the blocks and soil into one composite system. The use of FRP facial panels are proposed to increase the utility of reinforced SRWs by eliminating numerous structural problems faced by the conventional system. Reinforced SRWs pose complex structural questions due to the discrete nature of the blocks and the possibility of localized stress concentrations within the system. Moreover, computational analysis techniques are not easily applicable to reinforced SRWs due to the complex nature of the constituent elements. FRP panels could serve as a viable substitute for concrete blocks, and properties of FRP panel walls would be more predictable than concrete block walls, thereby, allowing a less conservative design. Furthermore, the use of the proposed FRP panels would allow significant cost savings and reduce construction time. The blocks that are used are rectangular in dimension and have a surface area of 1--2 ft²; an increase in block size would mean a reduction in construction time. The weight of the concrete blocks lends constraints to the block dimensions. However, using a different and lighter material for the units (i.e., FR polymers) would allow a larger size and would thereby reduce the construction time. To encompass all the aforementioned justifications, an FRP panel wall is proposed and its benefits are discussed. The article also compares the proposed FRP facial system to a concrete block facial system from structural and economical perspectives.

CE Database Heading: FRP Construction, Segmental Retaining Walls, FRP Panels, Sandwich Panels

INTRODUCTION

Reinforced Segmental Retaining Walls (hereinafter reinforced SRWs or simply, SRWs) were conceptualized in the late nineteen sixties. The first reinforced SRWs were built in France in 1970 and 1971 (Allen Bathurst and Berg 2002). Reinforced SRWs were first used in the United States in 1974; and ever since, their use has increased dramatically over the past decades (Bathurst and Simac 2004; Allen Bathurst and Berg 2002). Reinforced SRWs have amassed worldwide popularity. Segmental retaining walls are used in Taiwan alongside large-scale housing and industrial development sites located at the slopes and hillsides (Ling Leshchinsky and Chou 2001). In Japan, more than 26 km of segmental retaining walls were constructed during 1986-1996; furthermore, a 4.6 to 8.6 m high and 260 m long reinforced SRW was used to support a new bullet train line (Tatsouka et. al. 1997). These walls have rapidly grown in popularity partly because, for walls >1 m high, the total cost is 25–40% lower than the cost of conventional concrete retaining walls (Helwany Budha and McCallen 2001). In Korea, Reinforced SRWs were introduced in 1990s and have been rapidly growing in number ever since (Yoo and Jung 2006). Procedures for structural stability design of geosynthetic reinforced SRWs walls, which define the required strength, spacing, and length of the reinforcement, have become more conservative over the past 20 years, primarily due to the increase in knowledge regarding the durability of the structural system and geosynthetic reinforcement (Allen Bathurst and Berg 2002). National Concrete

and Masonry Association Manual for Design of Segmental Retaining Walls is the most widely adopted document for segmental retaining wall design.



Source: Bathurst and Simac 1994

Figure 1: Principal components, geometry of Reinforced SRWs and earth pressures assumed in NCMA method

Though Reinforced SRWs have been successful, they are a relatively newer structural system and the scope for improvement is still not limited.

POSSIBILITIES OF IMPROVEMENT IN THE EXISTING SYSTEM

Structural Viewpoint

Reinforced SRWs, with concrete block facial systems, have, in past, encountered fewer problems when compared to other neo structures and structural systems. But, a number of reinforced SRWs across the globe have failed due to a variety of reasons. Collin (2001) attributed most of the failures to poor construction, poor engineering, inferior quality materials, unexpected conditions (Acts of God) and lack of coordination of responsibilities between the Owner, the Design Consultants, and the Contractor. Adoption of FRP panels instead of concrete blocks is aimed at eliminating, at least, the following structural problems:

1. Settlement
2. Facing connection Failure
3. Lateral localized movement of the wall face

Settlement

The weight of the facial units often renders SRWs inapplicable in soft and light soils. Soft soils being compressible in nature yield under the weight of the wall, thereby, raising serviceability concerns. The second segmental retaining wall built in the United States suffered creep suffered an initial settlement of more than 1m (Allen Bathrust and Berg 2002). Borges & Cardoso (2001) concluded the principal stresses during the construction phase are different from principal stresses in service life. In the post construction period, characterized by consolidation, the volumetric stresses in soil increase and cause generalized settlements. Skinnera and Rowe (2005) evaluated the finite element strain values at the

bottom of the wall. They concluded that although it has been shown that current design methods can be conservative with respect to the expected reinforcement strain (Allen and Bathurst, 2002), this may not be the case for a wall which experiences significant unexpected yielding of the foundation soil. Therefore, it is evident that a solution to reinforced SRW settlement is required.

Facing Connection Failure

Figure 2 shows a typical connection between the reinforcing geogrid and concrete block facial system. The connecting pins are typically used to anchor geogrid reinforcement to the wall face. Geogrid connection failure is said to have occurred if the face of the wall shears off leaving the geogrid reinforcement intact. Collin (2001) concluded that facing connection failure is common and is usually due to negligence in design procedures and onsite contractor negligence. In the currently used connection detail, the dislodging of the concrete blocks and anchorage failure are inter-related subsequent events and occurrence of one leads to the other one as well. During seismic activity, the geogrid is sandwiched between the concrete blocks and relative movement between the two blocks results in abrasion on the geogrid. Allen, Bathurst and Berg (2002) attributed, in some cases, the strength loss in geogrid to chemical interactions between concrete, soil and geogrid.

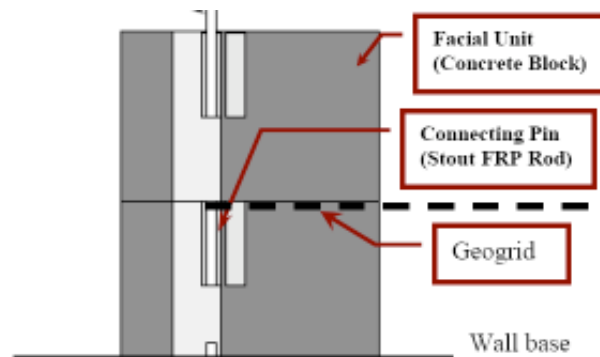


Figure 2: Geogrid Connection to the Wall

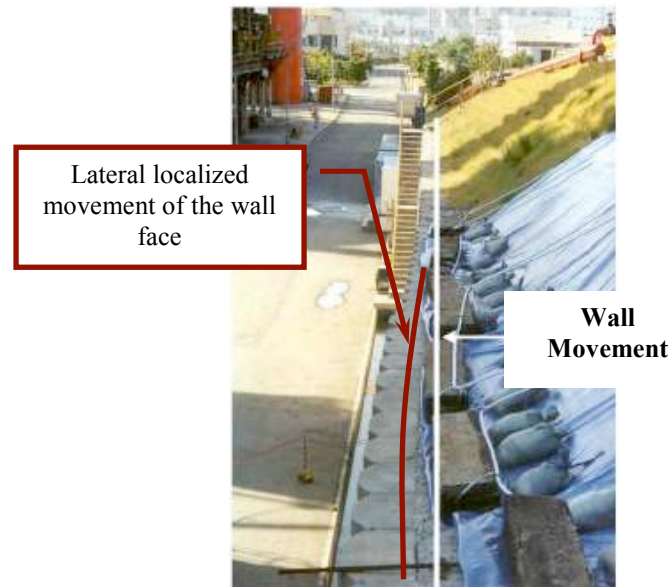
The allowable strength of the reinforcement as per the NCMA Manual is given by

$$T_a = \frac{T_{ult}}{RF \times FS} = \frac{T_{ult}}{RF_{ID} \times RF_{CR} \times RF_D \times FS}$$

where RF_{ID} and RF_D are the reduction factors that account for strength loss due to installation damage and chemical and biological degradation respectively. Use of FRP Panels would allow a reduction in the aforementioned factors, thereby, allowing a less conservative design.

Lateral Localized Movement of the Wall Face

Localized deformations of the wall face perpendicular to the plane of the wall are referred to as lateral localized movements of the wall. Figure 3 depicts the lateral deformation of a 6 yr old wall as observed by C. Yoo(2003).



Source: Performance of 6 Yr Old SRW by C. Yoo, 2003

FIGURE 3: Lateral Localized Movement of Six Yr old SRW

The lateral deformation in the walls is attributed to mainly, two reasons: creep and stress relaxation and strength loss in the geogrid. Allen and Bathurst (2001) concluded that resistance to lateral movement is affected by the stiffness of the facing and restraint at the wall toe, decreases w.r.t. time but is independent of the wall height. Additionally, Allen, Bathurst and Berg (2002) concluded that stiffness of the geosynthetic reinforcement decreases with time, causing release of stress and horizontal displacements of the wall face. Tatsouka et al (1997) found that the backfill immediately behind the walls undergoes lateral creep deformation over time. Creep displacements and subsequent stress development is hard to analyze; therefore, the creep resistance is provided using large safety factors during the design process. Reinforced SRWs are designed utilizing the friction between consecutive columns of blocks to resist the forces in a horizontal plane. These aspects of the design can be improved upon.

Economic Viewpoint

Reinforced SRWs are constructed in successive rows, consecutively backfilling behind the rows and installing geogrid reinforcement along process; Therefore, an increase in row height would mean lesser number of rows for a given wall height; which, in turn, would require lesser use of resources and speedier construction. But, height of the blocks is constricted by the block weight. Low block weight favors workability, speedy construction and transportation. Larger facial units would be very favorable but, would require a material lighter than concrete. Additionally, concrete blocks have high unit weight; therefore, heavy lifting machinery is required for construction. Use of a lighter material would reduce the construction cost by eliminating the need for heavy lifting equipment.

PROPOSAL / DESCRIPTION OF THE PROPOSED PANEL

Owing to the complex nature of the application, a number of panel designs were investigated and evaluated with the following in mind:

- The panels would have to be strong enough to resist the exerted soil loads.
- The panels would be exposed to adverse weather conditions.

- The panels would allow for geogrid anchorage to the panels. Geogrid anchorage would result in stress concentrations at the points of anchorage. Thereby, causing deflections in the front face of the panel.
- In order to form a wall, the panels would require an interlock between consecutive panels.
- The panels would have to allow an assembly for geogrid anchorage to the wall.

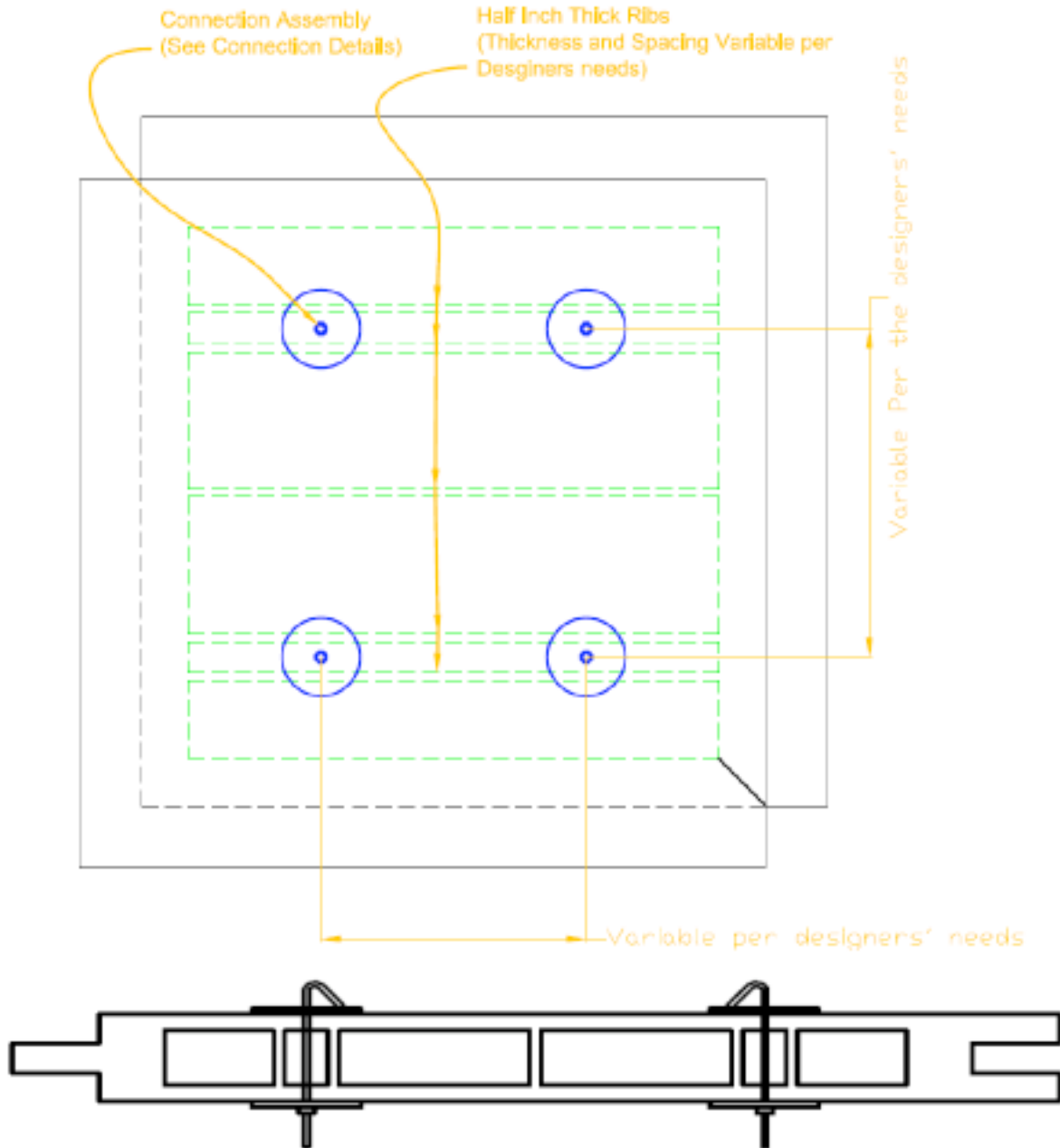


Figure 4: Proposed FRP Panels for SRW Walls

Figure 4 shows the proposed FRP panel shape for use as the facial unit. The main components of the 48 in long panels include:

- $\frac{3}{4}$ in Thick FRP plates
- 4 inch long end zones, groove on one side and a protruding tongue on the other
- 5- $\frac{1}{2}$ Ribs with 2 pairs spaced 2 inches apart.

- Steel connecting plates as a part of connection assembly
- Hooked Steels threaded at the straight end as a part of the connection assembly

The voids in panel could be filled with foam or asbestos or some other cheaper or viable alternative. The stiffness of the filler material was not used in any of the structural calculations for the panel.

Sandwich panels can be fabricated using two processes i.e. pultrusion process or resin molding process. Maximum panel thickness produced by the pultrusion process, till date, is 4 inches. Panels thicker than 4 inches would have to be manufactured using the resin molding process. The designed panel suffices the criterion for easy, cheap and assembly line mass production based on the pultrusion process and panels could be core voided to decrease cost. The number and the location of the panel reinforcement and the geogrid connections within the panel would be alterable based on the designers needs. The connections between successive panels would be moment relieving and would be designed to resist shear stresses arising from soil loads only. Successive panels would be attached to form the wall as shown in figure 5.

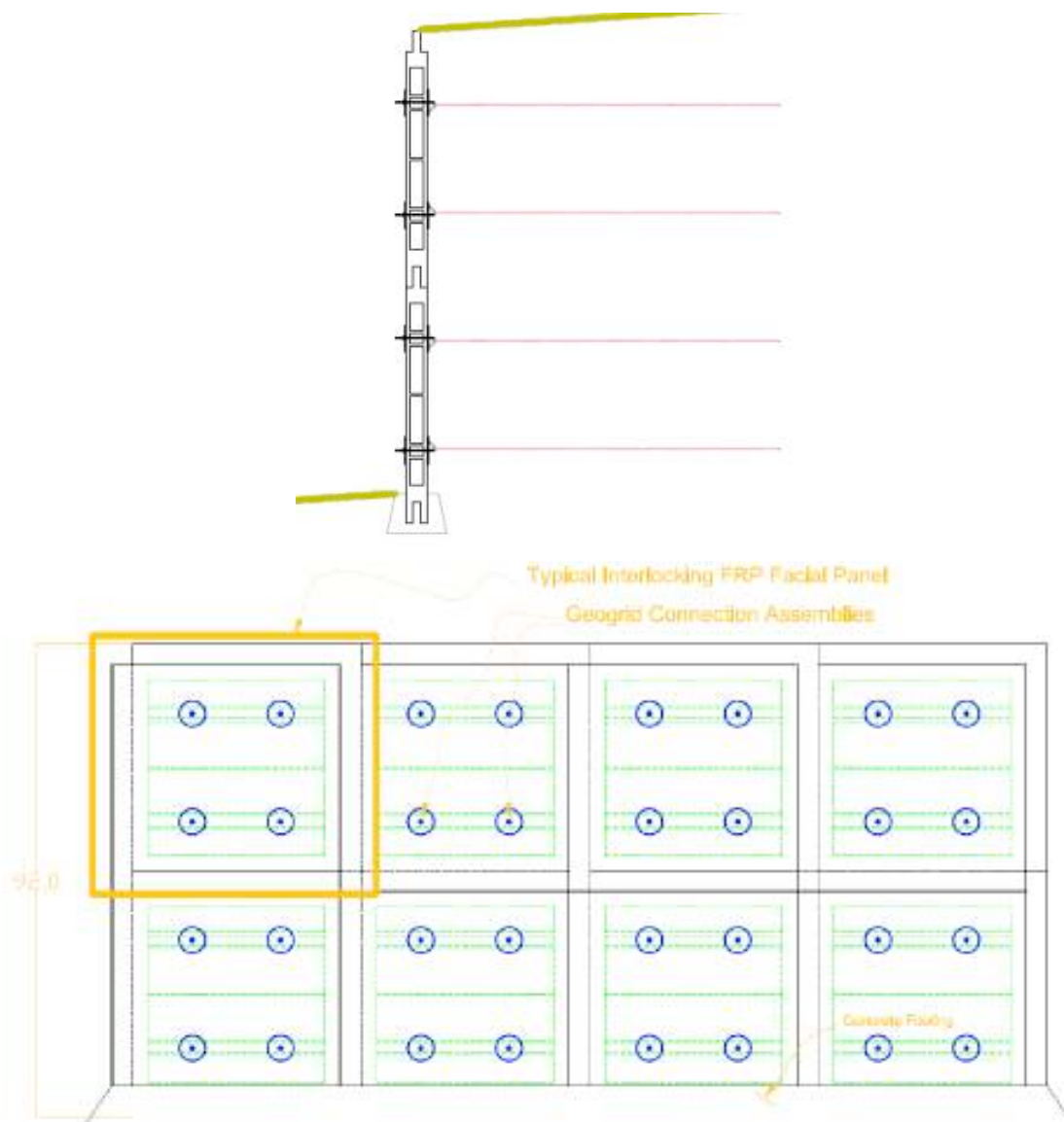


Figure 5: Proposed Wall Elevations

Remediation of the aforementioned problems

Structural Problems

Settlement

FRP weight density is very low as compared to concrete therefore, the proposed wall would be lighter and would greatly reduce the settlements; both localized and overall. Use of FRP panels would also allow an increased applicability of reinforced SRWs in areas with soft compressible soils.

Facial Connection Failure

FRP panels in the proposed system would be individually anchored to the reinforcement. Therefore, the reinforcement failure at one location would not be allowed to progress to the other parts of the wall. The geogrids would be anchored to the center of the panel rather than its edges thereby eliminating gravity induced abrasion. Moreover, the FRP panels being chemically inert would function well with all kinds of soils. The number of geogrid connections per panel could be modified to meet the designer's needs, thereby lending additional resource to the designer.

Lateral Localized Movement of the Wall Face

The proposed facial panels would be symmetrical along both the axes; therefore, panel rigidity along both the axes would be equal. Moreover, all the panels would receive geogrid anchorage and panel interconnections are similar in both the axes. Therefore, it can be concluded that the FRP walls would have equal rigidity in both the directions. Thus, FRP walls would be favorable in resisting lateral deformation due to the increased wall rigidity in the horizontal plane.

Economic Feasibility

FRP panels have very low mass densities therefore; even large sized panels would be easily workable. Thus, the maximum height or length of the panels would be determined by the structural capacity rather than their weight. The structural capacity of the section can, in turn, be modified by modifying the amount of reinforcement or the section thickness. Therefore, theoretically, a wide variety of panel sizes could be utilized. Use of FRP panels would speed up the construction process, along with additional cost saving due to the reduction in use of heavy machinery.

PRELIMINARY STRUCTURE ANALYSIS DESCRIPTION

Soil Distribution behind the wall

Soil pressure distribution behind the wall face was calculated using the NCMA procedure with modifications to suit the FRP wall case. The manual uses concrete blocks as the facial system for the wall. The surface of FRP panels is very smooth as compared to that of concrete blocks; thereby, the angle of interface friction between the soil and FRP panels is lower than that between soil and concrete blocks. In order to adapt the manual to the proposed system, the active pressure equations were modified to incorporate a very low angle of interface friction angle.

Local and Global Stability Analysis

National Concrete and Masonry Association methodology requires the wall stability analysis in 9 different local as well as global failure modes. Each of these failure modes was individually investigated

to determine the effects of facial system change. It was found that the effects of changing the facial system were minimal and were easily rectifiable.

Bending and Shear Analysis

The active pressure force at the bottom of an 8' high FRP SRW was evaluated to be 3 kips/lineal foot per the NCMA methodology. The bending analysis on the panels was performed using SAP 2000. A 4 foot X 4 foot panel was defined in the program using shell elements. The panel was modeled in the X-Y Plane. Four joints between the shell elements were used as the support locations. The panel was simply supported on four interior points, analogous to the actual case. In order to provide stability to the model on of the corners of the panel was provided by an additional support. A static load of 2000 lb/ sq ft was imposed on the panel along the Z-axis. The interpane006C connections were modeled as hinges transferring shear only.

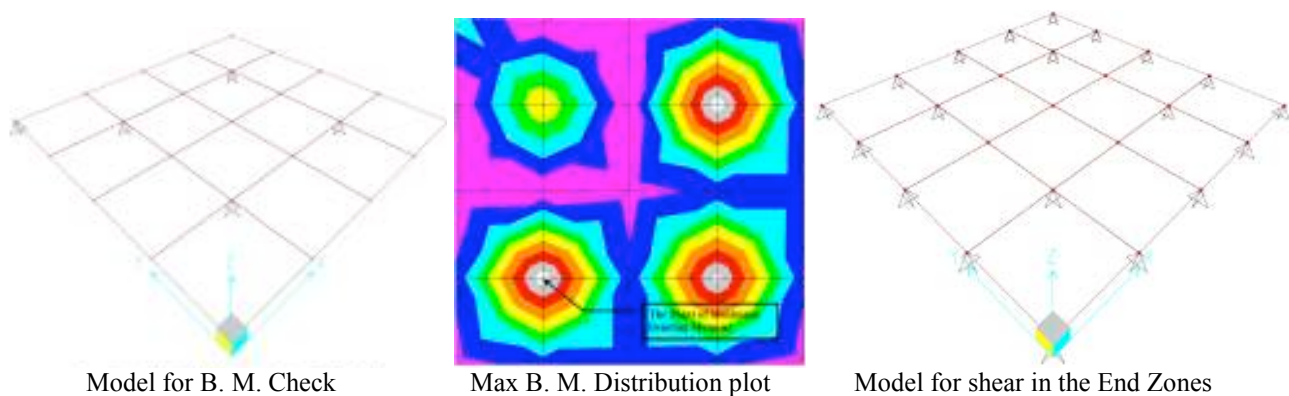


Figure 6: SAP2000 Analysis Models and Bending Moment Distribution

The shear force at the panel end zones were evaluated using SAP 2000. In this case, the panels were modeled with pin supports at all the four edges. The panel was analyzed with a superimposed load of 2000 lbs/ sq ft. The reaction at all of the edge supports were summed and distributed over the perimeter of the panel.

The results from the aforementioned analysis were used to design a FRP panel with fiber glass reinforcement. It was theoretically ascertained that panels could be manufactured using 60% volume fraction glass/epoxy composite.

CONCLUSION

Additional practical analysis of the aforementioned FRP panels is required to ascertain the range of applicability of the proposed retaining walls. Laboratory testing of FRP panel specimen would be very beneficial. It can be safely assumed that the proposed walls would increase the applicability of SRWs, would ease the construction procedures and result in considerable cost savings.

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CONCRETE INHOMOGENEITY OF VERTICAL CAST-IN-PLACE ELEMENTS IN SKELETON-TYPE BUILDINGS

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Abstract

Investigation of concrete inhomogeneity of concrete fragments of cast-in-place skeleton-type buildings, which were produced simultaneously with the building vertical structures erection, is carried out. Ultrasonic pulse method and the method of sawing the full-scale fragments into experimental samples – prisms, with their following testing were used. The analysis of the revealed differences of density, strength and deformation characteristics of different zones of concrete column fragment is discussed and presented.

1. INTRODUCTION

Investigation of strength and deformation characteristics of the vertical concrete fragments [1] [2] [9] [11], cast in the conditions of a construction site (simultaneously with the erection of vertical members of a 25 and 16 – storied cast-in-place skeleton-type buildings) showed the existence of essential inhomogeneity of strength and deformation characteristics of different zones of vertical members (the character of experimental concrete column fragments failure - location of cracks in the top part of fragments and opposite result in case of turning it over).

The differences in strength, elastic characteristics, shrinkage and creep deformations and modulus of elasticity of different zones of vertical concrete elements are the consequences of inhomogeneity and the differences in stress state, caused by the processes, taking place during concrete hardening [3]. Inhomogeneity of the concrete structure by the volume of the construction is due to different humidity content of separate layers and to the influence of various degree of concrete compaction during the placement and vibration. Among the works dedicated to the inhomogeneity of both precast and cast-in-situ concrete and consequent strain gradients) works [4] should be mentioned. The ultrasonic pulse method and the method of sawing samples into the reference prisms with subsequent determination of strength and deformation characteristics of concrete of different zones were also used in work [4]. But the investigation was carried out using samples, produced in standard conditions of a laboratory building, and placement of concrete mixture and its compacting (by a vibration exciter) were carried out in a horizontal position at that. Thus, concrete inhomogeneity of full-sized samples, produced in the conditions of a construction site is a valid one for further investigation.

2. PROCEDURE OF EXPERIMENTAL COLUMN FRAGMENTS PRODUCTION AND CURE

The procedure of experimental samples production was planned taking into account necessity of production and keeping some experimental fragments (till the attenuation of strengthening) nearby the place of real vertical structures production.

Research has been carried out on two full-sized concrete fragments of vertical members with cross section of 400×400 mm (Table 1), made directly on construction site to avoid any differences between properties of real structures and experimental column fragments. The height of fragments was assumed 1500...1700 mm.

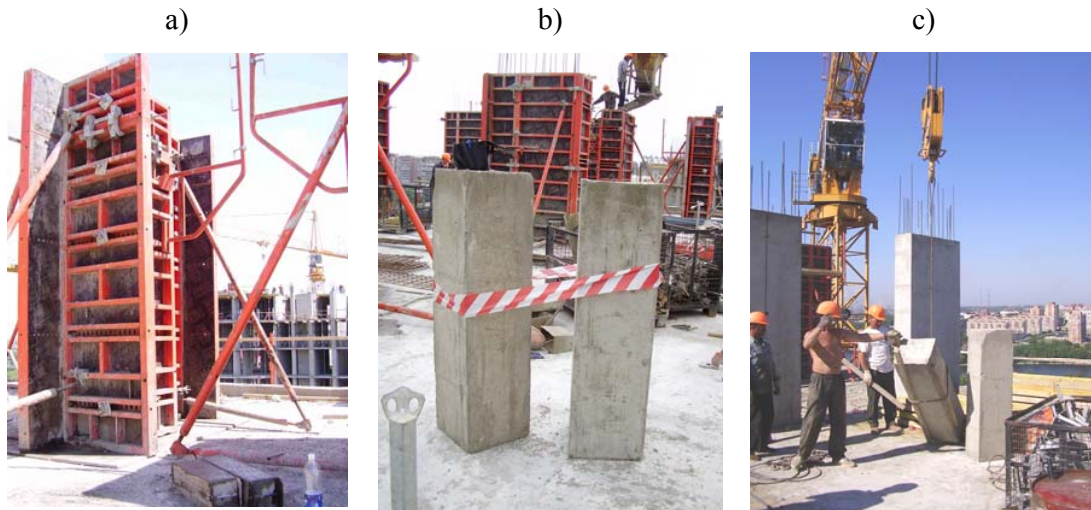


Figure 1: a) Production of experimental column fragments; b) keeping them on current division; c) taking them down to the ground level

The concrete mix (B-25) of the following content, at the rate of on 1m³ was used for producing vertical structures of considered cast-in-situ skeleton type buildings: Balakleja Portland cement – ‘400’ – 460 kg.; quartz sand from deposit “Krasnaja Poljana” (the module of particle size Mps=2.61) – 360 kg.; quartz sand from deposit "Vodino" (Mps=1.0; average Mps=1.8) - 250 kg.; granite crushed stone from Teljmanovo deposits (fraction 5-20 mm) – 1200 kg.; water 250 l; plasticizing additive LST (technical lignosulfonat)-0.92 kg.

Concreting of experimental column fragments has been fulfilled simultaneously with concreting of vertical structures of the 9-10 the storied residential building. Identical: formworks, geometrical dimensions of cross-section, concrete mixture content, ways of its placement and compaction, way of formwork dismounting were used. Experimental fragments were situated close to building vertical structures what allowed to avoid differences in: external air temperature, its humidity, intensity of air blow-off, the influence of solar insulation etc.

Table 1: Description and characteristics of the column fragments and reference samples

Dimensions and description of the concrete column fragments of the considered series	Strength and deformation characteristics of concrete column fragments			Averaged strength and deformation characteristics of 6 reference samples (prisms 0,15×0,15×0,6 m)		
	Strength (R), MPa	Modulus of elasticity (Eb) ×10 ³ MPa	Utmost compressibility (ε _{ub}) ×10 ⁻⁵	Strength (R) MPa	Modulus of elasticity (Eb) ×10 ³ MPa	Utmost compressibility (ε _{ub}) ×10 ⁻⁵
0,4×0,4×1,5 m ('not-sawn')	27,5	—	190	24,7	21,7	270
0,4×0,4×1,5 m ('sawn' - Fig.3)	Fig. 5,6,7					

The formwork ("PERI", shields – laminated plywood 21 mm by thickness) for erection of vertical bearing structures (columns, pylons and walls) was 3300 mm in height. To avoid adhesion between the fragments' bottom and slab on current division (9-th storey) two layers of the polyethylene film were put under. Concrete mix feeding, placement and compaction

carried out in layers through all height by a tub and immersion vibrator correspondingly. Working part of immersion vibrator both in real columns with the same cross section and in experimental column fragments submerged in 2 points on cross section. Duration of vibrating was 30...40 seconds. Velocity of withdrawal of the immersion vibrator working part from the concrete mixture, approximately, equaled the velocity of its immersion due to own weight, what allowed decreasing possibility of air cavities formation.

The reference samples were made simultaneously with the placement of concrete mixture into the formwork of building vertical members and experimental fragments. With the purpose of taking into account possible changes of the concrete mixture content (water cement ratio), the concrete mixture for production of reference samples (Table 1) was taken directly on the construction site after delivery of the concrete mixture by transit-mix truck from the mixing station, where the components were mixed in the concrete mixer of the forcing type.

Taking of the first probe was fulfilled directly at the moment of discharging the concrete mixture from the transit-mixer truck into a tub, and second probe at the moment of placing the mixture into formwork of fragments (on the 9-th floor). The mixture of the second probe was used for remolding reference samples directly on the working level, where they have been kept together with column fragments. The mixture from the first probe was moved to the construction site laboratory where the reference samples were remolded according to requirements of [12][13][15].

Dismounting of the vertical structures' and experimental fragments' formwork was carried out next morning (in approx. 18-20 hours). Column fragments were left on the working division exposing conditions identical to those real vertical structures were in.

After 3-4 days, experimental fragments were lifted to the next operating level (division) and were kept on the building for a month in total. It was necessary to provide the identity of conditions during the period of active strengthening (maturing). Then the fragments were taken down from the building and were kept on the ground level of construction site for 3 months and then were transported to the laboratory building of DonNACEA for further curing.

During preparing of the fragment for testing the leveling of top and bottom end bearing surfaces of the fragment with the cement-sand mortar was made. Also the top and bottom bearing surfaces were strengthened with steel frames; gaps were thoroughly filled with cement-sand mortar.

Longitudinal deformations were measured by dial gauges graduated in 0.01 mm along the base of 550 mm (400 mm for reference samples – prisms), and lateral ones by dial gauges graduated in 0.001 mm along the base of 400 mm (150 mm correspondingly) [14]. Static axial compression tests of the first 'not-sawn' column fragment (Fig. 4a) were carried out using test instrument PMM-1000 (rated capacity 1000 tons).

Straight after failure of the fragments probes of concrete from external and internal zones of fragment (at a 3-rd layer level) were taken. The content of water (mass humidity) was determined by drying probes till the constant mass.

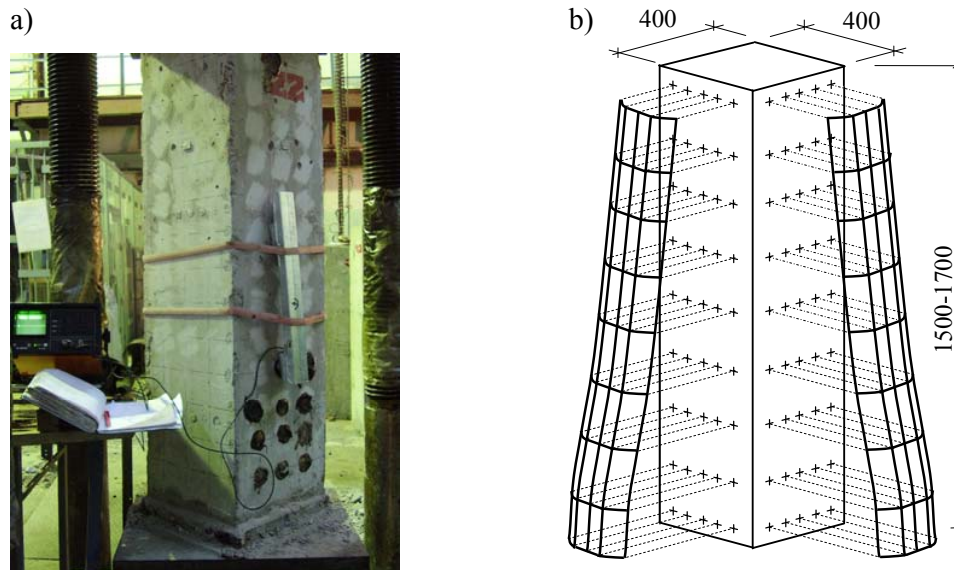


Figure 2: a) Measuring of velocity of ultrasonic waves before the compression test; 2) change of the concrete density of different zones of column fragments

3. ULTRASONIC PULSE METHOD

3.1 Procedure

For research into inhomogeneity of concrete of different zones of the concrete column fragment the ultrasonic pulse method was applied according to recommendations of [4][12].

In the process of investigation the testing instrument GSP UK-10 PMS was used. The velocity of the first front of longitudinal ultrasonic wave (from now on 'velocity of ultrasonic') 'v' propagation through the concrete point by point preliminary marked grid on the fragment. Special holders, fixed to column fragment by means of elastic strips, were used for holding in constant position transmitting and receiving sensors and application to them clamping force of 20-30 N (Fig.2a). Lubrication of contacting surfaces of concrete and sensors was used to provide reliable acoustic contact.

3.2 Results

Curves of changing of ultrasonic velocity in the transversal directions and in the direction of column fragment height are presented in the Picture 2b. Velocity of ultrasonic propagation (and correspondingly the density of concrete) in the transverse direction higher in the middle and lower to sides. And the same decreases from the lower part of concrete column fragment to the upper part. Unfortunately variation of the velocity values gives only relative values of density changing and accordingly changing of strength.

4. METHOD OF CUTTING OF THE FULL-SCALE CONCRETE COLUMN FRAGMENT INTO EXPERIMENTAL SAMPLES - PRISMS

4.1 Procedure

To get the absolute values of changing strength and deformation characteristics one column fragment has been sawn into samples - prisms (Fig. 3a; Table 1). Sawing of the column fragment was performed by diamond cutting discs (in diameter 1.0-1.2 m) on orthogonal industrial stone-cutting machines (used for granite cutting). Age of concrete column fragment at the moment of cutting was more then one year. By means of decreasing to minimal of the

immersion value of the cutting disc's operating body into the body of fragment for one advance, we have achieved high quality (Fig. 3b) of the slice surface and absence of aggregate grains (particles) knocking-out. In the process of cutting, samples were marked, according to the layer number (Fig. 3a) and their location in the body of every layer Fig. 3e.

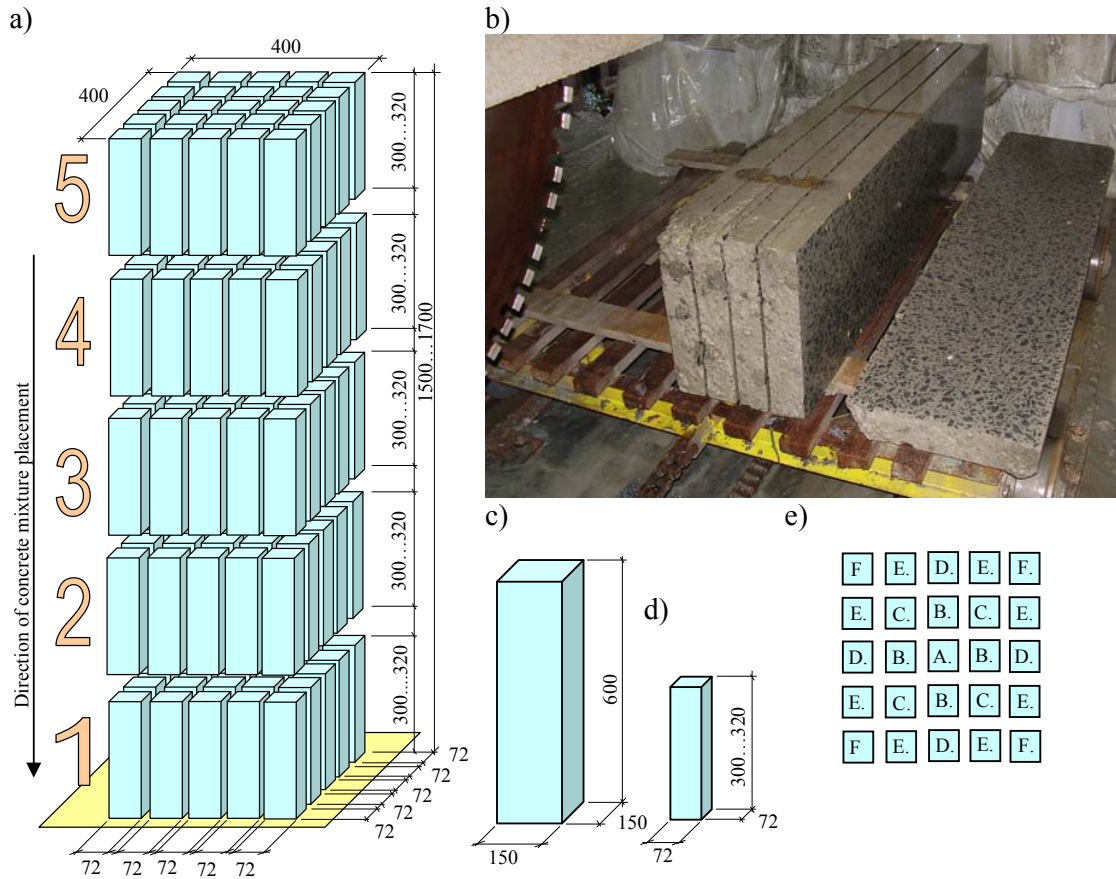


Figure 3: a) Dimensions, location and marking of layers and sawn out samples from column fragment's body; b) stage of sawing; Dimensions of: c) reference samples (prisms); d) sawn out samples; e) sawn out specimens in cross-sectional direction (3-rd layer) grouped and marked according to the similarity of their location.

After cutting, samples (prisms) were kept in the normal conditions of the laboratory building for about one year and then were tested by axial compression tests [14] to determine main strength and deformation characteristics of different zones' concrete. Samples with defects (soil inclusions etc) were rejected.

Strength and deformation characteristics of the reference samples with dimensions (prisms 150×150×600 mm) and sawn out samples (prisms approx. 70×70×310...320 mm) were determined by means of their testing (axial compression test). Testing was fulfilled in the laboratory building of DonNACEA (Ukraine): air temperature - 20±5°C; air humidity - 70±10% using hydraulic pressing machine P-125, according to requirements of [14]. The aim of the testing was to obtain: compression strength (R); modulus of elasticity (Eb) and utmost compressibility (ϵ_{ub}).

We accepted the utmost compressibility as the longitudinal deformations corresponding to stress equaled to compression strength (R). Unfortunately, the holders used (Pic.4c) for fixing

dial gauges in longitudinal direction has not allowed to determine utmost compressibility (ϵ_{ub}) of sawn out prisms, because we had to take off rigid holders before the failure.

Modulus of elasticity (E_b) was calculated according to demands of [15].

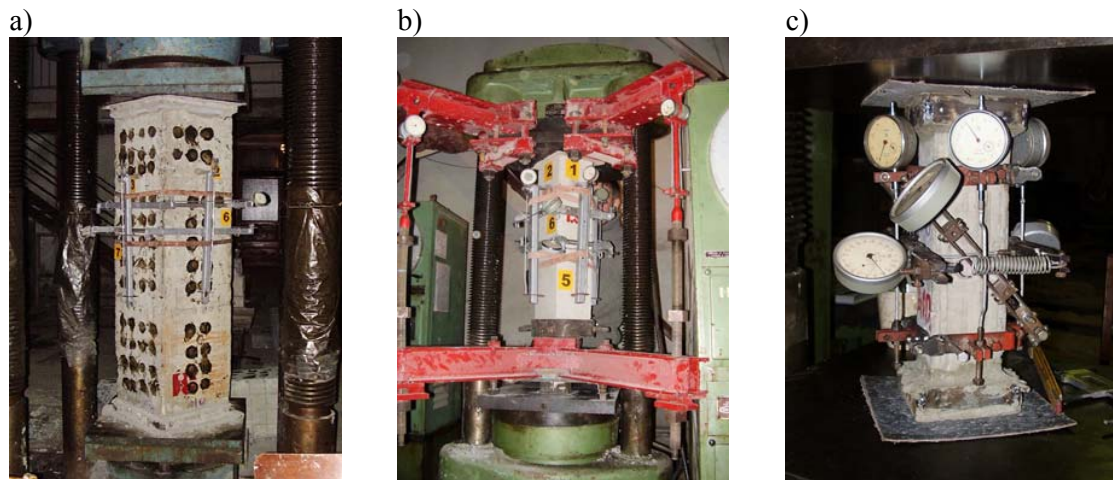


Figure 4: Testing of : a) experimental column fragment (400×400×1500...1700 mm ; b) reference samples (150×150×600 mm) ; c) sawn out sample (70×70×310...320 mm)

4.2 Results

The variations of compression strength R and modulus of elasticity E_b along the 3-rd (middle) layer (Fig. 3a) of concrete column fragment, averaged between samples with similar location in fragment's body (marked correspondingly Fig. 3e) are presented in Figure 5. Strength as well as modulus of elasticity decreases from the middle (core) sample 'A' to the sides and the least values are obtained in the corner samples 'F'. To our regret, during the testing of the first 'not-sawn' column fragment difficulties of precise stabilization of the load occurred. Slight deviation of the load during keeping one load value was permissible enough for getting reliable data on strength and utmost compressibility (Table 1) but disabled us to correctly determine modulus of elasticity according to demands of [15].

The change of these characteristics in vertical direction between corner samples of the lower and upper layers is shown in Figure 6. Both compression strength and modulus of elasticity are higher in the lower layers than in upper ones.

We may see that in cross-sectional direction (Fig. 7a) strength and deformation characteristics of some samples from corners and sides appeared to be lower than characteristics of the whole unsawn column fragment and corner samples even lower than averaged characteristics of the reference samples. In much the same way the characteristics of corner samples in vertical direction change (Fig. 7b). Higher values are at the bottom layers of the column fragment and lower at the upper ones.

5. CONCLUSIONS

Conducted investigation confirmed the existence of essential inhomogeneity of concrete by volume (body) of vertical constructions. Change of the velocity of ultrasonic waves passing closer to the sides and especially corners in transversal direction of the fragments under consideration testifies that the density decreases in external layers and that it increases downright. The reason of such a change in transversal direction is possibly range of factors

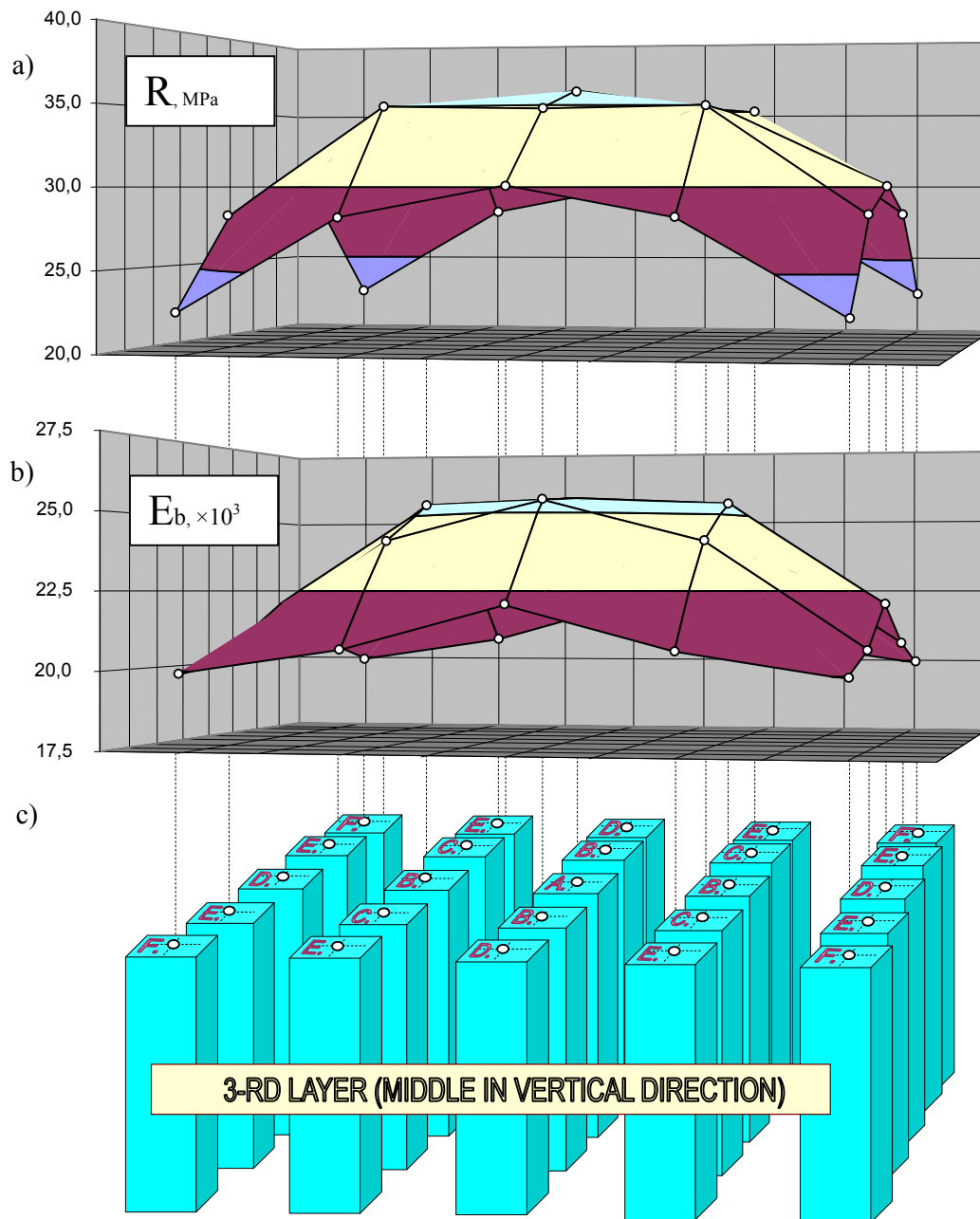


Figure 5: Compression strength - R (a) and modulus of elasticity – E_b (b) distribution across the 3-rd (middle) layer (c) of concrete column fragment, averaged between samples with similar location in fragment’s body (marked correspondingly) (Fig. 3e).

connected with conditions of concrete curing. Thus internal layers of concrete are subjected to the influence of triaxial compression, what results in some compaction of structure (density is higher). External layers, in turn, are subjected to the terms of uniaxial tension caused by the shrinkage of concrete, what leads to not only decrease of the concrete density but also to the occurrence of micro cracks, and as the final result to decrease of strength and change of deformation characteristics of concrete. Moreover, the change of velocity of ultrasonic waves passing also may be caused by the different humidity of internal and external zones of column fragment in transversal direction. The difference of mass humidity obtained in this work supports this idea.

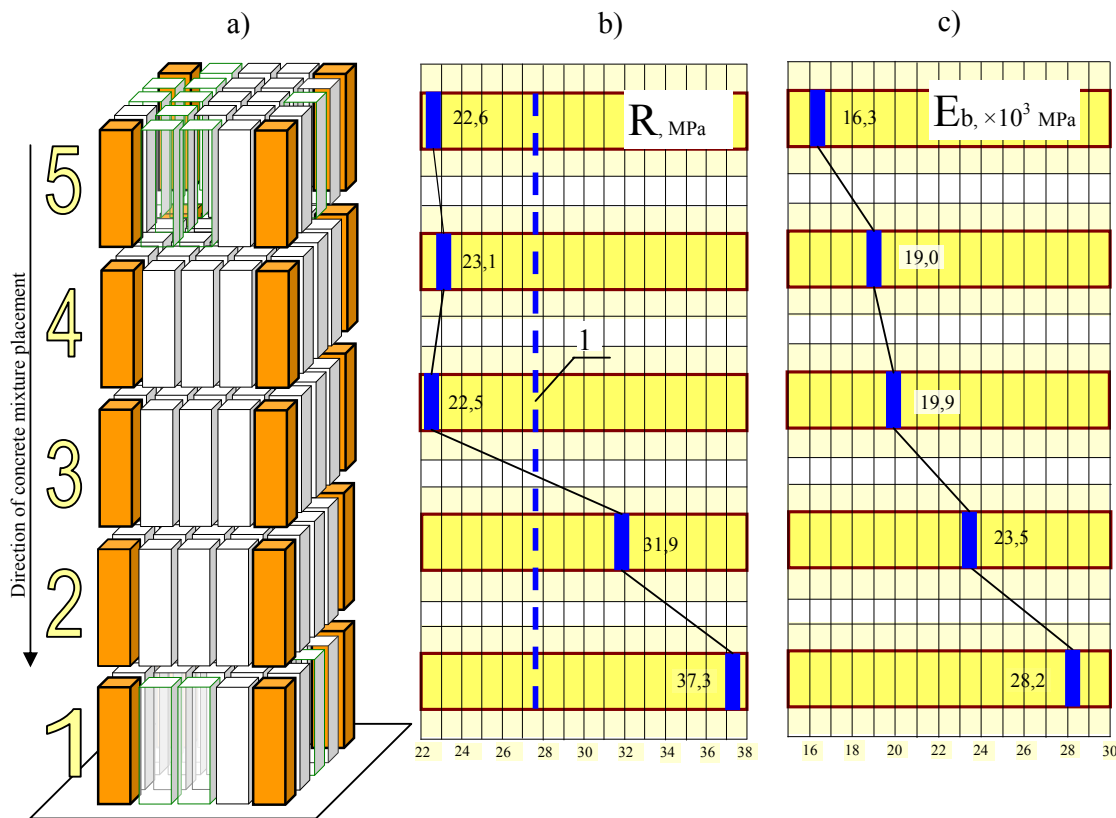


Figure 6: Variation of compression strength – R (b) and modulus of elasticity – E_b (c) of the corner samples ‘F’ (a) in vertical direction; 1 – compression strength of unsawn column fragment (Fig. 4a)

Concerning the influence of inhomogeneity of concrete and of internal stresses in a structure onto strength then we verified that during axial loading of the column fragment (vertical member) at first in the work only internal layers joined (which are already subjected to some internal compressing stresses) and then, at some stage, external layers and finally corner zone concrete (as the internal tensile stresses become changed by common compressive stresses) are joining in the work.

In this way, the change of strength and deformation characteristics of concrete of different zones of vertical members depends on the degree of the concrete zone remoteness from the core zone (the influence of humidity and degree of compression of internal layers by external ones) and on degree of openness of different zones (in conformity to sawn out samples-prisms – on modulus of the open surface of their own and in some extent of neighboring samples).

The change of strength and modulus of elasticity throughout the height of the column fragment (Fig. 6b,c) is apparently the result of concrete mixture disintegration during its placement and compaction, carrying into more high concentration of coarse aggregate (granite crushed stone 5-20 mm) in the lower part of vertical member (1 and 2 layers). The compression strength of sawn out corner samples of 3,4 and 5 layers turned out to be lower than the same of the whole concrete column fragment and rather lower than the same of corner samples of 1 and 2 layers in vertical direction.

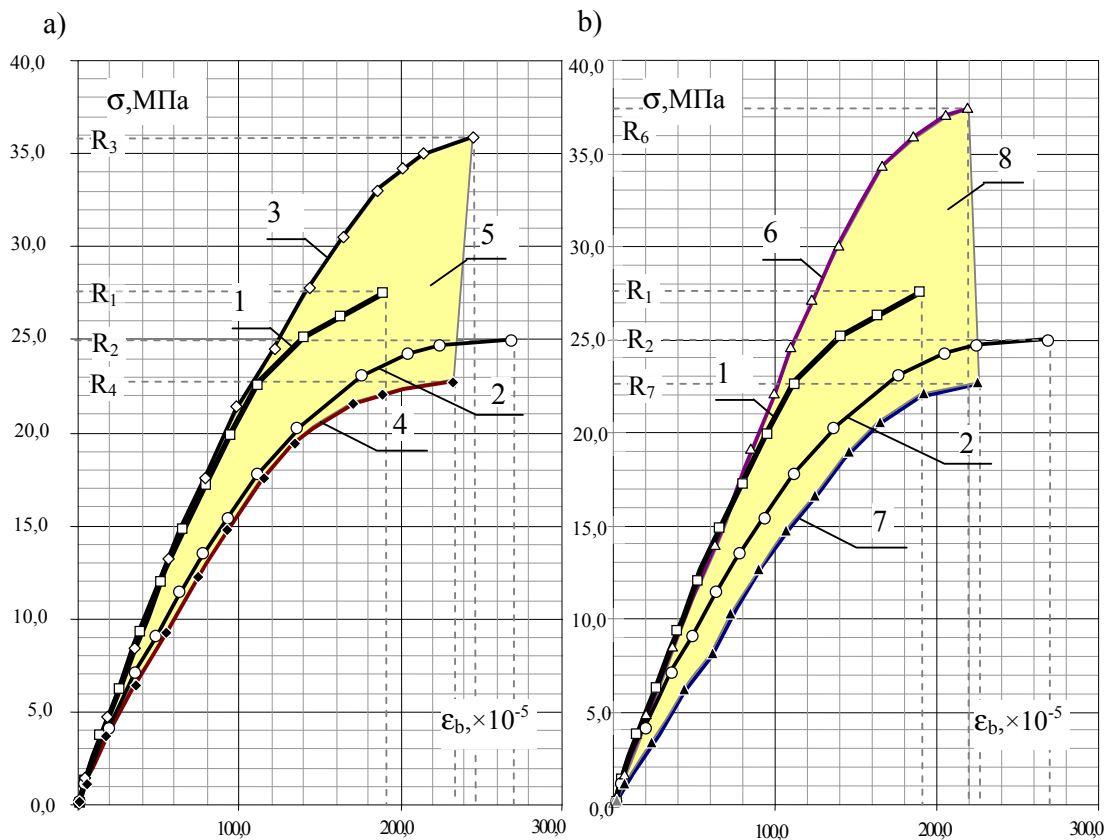


Figure 7: Plots of deformations during compression test a) across the 3-rd layer b) in vertical direction, where: 1 – column fragment; 2 - reference samples, averaged; 3 – ‘A’ (core) sample (Fig. 3e,5c) of the 3-rd layer; 4 – corner samples ‘F’ (averaged) of the 3-rd layer; 5 – area of variations within the 3-rd layer; 6 - corner samples ‘F’ (averaged) of the 1-st (lower) layer; 7 - corner samples ‘F’ (averaged) of the 5-th (top) layer; 8 - area of variations within the corner samples of all 5 layers of the concrete column fragment; R1 ...R7 – compression strength of 1...7 curves correspondingly.

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2008 Architectural Engineering Conference

Rehabilitation and Strengthening of Concrete Structures Using Carbon Fiber Reinforced Polymer

By

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Abstract:

This paper gives the presentation of the author's experience with the use of carbon fiber reinforced polymer (CFRP) to rehabilitate and strengthen existing and new concrete structures. The various methods of strengthening of structures are first described. These include the external post-tensioning using high tensile bars or strands, the reinforced concrete jacketing, and the reconstruction of structural members.

The paper describes the advantages of CFRP in its high tensile strength of 100 ksi minimum, its constructability, and its environmental friendliness. The process does not require demolition or closure of bridges or buildings that are being strengthened, as compared to some other methods. With good products and skills, the procedure of strengthening using CFRP wrap is simple and quick.

The author presents his experience in evaluating and selecting an appropriate method for several cases under different conditions. These are the strengthening of existing reinforced concrete circular columns in existing bridges overseas and in the US, the strengthening of rectangular concrete columns in new building, and of new post-tensioned concrete beams, both by concrete jacketing and CFRP wrapping. Suggested procedure for application of CFRP is given, and final precautionary notes are given in the conclusion.

Key Words:

Concrete, Structure, Repair, Rehabilitation, Strengthening, Epoxy, Grout, Injection, Patch, Carbon Fiber, Composite, Reinforced, Jacket,

Introduction

This paper presents the author's experience with the repair and strengthening of reinforced and prestressed concrete structures using carbon fiber reinforced polymers (CFRP). This composite material has been used more increasingly in structural repair and strengthening due to its high tensile strength and the ease of its application. The paper demonstrates several situations where CFRP wraps are used for strengthening of bridge columns, building columns, and post-tensioned concrete beams. The reasons and strategies for selecting CFRP wrap over other available methods are discussed.

The paper emphasizes practical approach of using CFRP for practicing engineers and architects. Suggested procedure of application and specification of CFRP are given.

Concrete Repair and Strengthening

Concrete repair and strengthening may be required both in old and new concrete structures. In practice, old concrete structures are repaired and strengthened to prolong their service life. Many old bridges, for example, need to be strengthened for the present day increased live loads. The causes and types of deterioration and defects, methods for determining degrees of deterioration of the existing structures, and various conventional concrete repair methods are given in Reference 1.

In new construction, there were cases where concrete cracks formed due to mistakes in the design and/or construction. For example, concrete might not gain strength as expected, or required reinforcing bars were not installed properly, or prestressing forces were not applied correctly, or the forms were removed prematurely, etc.

The common forms of defects are crack formation, chipping and spalling of concrete. Cracks in concrete may be caused by one of or a combination of many factors. Some cracks only affect aesthetics but some may also affect strength and safety of structural members.

In general, fine internal cracks are repaired by epoxy injection. Area cracks are usually repaired by removing cracked or poor concrete until sound concrete is encountered, and the empty space then patched with high strength mortar or epoxy. The injection and the patching act to restore the concrete to the same previous conditions. They do not increase the strength of the existing members.

When strengthening is required, normally additional work will have to be performed. Partial reconstruction by removing and replacing with new members may be considered. The method generally requires construction staging and temporary supports. Enlarging of member sizes by placing reinforced concrete jacket is another alternative option. Jacketing technique has been commonly used for column and beam strengthening.

Strengthening by adding steel plates, or providing external post-tensioning using high tensile bars or strands have been used for rehabilitation of existing bridges and buildings. Figure 1 shows the picture of a bridge pier cross beam strengthened with high tensile bar post-tensioning.



Figure 1- Strengthening of Bridge Pier Cross Beam by High Tensile Bar Post-Tensioning

Figure 2 shows the picture of the reconstruction of the new pier of a bridge on US Interstate-195. The new pier was constructed on the existing foundation. The reconstruction took over two years to complete.



Figure 2- Reconstruction of New Concrete Pier on Existing Foundation

Role of CFRP in Strengthening of Concrete Structures

There have been many research reports on the use of CFRP in concrete beam and column strengthening. However, there are only few reports of its use in practice. The following sections demonstrate the situations where CFRP has been found to be a suitable material for the required strengthening. The method takes advantage of the high tensile strength of

the CFRP which is in the order of 100 ksi minimum, its constructability and its environmental friendliness requiring no demolition or interruption of services.

Two Cases of Bridge Column Strengthening

Two cases of bridge column strengthening are compared; one using the more conventional reinforced concrete jacketing, and the other using CFRP wrap.

The first case is the rehabilitation of highway bridges in Afghanistan under the sponsorship and financial aid of USAID. The rehabilitation design was carried out in 2004. The bridges were upgraded to support an increased vehicle live load of AASHTO HS20-44 by 30 %. This increased live load followed the live load design criteria recommended by the author in 1986 for another expressway project in Thailand (References 2 & 3).

In the Afghanistan project, there were several bridges whose substructures could be retained but requiring strengthening of the existing circular concrete columns for the increased live load. Reinforced concrete jackets, 150 mm thick, was used for the strengthening. The strategy was to keep the design simple and to utilize basic construction materials that were available in the region, considering the skill of the available work force and possible limited degree of supervision.

The second case is the rehabilitation of an interchange in northern Virginia, whose strengthening design was carried out in 2001 for the new AASHTO live load. In this project the circular concrete columns of the existing piers were strengthened by repairing and applying CFRP casing around the bridge columns. The details are similar to those shown in Figure 3.

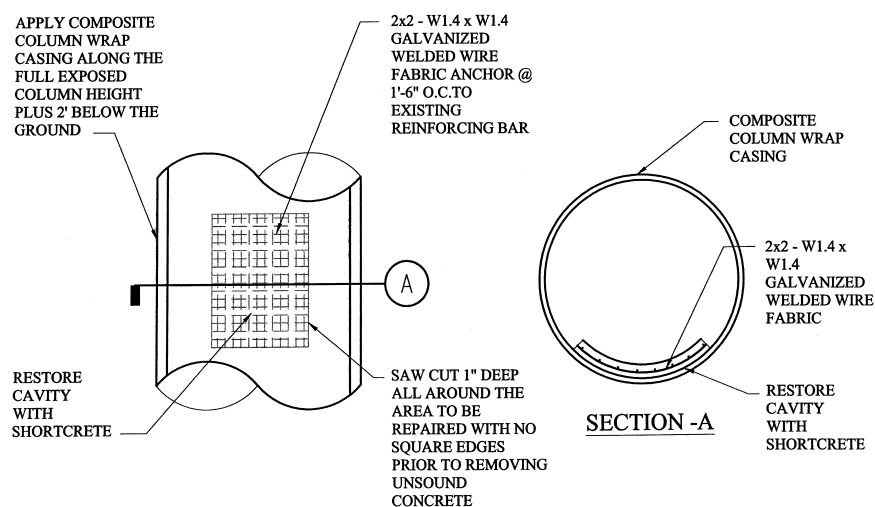


Figure 3 - Repair and Strengthening of Concrete Bridge Columns by CFRP Casing

The design provided performance specification and requirements. The CFRP casing around the column was specified to provide confined strength equivalent to that of reinforcing bar # 4 closed ties (grade 60 ksi) spaced at 6 inches on center.

Strengthening of New Rectangular Concrete Columns by CFRP

This section highlights the use of non-destructive test of new column by ground penetration radar technique, and the use of CFRP for a rectangular column.

In a recent project, there were situations where newly built concrete columns were found by using ground penetration radar test, to have closed-tie reinforcing bars for the columns installed incorrectly. Some ties were missing from the top of the rectangular concrete pedestals that supporting a 4-story building. After careful evaluation of various factors, the strengthening of the pedestals by CFRP wrapping was adopted. Figure 4 shows the CFRP wrap at one of the pedestals.



Figure 4 - CFRP Wrap around New Rectangular Concrete Pedestal.

The selection of CFRP wrap was based on the need to meet the following constraints.

1. It was desirable to complete the construction and open the building for use on time. Construction delay due to the required repair would create problems to all parties involved.
2. Two independent testing agencies were asked to perform the ground penetration imaging of the embedded reinforcing bars in all pedestals. The results, after careful interpretation of the images, confirmed that the vertical bars were installed correctly, the column ties were either missing or placed at a larger spacing than specified.
3. Concrete cylinder tests indicated the compressive strength of the concrete to be well above the minimum specified 28-day strength. In addition, the cross sectional dimensions of the pedestal were found to be larger than the specified dimensions. The calculated total service load maximum compressive stresses in

the pedestal were found to be less than 1/3 of the cylinder strength. In addition, the pedestal heights were short and stocky.

With above factors considered, it was decided that CFRP wrapping to provide additional confinement for the concrete would be adequate. The method was adopted and was found to be so simple that the work was completed satisfactorily within a few days.

Two Cases of Post-tensioned Beam Strengthening

Two methods of strengthening the torsional resistance of post-tensioned concrete beams by reinforced concrete jacketing and by CFRP wrapping are shown in Figures 5 & 6, respectively. In a recent project, concrete cracks appeared near mid span of post-tensioned concrete beams after removing the shoring and stripping of the forms. The post-tensioned beams were subjected to high torsional moments due to the complex framing configuration of the beam and slab system subjected to high dead and live loads.

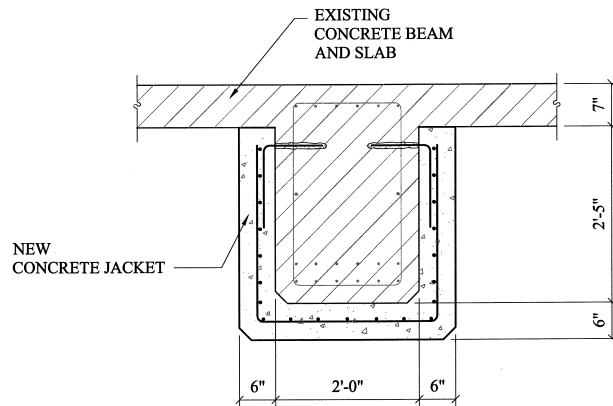


Figure 5-Strengthening of Post-tensioned Concrete Beam by Reinforced Concrete Jacket

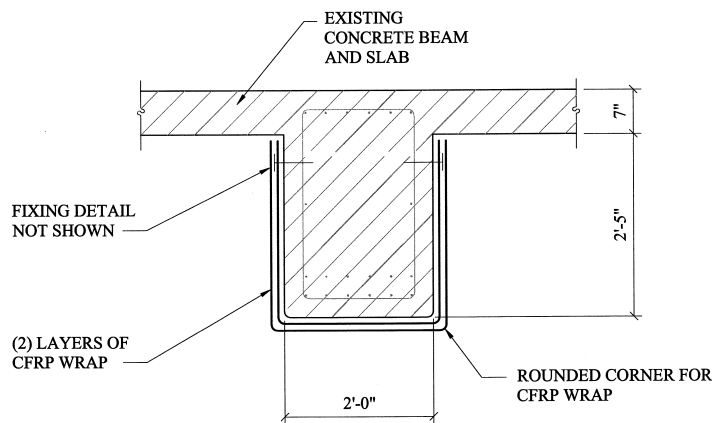


Figure 6- Strengthening of Post-tensioned Concrete Beam by CFRP Wrap

These fine cracks were allowed to stabilize before proceeding with the repair. These cracks were subsequently repaired by epoxy injection, and then were strengthened by reinforced concrete jacking as shown in Figure 5. The jacket increased the beam depth by 6 inches.

For those beams whose depth could not be increased due to vertical clearance limitation, the alternate method using CFRP wrapping as shown in Figure 6 was adopted. Two layers of CFRP wrapping were used along the full length of the beam with carbon fibers orientated in the longitudinal direction in the first layer, and in the transverse direction in the second layer. The CFRP wraps were later painted over to the same color as the surrounding concrete.

Discussion

The above presentation gave case studies of using CFRP to strengthen concrete structures. The requirements for strengthening may be due to deterioration, or change in the applied loads/design criteria, or error in design/construction. The advantages of CFRP are its high tensile strength, as high as 100 ksi and over as compared to 60 ksi of the reinforcing bars, its relatively ease of application requiring no demolition of the structures, resulting in cost and time savings. The nominal dimensions of the members wrapped with CFRP remain practically unchanged, plus it can be painted over to the same color as the surrounding materials. This last factor may become significant consideration when there is restriction in the available width or head room and the need to maintain uniformity (of size) for aesthetic purpose.

The success of using CFRP depends on two important executions, first selecting the right situation for using CFRP, and second selecting the right manufacturer and installer of CFRP wrap. The examples and concepts used in the selection of methods have been given above. Procedure and specifications to achieve proper application are given in the next section.

Suggested Procedure and Specifications

The composite column wrap casing shall consist of carbon fiber fabric and epoxy resin and shall meet the following requirements:

Properties at 72 deg F +/- 2 deg F	Values	ASTM Test Method
Ultimate Tensile Strength	100 ksi (min.)	D3039
Ultimate Elongation	0.8 % (min.)/ 1.8 % (max.)	D3039
Tensile Modulus	9000 ksi (min.)	D3039
Fiber Volume	50 % (min.)	D3171

The Contractor shall also submit results of tensile strength from ASTM D3039 test performed on material samples exposed to the following environments. The tensile strength under those conditions shall be at least 90 % of the material properties at 72 deg F +/- 2 deg F.

Environment	Exposure Conditions	Test Duration
Water	100 % humidity, 100 deg F +/- 2 deg F	3000 hours
Salt Water	Immersion at 73 deg F +/- 3 deg F	3000 hours
Alkali	Immersion in Ca(CO ₃) at 73 deg F +/- 3 deg F and pH 9.5	3000 hours
Dry Heat	140 deg F +/- 5 deg F	3000 hours

Construction requirements of CFRP wrap (casing) include the following:

1. The concrete surface shall have been repaired, epoxy injection crack sealing performed and spray-on corrosion inhibitor applied, prior to installation of composite column casing.
2. Concrete surfaces to receive the composite wrap casing shall be free of fins, sharp edges and protrusions that could damage the fibers or cause voids or depressions behind the installed (column or beam) casing.
3. Voids and depressions, defined as volumes greater than 1" diameter by ¼" deep, and other uneven surfaces to receive the composite shall be filled with epoxy filler or other material as approved by the Engineer.
4. The contact surfaces shall be completely dry at the time of application of the composite. Newly repaired or patched surfaces shall be cured a minimum of 7 days prior to the application of the composite.
5. All surfaces shall have foreign materials removed and be broom cleaned or sand blasted as directed by the Engineer. Concrete shall be protected from rainfall & dirt in the interim period.
6. One prime coat of the manufacturer's epoxy shall be applied prior to wrapping the surface with the composite. Primer shall be allowed to become tacky to touch.
7. The Contractor shall apply saturated fabric to concrete surface, using methods that produce a uniform, constant tensile force that is distributed across the entire width of fabric. The system shall be placed by methods that assure a uniform, even final appearance, without any puckered edges & oil pockets.
8. Saturation of the fabric shall be performed and monitored according to the specified fiber-resin ratio. Fabric shall be completely saturated prior to application to contact surface in order to assure complete impregnation of fabric.
9. The ambient temperature shall be between 35 deg F and 100 deg F at the time of saturating the fabric and applying the composite. The composite shall be applied when the relative humidity is less than 85 % and the surface temperature more than 5 deg F above the dew point.

10. Subsequent layer(s) of the composite may be applied immediately while the first layer's resin is still curing or after it has cured. Entrapped air between each layer shall be released or rolled out before the epoxy resin sets.
11. A minimum lap length of 6 inches shall be used at all necessary overlaps in the longitudinal direction of fabric. Subsequent layers shall be applied continuously or spliced, until designed number of layers is achieved. Adjacent sections shall utilize a butt joint. Fiber shall not deviate from a horizontal line more than ½ inch per foot.
12. The system shall be protected from rainfall or water for a period of at least 4 days following installation.
13. During the installation of the composite column casings, the installer shall maintain a Daily Wrapping Log. This log shall provide material traceability and process records of each column casing installation and shall include the following information:
 - a. Casing identification with project name, contract number, and installation date.
 - b. Materials information including product name, description, date of manufacture, and lot or batch numbers.
 - c. Fabrication, inspection and verification data for the manufacturing and construction operations including square footage of fabric & volume of epoxy used each day, number of layers, thickness measurements, ambient temperature and humidity readings at the beginning, middle and end of each casing installation shift.
14. The cured composite casing shall be inspected for defects consisting of external abrasions or blemishes, delaminations, voids, chips, cuts, foreign inclusions or depressible areas. All defects shall be repaired as determined by the Engineer.
15. One sample per lot or batch of composite material used on the project shall be provided to the Engineer to verify compliance with the Materials Requirements of this Specification. The sample shall be at least 12 inches x 12 inches and 1 ply thick. It shall be fabricated in the same manner as the field-installed column casings and cured straight and flat under moderate tension and pressure.
16. Exposed surfaces of the composite column wrap casing, including surfaces 2 feet below ground, shall be cleaned and epoxy-coated with a minimum of two coats of type EP-3T. The coated composite shall be protected from rainfall or water for a minimum of 24 hours.

Conclusion

This paper described the author's experiences with CFRP for the rehabilitation and strengthening of concrete columns and beams. Nondestructive imaging of embedded reinforcing steel by the surface penetration radar technique could be used to evaluate the old and new concrete construction. Engineers have to evaluate the suitability and constructability of using CFRP on case by case basis. The evaluation and comparison with other methods have been shown in this presentation for existing concrete circular

columns for the bridge piers, for the new rectangular reinforced concrete columns, and for post-tensioned concrete beams. The other methods mentioned in the paper were the partial reconstruction to replace the defective parts, the use of external prestressing with high tensile bars, the concrete jacketing adding reinforcing bars and enlarging of existing columns or beams. For successful application, CFRP has to be selected for the right situation, case by case, and has to be applied properly by experienced operators and manufacturers. Then, it has been shown that, the use of CFRP wrap can be just as effective as other methods, yet much simpler and faster to apply. Finally, it must be noted that the purpose of this presentation is to share the author's experiences with other architects and engineers. It remains the responsibility of those professional to evaluate and judge for themselves and for their clients the suitability and applicability of the adopted method of repair and strengthening on case by case basis.

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EXPERIMENTAL BEHAVIOR OF TRIANGULAR LAMIANTED GLASS LITES

By

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Abstract

As a look of modern architecture design, the use of glass as curtain walls and window systems with different shapes has been quite up worldwide. The glass panel is not used only as a member for closing the space but also as a structural member for carrying loads such as wind pressures. As a result of using glass, new questions to the designer have been regulated in standards. Recently, a special test facility developed at Minufiya University, Egypt where laminated glass laboratory triangular specimens were tested under uniform lateral loads and simply supported boundary conditions. A previously reported model that used finite element methods with von Karman assumptions solved the glass ply problem. Both experimental and theoretical results compared and presented.

Keywords: Laminated glass; Finite element; Stress analysis.

Introduction

Laminated glass (LG) used in architectural glazing is comprised of two or more pieces of glass bonded together by a plastic interlayer (usually polyvinyl butyral-PVB) under heat and pressure. Glazing panels on modern buildings, which shatter under wind storms or blast loads, are the main cause of injury due to the high-velocity fragments produced. Upon failure in LG, the glass pieces tend to adhere to the plastic interlayer, PVB. That is why LG is widely used due its safety as construction material. LG unites behave nonlinearly under high wind load when their deflections exceed about 3/4 of their thickness. The new standards for curtain wall and window systems have made it mandatory to include the effect of large deflection in the design of glass panels. In general, glass plies experience large lateral displacements (compared to their thickness) when subjected to lateral pressures. Due to the development of membrane stresses, the behavior of these plies becomes nonlinear. The Mindline plate theory using von Karman assumptions have been found to be satisfactory as representing this behavior. In this paper, two specimen geometries, selected to represent used window areas and aspect ratios of triangular LG, were employed in the destructive tests. One thickness was employed, 6 mm (1/4 in.), and used as the total thickness of LG specimens. Samples are tested to failure using constant rates. Test results are reported herein. A very sophisticated finite element model (FEM) reported before by El-Shami and Norville, 2002 and modified

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by El-Shami et al., 2007 is used to analyze these samples. Direct comparisons of the results between experimental and FEM are reported and discussed herein. This is considered as initial step towards new design methodology for glass window with triangular shape.

Previous Experimental Research

Quenett (1967) reported tests on layered glass where bending and impact loads were applied. From theoretical analysis, he noticed that as the interlayer thickness decreases, shear modulus increases. In destructive testing, Pilkington ACI (1971) compared monolithic glass strength to the strength of LG specimens made with sheet and float glass. It found that, at normal temperature, LG lite specimens exhibit the same strength as monolithic glass lite specimens having the same rectangular dimensions and glass thicknesses. Hooper (1973), tested glass beams in four points loading with varying temperatures and interlayer harnesses. He found that the load duration was a primary factor controlling bending resistance of LG.

Linden et al. (1984) conducted non-destructive testing on different geometries to study the effect of interlayer thickness on strength and deflections. They found that LG lite specimens tested at room temperatures were 22 % stronger than monolithic lites having the same dimensions and nominal glass thicknesses. Reznik and Minor (1986) destructively tested three sizes of LG rectangular lite specimens Their main conclusion was LG lite specimens having twice the nominal glass thickness of monolithic lite specimens displayed strength greater than or equal to twice the strength of the monolithic lite specimens.

In 1990, Norville tested two geometries of LG to failure under uniform lateral loads. The conclusions from this research indicate that glass ply thickness is the governing factor for the load resistance of LG. Norville et al. (1993) conducted tests to determine load resistance of heat strengthened for both monolithic and LG. They found that the load resistance of LG with heat strengthened glass plies was approximately 2.5 times that of annealed monolithic glass. Norville and Mackay (2001) tested rectangular LG with different aspect ratios to investigate the behavior of LG subjected to uniform load at elevated temperatures. Their main conclusion was that the maximum principal tensile stresses in the LG decrease with increasing the temperature.

Finite Element Model

The authors employed in this paper a nonlinear finite element performed by El-Shami and Norville, 2002 and El-Shami and Norville, 2003. This finite element model was tested by comparing its results with those of previous researches. Based on that, the authors satisfied that the finite element model representing the behavior of LG for rectangular and trapezoidal (with one inclined edge only) shapes. The formulation of this model and solution techniques were published earlier by El-Shami and Norville, 2002 and El-Shami and Norville, 2003. The boundary conditions are

specified as those for simply supported ply without any in-plane edge restraints. To ensure such boundary conditions for the inclined edges, as in the present case of triangular LG, the authors used penalty approach (Cook et al, 2003) to modify the FEM (El-Shami et al., 2007). This modification ensures simply supported boundary conditions for all nodes located along the inclined edges. Figure 1 shows an inclined edge. The rotation in the direction of M is zero (simply supported). This condition can be written as:

$$\alpha\theta_x + \beta\theta_y = \theta_M \tag{1}$$

Where

$$\alpha = \sin \phi, \beta = \cos \phi$$

θ_x , θ_y and θ_M are the rotations about X, Y and M axes, respectively

In the penalty approach (Cook et al., 2003), the total potential energy function is augmented by the strain energy of a fictitious spring with a very large spring constant **C** in the direction of the prescribed displacement. So the total potential energy function (Π_M) can be written as:

$$\Pi_M = \frac{1}{2} \{u\}^t [K] \{u\} + \frac{1}{2} C \nabla^2 + V \tag{2}$$

Where $\{u\}$ denotes the displacement vector, $[K]$ denotes the global stiffness matrix, V denotes the potential energy, and ∇ is the prescribed condition such as:

$$\nabla = \alpha\theta_x + \beta\theta_y - \theta_M \approx 0 \tag{3}$$

Differentiating Π_M with respect to both θ_x and θ_y , we get the necessary coefficients of the global stiffness matrix at the prescribed node i . The only change in the global stiffness matrix will be with the terms associated with θ_x and θ_y as following:

$$\begin{bmatrix} \vdots & \vdots & \\ \cdots & K_{5i-4,5i-4} + C\alpha^2 & K_{5i-4,5i} + C\alpha\beta & \cdots \\ \cdots & K_{5i,5i-4} + C\alpha\beta & K_{5i,5i} + C\beta^2 & \cdots \\ \vdots & \vdots & \vdots & \end{bmatrix} \tag{4}$$

Where $5i - 4$, and $5i$ are the order of θ_x and θ_y associated with node i (El-Shami and Norville, 2002).

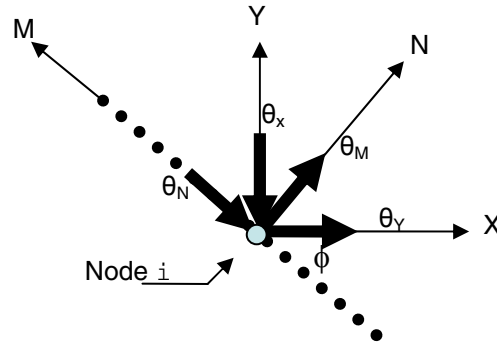


Figure 1

Experimental Model

The Structural department at Minufiya University, Egypt developed the test facility used for this research. It consists of testing chamber, a vacuum system, and data collection system.

Test Chamber

The test chamber forms a triangular box that provides support for test specimens. Two chambers were used (two geometries). Figure 2 shows the test chamber of specimen # 1. Each test chamber conforms to ASTM Standard Test Method E 998-05 (2005). The test specimen mounts on the front of the test chamber to form a vacuum chamber. A test chamber has a 12.0 mm (1/2 in.) steel base plate. 100 mm (4.0 in.) steel channels welded to the base plate, form the support for the glass lite. The authors sized the channel to adequately support the glass during loading and provide sufficient depth to accommodate large deflections expected during failure of the specimen. One side of the test chamber has 12.0 mm (1/2 in.) diameter opening to allow air to be evacuated from the test chamber. The criteria for edge boundary conditions are to limit lateral movement and to allow rotational and in-plane movement.



Figure 2. Test Set Up (Specimen # 1)

Vacuum Accumulation System

The vacuum accumulation system consists of vacuum pump, a loading control system, and associated piping. The loading valve allows the test operator to load specimen at a desired rate and hold the pressure for an indefinite time (Figure 3).

Data Collection

The data collection system consisted of measuring devices and associated wiring and tubing. The data collection system monitored and recorded the differential pressure between the test chamber and the atmospheric pressure and the deflection at the center of the specimen during testing.



Figure 3. Vacuum Accumulation Pump

A pressure transducer measured the differential pressure across the specimen. The authors calibrated the pressure transducer by applying a known pressure to the transducer and confirming the output. They accomplished the calibration at varying pressures within the range of the instrument to assure accurate pressure reading throughout each test.

A Trans Tek linear variable deflection transducer (LVDT), model 0246, measured deflections (see Figure 4). The authors calibrated the LVDT by recoding the voltage signal at known deflections throughout the range of the instrument and solving for the scale factor or voltage to deflection conversion. To convert the signal from voltage to millimeters the following equation is applied:

$$\delta = (V + OC).SF \quad (5)$$

Where δ denotes deflections (mm), V denotes instrument voltage signal (volts), OC denotes offset constant corresponding to excitation voltage (volts), and SF denotes scale factor (mm/volts)

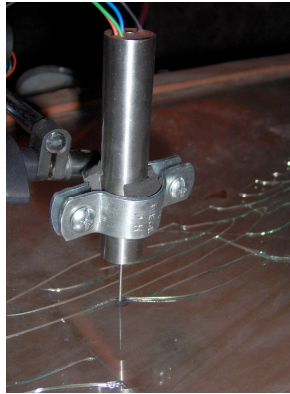


Figure 4. Deflection Transducer

Experimental Procedure

A test procedure delineated a series of steps for testing. The testing procedure remained constant for all test samples except for the loading rate. These steps can be summarized as following:

- 1- The authors measured the overall thickness of the specimen four times at random locations, then calculated the average total thickness and recorded it.
- 2- The authors determined the tin side if there is one and be sure to place it down in the frame. (Tin side in tension). Placing the tin side in tension represented a worst case loading condition.
- 3- Next, they carefully moved the specimen from its shipping crate to the test frame. They did this to be certain that the frame was clean and the neoprene was clean and in place.
- 4- The authors mounted the LVDT on the support arm and attached the arm to the test frame. The LVDT was then positioned properly in relation to the specimen. Next, they checked the excitation voltages from the LVDT.
- 5- Next the authors connected the vacuum accumulations system to the test chamber, and they got ready to test the specimen. The test operator loaded the specimen to failure by adjusting the pressure in the test chamber with the loading control system. Following failure of the specimen, he returned the test chamber to atmospheric pressure.
- 6- During the test operations the authors recorded the output voltage of LVDT corresponding to each load increment. It should be noted that the several load cycles were done before the failure of the specimen. This was to verify the pressure-voltage at each load increment.
- 7- After failure, the authors identified and recorded the fracture origins for the inside and outside glass.
- 8- Finally, the authors disposed the specimen, cleaned the test area, and prepare for the next test

Experimental Data

These experiments involved two different sizes of LG. The interlayer material for the LG was PVB which was 1.524 mm (0.06 in.) thick and had a shear modulus of 689.476 KPa (100 psi). The overall dimensions for both test specimens are shown in Figure 5. The overall thickness of both specimens was 6 mm (1/4 in). The modulus of elasticity of the glass plies was tested to be 10.15 E+3 psi (70.0 GPa) with Poisson’s ratio of 0.22. Uniform pressure was applied on the specimens until failure. The displacement transducers were used to measure the lateral displacements at the center of gravity of each specimen. The load displacement relations are shown in Figures 6 and 7, and the results are compared with finite element solutions. The authors observe a good agreement between both experimental and theoretical results.

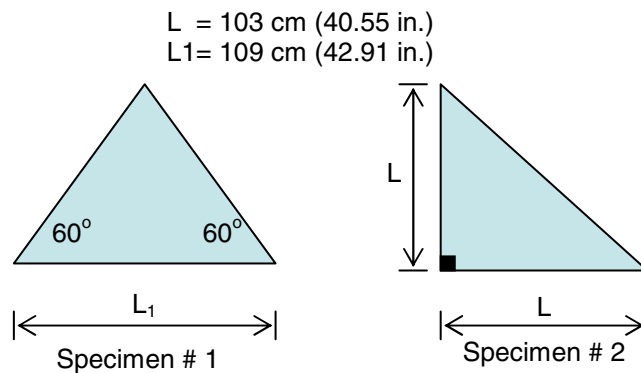


Figure 5. Geometry of Test Specimens

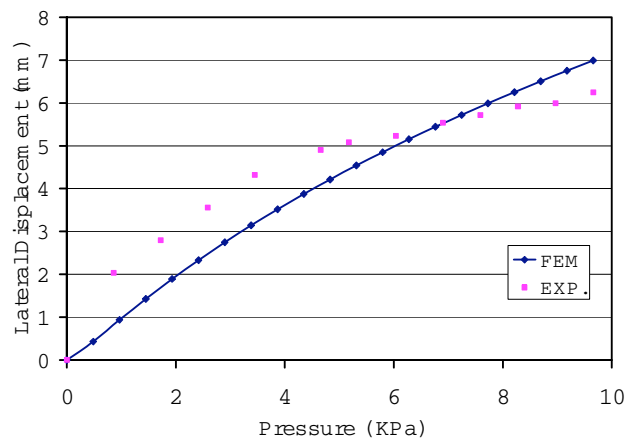


Figure 6. Later Displacement vs Pressure at the Center of Specimen # 1

Figures 8 and 9 present contours of maximum principal tensile stresses on the surface of the each specimen. Due to the symmetry, only one half of the ply is shown in the figures. The values of the stresses are in MPa. The authors observe that the locations of the highest magnitude of the maximum principal tensile stresses are

matching with the fractures. This observation provides a high degree of validation to the FEM (Figure 10).

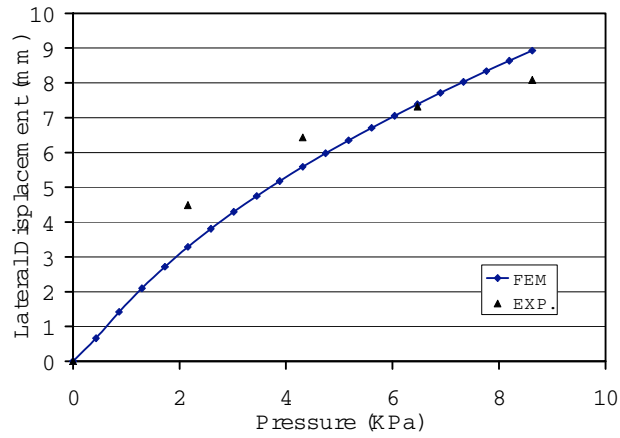


Figure 7. Later Displacement vs Pressure at the Center of Specimen # 2

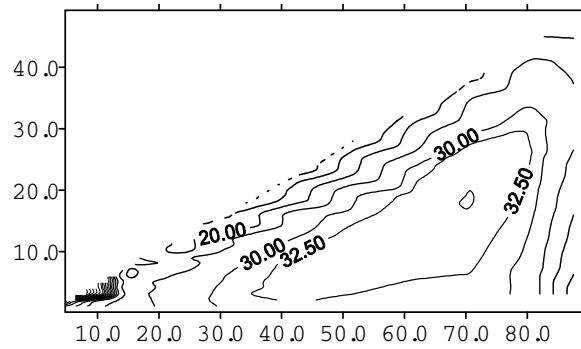


Figure 8. Maximum Principal Tensile Stresses (Specimen # 1)

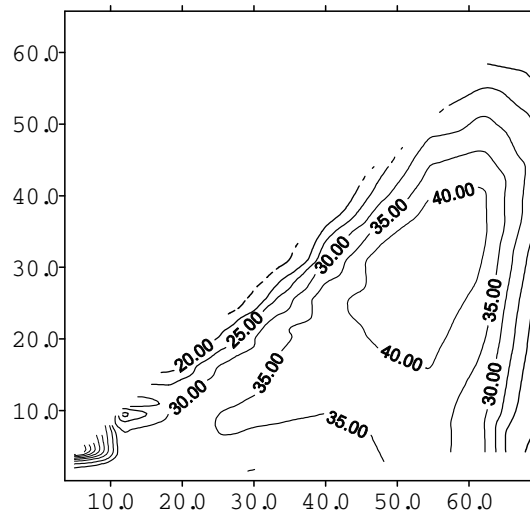


Figure 9. Maximum Principal Tensile Stresses (Specimen # 2)



Figure 10. Failure of the Specimen # 1

Conclusions and Recommendations

This paper presents ongoing work in the development of a design methodology for triangular window glass lites and window glass constructions. The work entails several tasks: destructive testing of triangular wind glass plies, non-linear finite element stress analysis, the use of failure prediction methodology, and finally the formulation of software that will facilitate proper thickness selection based on wind loads. The paper contains preliminary results from each of the first two tasks. Certain aspects from the experimentally obtained failure data, specifically experimental load-center deflection relationships and locations of fracture origins lend credibility to the finite element technique. The comparison between the results of the FEM and the results of the experiments on LG is good. More experimental research is necessary. The fact that the majority of fractures originated in higher stress regions of the lites lends further credibility to the FEM.

Acknowledgments

The authors conducted the research described herein at Material Laboratory, Faculty of Engineering, Minufiya University, Egypt (Spring 2007). The authors gratefully acknowledge the Dr. Grach factory, 10th of Ramadan city for providing test specimens for this research.

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Parking Garage Security and Lighting

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Abstract

Historically, parking structures are perceived to be dark and dangerous places. Proper design of the passive security features within a garage, combined with appropriate active security measures, can create a welcoming, safe, and secure space. With features that deter criminal activity and promote safety, a garage will be well- received by the user, the community, and the owner.

An explanation of security considerations for a parking garage is presented herein, including a discussion of garage lighting. Other security considerations include openness, visibility, access control, perimeter control, and electronic security measures (e.g. CCTV, intercoms, etc.). Active security, primarily consisting of manned patrolling, is also briefly discussed.

Introduction

There is a common perception that parking structures, and especially underground parking garages, are unsafe and uncomfortable places. In most peoples' minds, a parking garage is a dark, enclosed space full of hiding places for vandals and predators. This idea is perpetuated by the news media, in movies, and on TV. Consider the last police drama or spy movie that you watched; the chances are good that there was a scene depicting criminal activity in a parking garage. Unfortunately, this portrayal is not unfounded. According to recent U.S. Department of Justice statistics, parking lots and garages rank as the second most-frequent location for non-violent crime and third for violent crime.

Beginning in the mid-1980s, the parking design and operations community began to acknowledge that the driving public perceived parking structures to be unsafe and unsecure. The terrorist attacks of September 11, 2001 brought security to the forefront and reinforced the need for a comprehensive approach to security in every facility. Over the past twenty-five years, parking professionals have developed a variety of techniques and tools in an effort to continually improve the security of the patrons and staff of parking facilities. This discussion summarizes security considerations and how to address them in the design process and once a facility is in operation.

Security versus Safety

One initial clarification must be made regarding the terms "security" and "safety." In the parking design industry, "safety" typically implies life safety considerations such as the structural integrity of the building, pedestrian guardrails and vehicular barriers, fire egress and suppression, and similar issues. By contrast, "security" generally implies the passive and active measures employed to prevent malicious and dangerous human activity within a facility. The focus of this discussion is limited to "security" design considerations.

Security measures are usually defined in terms of "passive" or "active." Simply put: active measures are those that focus on a human response, such as security patrols, and passive measures are those that do not.

Passive security measures

Passive security measures are those that are inherent in the design of a facility. In other words, they are a physical part of the parking structure, and, as stated previously, do not require a human response. Passive security design centers on the promotion of openness and visibility within the structure, as well as control of access into the facility. The common-sense approach is that those with criminal intent are less likely to commit a crime if they know that they are likely to be seen doing so.

Openness

Designing for visibility and openness begins with the structural framing. Long-span or clear-span construction, usually consisting of a column spacing around 60 feet, is an economical means of reducing visual obstructions by essentially eliminating half of the columns in the building. Shear walls can be restricted to the exterior of the building, or, alternatively, replaced by open shear frames. Precast “lite” walls are a good example of this. The intent is to eliminate as many potential hiding places as practical.

[picture]

Maximizing the flat floor area, versus ramped floors, improves lines of sight as well. If practical, ramps can be located along the exterior of the building to allow contiguous flat floors in the remainder of the garage.

Increasing the floor-to-floor height improves the perceived openness, as does the incorporation of light wells and areaways.

[picture]

Following the same principles, stairs, lobbies, and elevators can be designed for openness and visibility. Locating the stairways at the exterior of the building, faced toward the public view, allows for visibility into and out of the space. Glass curtain walls enclosing the stairs are an effective means of promoting visibility while protecting patrons from the weather. Similarly, elevator cabs and shafts can be constructed with glass walls to allow for visibility.

[picture]

Glass-enclosed stairs and elevators also promote pedestrian way-finding by allowing pedestrians to orient themselves to their destination before they leave the garage. Similarly, effective signage, graphics, and other visual cues communicate to the patrons in order to limit their time within a garage by quickly finding a parking space, getting directly to and from their destination, and efficiently exiting the facility. The less time one has to spend in a garage, the less opportunity there is for something inopportune to happen.

Lighting

Lighting is universally acknowledged as the most important security feature in a parking facility. Eliminating dark areas deters crime and promotes user comfort and the perception of security. Good lighting permits the safe movement of pedestrians and vehicles within the parking facility, and it promotes way-finding.

Light levels are generally not mandated by the building codes, other than certain minimum levels required for emergency egress. The industry standard for lighting design

in parking facilities is published by the Illuminating Engineering Society of North America (IESNA). While not a legal building code, failure to comply with IESNA standards does carry significant liability.

The IESNA standards are defined in terms of illuminance, or, in other words, the amount of light falling on an object. In general terms, parking structures require a minimum illuminance of 1 foot-candle and 10:1 maximum to minimum ratio for uniform light distribution. Stairways and lobbies require 2 foot-candles minimum. Vehicle entries and exits require 50 foot-candles to safely transition from bright sunlight to the interior space.

[picture]

Typically, the white light provided by metal halide or florescent lamps is more comfortable to patrons because it is perceived to be brighter. Florescent lighting is the least expensive to operate, due to lower energy demand and longer lamp life.

Ideally, fixtures should be paired in each parking bay, spaced at approximately 30 feet on-center in each direction. The fixtures should generally align near the aisle ends of the parking stalls. This somewhat mitigates the shadows created by the parked cars, as well as the lighting glare in the drive aisles. The paired fixtures improve the uniformity of the lighting and allow for a certain amount of forgiveness if a single lamp fails.

[picture]

Staining the ceilings and walls of a garage white is also a cost-effective way of further distributing light by increasing reflectivity.

Perimeter Control

Generally speaking, access to the parking facility should be restricted to the parking patrons. Access control can be achieved on the lowest levels by security screens, pedestrian doors, and vehicular grilles and doors. In certain high-security applications, solid walls may be required. Pedestrian access should be limited to a few monitored locations, near attended locations, such as a cashier booth or security office, if possible. Common sense also dictates that pedestrian entry and egress should be directly to the public way or into the facility that the parking garage serves, not into alleys or other unsecure locations. In private garages, key cards can be used to restrict both pedestrian and vehicular access.

[picture]

Active security measures

As discussed previously, active security measures necessarily require a human response. The most basic active security measure is a periodic patrol by trained security personnel.

This typically involves a uniformed “security guard” circulating through the parking facility on foot, bicycle, cart or other vehicle.

[picture]

Electronic security systems can also be considered active security measures, provided that they are continuously monitored and prompt an immediate response to a criminal act. Electronic security systems include intercoms and emergency call boxes, as well as closed-circuit television (CCTV) systems. CCTV monitors should be visible to patrons to promote the sense of security and deter incidents. Recordings of the CCTV monitoring can also allow investigation and confirmation of alleged incidents.

[picture]

Conclusion

A comprehensive security program is a key feature to any parking facility and operation. Passive security measures incorporated into the design of garage and active security measures such as foot patrols or CCTV monitoring will create a welcoming, safe, and secure space. Parking professionals have a variety of tools and techniques at their disposal to tailor a solution specific to each parking facility. With features that deter criminal activity and promote safety, a garage will be well- received by the user, the community, and the owner.

References

- 1.

An Integrated Framework for Enhancing Campus Security

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Abstract

Applying security to campus buildings, warning systems, communication and computer network systems will greatly reduce the risks of hazards during network attacks, emergencies, and day-to-day operations. Today's campus heavily relies on computer network systems to function and perform most of their activities in teaching and research. As campus resources are connected to the Internet, the implementation of an effective security system becomes imperative. This paper outlines the risks of campus network systems and presents a survey of examining the practice of intrusion detection and prevention, which encompasses possible security aspects in a campus or an organization environment and the technologies to protect the communication systems from increasingly sophisticated attacks. Specifically, an integrated security framework has been introduced for protecting people, systems and information in the campus.

I. INTRODUCTION

Campus is known for high security treats; in fact it is open to the public. When building up a new service facility in a large organization like university campus, it also needs to make it a protected system and accompany it with a well-designed security plan. Indeed, security is expensive, time consuming, and sometimes hard to estimate the benefits. However, the potential loss of an unsecured system will be much higher than its investment in security.

Computer network security is a big issue in campus as it affects users, teaching and research activities in a variety of ways. Today, the use of Internet via campus networks for data exchange has substantially increased the vulnerability of information systems in the campus. Due to the potential increase of security incidents, protecting the information, devices, and the infrastructure that comprises the campus network is essential to the campus operation.

Campus systems come under constant threat from attackers. The biggest threat to campus comes from the network attackers and their intentionally trying to gain entry into the campus information systems. Campuses must be fully aware of the risks of their systems and all security countermeasures available in the network and communication technologies.

Intrusion detection and prevention is the capability of hardware and software to alert users to suspicious connection attempts that could represent attacks trying to gain unauthorized access to campus network systems and their resources, and the ability to react to the attacks. Using intrusion detection and prevention techniques is important for an organization to discover intrusion attempts, respond to break-ins, trace network attacks, and assess the attack damage. Common design and implementation of this system include packet filters, firewalls, IPSs (Intrusion Prevention Systems), anti-virus, anti-spyware, and anti-spam software, network access control, virtual private networks (VPNs), and network traffic signatures, etc.

In order to have a safe and secure campus we must look at all aspects including network systems that could encompass security issues. Campus needs to be aware of the

security risks associated with systems and facilities and develop strategies to eliminate them.

This paper outlines the risks of the campus systems. Specifically, the computer network system threats have been examined. The paper presents a survey of examining the practice of intrusion detection and prevention, which encompasses possible security aspects in a campus and the technologies to protect the communication systems from increasingly sophisticated attacks. A cumulative information system defense strategy will be introduced, including host security for protecting the equipment and data recovery from attacks, Internet security taken to reduce the risk of Internet attacks and email vulnerabilities, and network security for wired and wireless communication system security set up. As a result, we introduce an integrated security framework for protecting people, systems and information in the campus, which will be discussed in the following sections.

II. AN INTEGRATED SECURITY FRAMEWORK

With all campus activity concerns, we understand that a reliable security system must be well designed and implemented to protect people and the facilities.

Specifically, in order to have a safe and secure campus, we must look at all the different aspects that could encompass security issues, which are listed on Table 1. All possible threats of the campus also be identified and listed on the same table. When a security aspect is encountered with possible threats, the cross points or cells become the danger spots, shown on the table.

Without Integrated Security		Threats of campus		
		Network system	Physical activity	Disaster happens
Security Aspects	Location / Place / Area	Danger	Danger	Danger
	Special Situation / Event / Activity	Danger	Danger	Danger
	Organization / Responsibility	Danger	Danger	Danger
	Continuous Service	Danger	Danger	Danger

Table 1. Campus security concerns

Contrarily, applying appropriate security resources to the aspects listed on Table 1 will provide an integrated security framework to the campus. It is shown on Table 2.

A. Security Aspects for Campus

Security measures should be conducted at every point in campus to avoid having a vulnerability that could be exploited. As shown on Table 2, the security aspects in campus may be categorized into location/place/area, special situation/events/activities, organization/responsibility, and continuous services.

Integrated Security Framework		Threats of campus		
		Network system	Physical activity	Disaster happens
Security Aspects	Location / Place / Area	Security Resources of Campus		
	Special Situation / Event / Activity	Equipment / facility		
	Organization / Responsibility	System / Software		
	Continuous Service	Group / people		

Table 2. An integrated security framework

The location/place/area in a campus usually includes the library, classrooms, faculty offices, research labs, parking lots, campus walkways, dormitory, food court and restaurants, restrooms, administration center, gym, swimming pool, sport field, and bus stations. The special situation/events/activities of the campus indicates the time when the campus is closed at night and holidays or opening for a special event such as the commencement. During the time, potential threats would be different from the normal operation of the campus and more security resources may need to be applied. The organization/responsibility aspect of a campus can include the university admission office, cashier office, IT center, department offices, faculty member, staff, and students. Special treatment in security may need to be considered.

In addition, the continuous services of the campus usually include wired and wireless computer networks, power supply, and water supply. The continuous services play an important role in campus activities. The increased risks in security come as well.

B. Campus Threats

There are many different types of threats that affect users and facilities in the campus, categorized into three kinds: computer network system threats, physical activity threats, and disaster threats. It has been listed on Table 2.

B.1 Computer network system threats

Computer network security is a big issue in campus as it affects users, teaching and research activities in a variety of ways. Security risk examination is one of the first steps in assessing the network's security configuration. The network group needs to identify potential security risks by collecting the information on threats, vulnerabilities, exploits, and countermeasures.

Network threats can be broadly classified into several kinds: eavesdropping or packet sniffing, compromised-key attack, tampering, probing and scanning, IP address or email spoofing, denial of service (DOS), malicious software, and information disclosure, etc. Simply having computers connected to the Internet will make the system instantly vulnerable to network threats

Each threat is different and is specific in what it performs. Eavesdropping or packet sniffing is a typical threat that someone tries to capture and analyze the not-properly encrypted data traffic from the transmitting packets that pass through a given network device. Compromised-key attack is a result after the attacker captured the data encryption logic and the key. A more serious threat is tampering; this involves the modification of data. An attacker will not only capture the information, but also change its contents. For example, alter data during transmission, change data in files.

A probe is an unusual attempt to gain access to a network or to discover information about it. One example is a try to log in to an account by using guessing programs to find the user name and password. Probes are often followed by more serious security events. A scanning of a network is simply a large number of probes. It can begin when an intruder identifies a public IP address that may be on a DMZ (demilitarized Zone, it will be discussed shortly.) or in a zone that may seem secure at first. Scans could be the result of some misconfiguration or other errors, but they are often more directed attacks on systems that are vulnerable.

The network's software systems may trust an IP packet from an intruder. A receiver may trust an e-mail that was generated with fake sender information by an attacker. The IP address or email spoofing is the attempt to obtain and use other's authentication information. Examples are using other's username and password, forge email messages, and resend authentication packets.

The goal of denial-of-service (DOS) attacks is not to break-in a system, but to prevent legitimate users from using it. Attackers may flood a system by sending in a large volume of data or deliberately consume the system resources. For instance, spam mails can make the mailbox full and the mail service disabled. A spam is an unwanted junk e-mail usually sent to people for products or services. The vast amount of spam sent across the Internet can be bothersome and even offensive such as DOS attacks.

Malicious software is the program that, when executed, would cause undesired results on a system. Users may not be aware of its existing until the damage occurs. Common malicious software includes virus, worm, Trojan horses, and spyware. Virus and Trojan horses are commonly hidden in legitimate files that attackers have altered to perform harmful functions. A virus can replicate itself, usually through an executable program attached to e-mail and viruses pass from computer to computer. But Trojan horse program usually appears to be something useful. Worms are self-replicating and self-propagating programs to consume disk space, but sometimes require user actions to spread inadvertently to other systems over the network. Moreover, some worms and Trojan horses can install "backdoors" that make the attacker to gain unauthorized access to a computer and network resources and even gain control over the system.

A spyware program is installed secretly (e.g., after a user clicks a hyperlink or runs a program without verifying its authenticity) on a computer to intercept or take partial control over the user's interaction with the computer. It then sends the collected personal information from the infected computer to the attacker who initiated the spyware. Spyware can also make system modifications, resulting in a slow Internet connection, different home pages or startup files, and even loss of programs.

It is worth noting that a security risk, information disclosure, is usually not perceived by the users and may cause potential harmful actions. Information disclosure involves the exposure of information to people who are not supposed to have it. Examples are to expose information in error e-mail messages or on websites.

B.2 Physical activity threats and disaster threats

Threats may come from physical activities, such as stealing and violence. Natural disasters such as earthquake, floods and fires can also be a serious security problem or hazard. Human errors like use of poor wiring for electrical gadgets may cause short circuits and start fires. When it comes to water damage, the effects are just as severe or as bad just like in the case of fires. Water damage may be caused by people or it may occur naturally. People-made-damages may be brought about by errors like poor planning and design of water piping plans. Natural floods may be caused by heavy rains, dam breaks or even tsunamis. An event like a tsunami may seem far-fetched, but all kinds of disasters should be well prepared for no matter the magnitude.

C. Protecting Campus with Security Resources

The threats discussed in last section have provided challenges for day-to-day operations of the campus. We need look at the possible security resources that can be used to reduce the risks of any attacks. Besides the group/people resources, which will usually include the campus security committee, internal and external auditors, system security administrators, campus police, and campus patrol, we will focus on the resources for network security design and implementation.

Computer network security design and implementation play a very important role in protecting communication and computing systems and the large amount of information they store in campus. The design should involve the basic security concepts important to information to be protected. Besides the well-known CIA-triad principle [16], the network security system design will ensure the data confidentiality, integrity, availability, adequate protection, effectiveness, and depth protection. Confidentiality states that the system and data resources should only be accessed by authorized users. The integrity suggestion is that data and information must be consistent and accurate during the transmission and processing. The availability requires the systems are always on and is making sure the information be there when authorized users need it. Adequate protection means whatever security component or parameter must be consistent at all times. If any control or change is implemented, effectiveness must be appropriate. Depth protection is responsible for a solid secure mechanism that will come up upon the network attacks. Concepts relating to the users are authentication and authorization. Based on these, we need to take in consideration a well organized and structured network security development.

C.1 Equipment/facility resources

A basic security precaution for campus networks will include data backup, anti-virus processing, and setting up firewalls. Backup of essential information stored on computers must be done on a regular basis and it should make sure the restoration is possible. This can avoid the threats of hardware failure and system attack that could result in a loss or compromise of data. Also, in order to avoid campus backbone network break up, a backup WAN (Wide Area Network) will help for the system coming up upon a disaster happens.

Intruders often attempt to gain access to the system by pretending to initiate connections from outside trusted networks. A typical attack will be like they disable the functions of a genuine host using DOS attacks and then attempt to connect to the campus network using the victim's IP address. To combat these IP address-spoofing attacks and add limitations on authorized connections to the campus network, it is necessary to filter all incoming traffic by using a firewall.

A firewall can be a hardware or software system. It is designed to examine the incoming message and service requests routed to the host or the whole campus networks. Its purpose is to eliminate the IP packets or service requests that fail to meet the security criteria established by the user or campus network administrator. The client computers in campus are usually installed the software firewalls that run simply as programs on them. The hardware firewall works at the entry gate between the whole protected campus networks and the outside world like the Internet. This is true for filtering any inbound traffic and outbound as well. For example, the campus can use the firewall to prevent

users from viewing offensive web sites or downloading some software. Because hardware firewalls are typically the first defense systems against intruders, they must be carefully configured and implemented. The Cisco PIX (Private Internet Exchange) [10] is one of the most popular firewalls on the market.

A firewall may be simply a filtering router that just discards packets arriving from unauthorized IP addresses or attempting to connect to unauthorized campus service ports. More sophisticated design may include a proxy server operating on behalf of inner services of the campus. The proxy server authenticates requests, inspect the data for viruses and other malevolent intentions, and relay approved requests to the appropriate servers.

Modern firewall systems can be hybrid and have much more functionalities for port/packets/IP address/virus filtering, and usually have a network-based IPS system behind for large organizations like a campus. Instead of just filtering packets based on where they come from (IP address or ports) like a traditional firewall does, an intrusion prevention system (IPS) monitors the traffic on campus networks, scans usage files for suspicious patterns, looks for activity that looks like an attack or unauthorized access, drops attacking packets or even tracing the source of an attack, and generate alert to system administrators. The continuous monitoring of network activities is required by the IPS. It should be noted that an IPS doesn't replace a firewall but rather works in conjunction with it. There are many IPSs on the market, such as Cisco IPS 4200 sensors [10].

Honeynets and honeypots are usually implemented as parts of an IPS. A honeypot is a campus computer that sets a trap to attract attackers [3]. Having the attacker to spend time long enough trying to exploit the "vulnerabilities" of honeypots, the type of attacks will be characterized, and the attacker's port number and IP address are recorded and further to be traced. The campus with a honeynet formed by multiple honeypots can monitor the attacking activities and analyze the security threats faced by the large organization. With the information captured by honeypots, system administrators can configure firewall policy to enhance the network security.

Apart from filtering the traffic to protect the networked computers from hostile intention by using firewalls and the IPS, the security of campus networks can be enhanced by adding a demilitarized zone (DMZ). The DMZ is another network sitting outside the secured campus networks, as shown in Figure 1. The DMZ usually contains the web server and the e-mail server for remote access. Outside users can access the DMZ, but cannot access the secured campus networks unless through a VPN (virtual private network) which will be discussed shortly.

C.2 System/software resources

The software system resources provide host protection and data protection of campus computer networks.

First, the security must be implemented on each host and their operating systems in the network. An anti-virus solution is essential. The anti-virus program searches the computer and network storage for any known or potential viruses and must be installed and configured correctly on the hosts. When anti-virus program detects a virus, it either deletes it from the system or places it in a "quarantine" area in storage where it cannot

self-replicate or harm to others. As anti-virus software compares virus signature files again the programming code of known viruses, it is crucial to regularly update virus

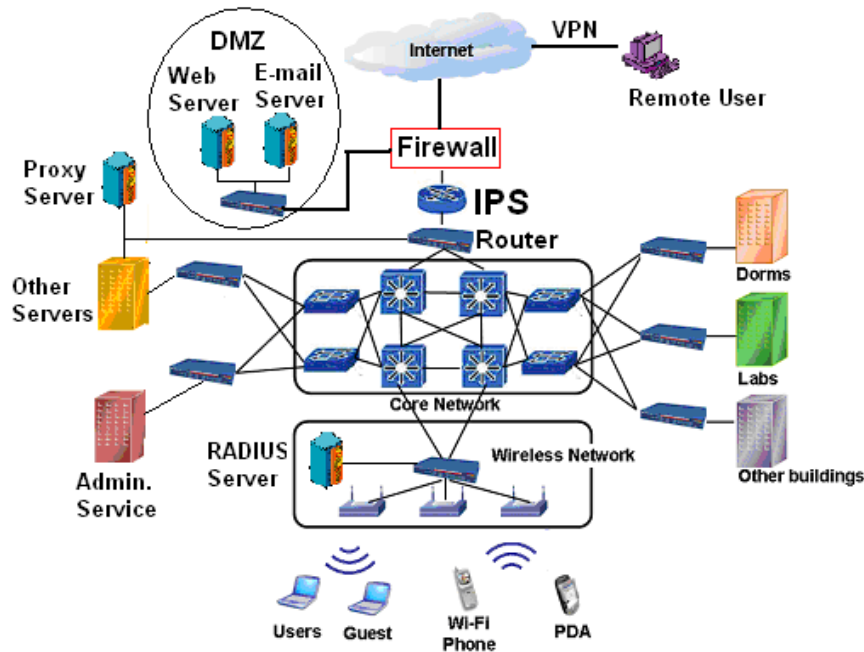


Figure 1. Campus computer network protection

signature files to provide hosts the best protection.

There are many Enterprise version anti-virus programs on the market, such as McAfee VirusScan [7] and Symantec Endpoint Protection [15]. Some combined solutions are also available. Examples include the Total Protection for Endpoint from McAfee, which provides a centralized security management of multiple computer endpoints for anti-virus, anti-spyware, anti-spam, and network access control [7]. More features of this product include desktop software firewall and host-based intrusion prevention (IPS).

The host-based IPS is usually used to protect a critical server system. The program is installed on the protected servers just like anti-virus software installed on each host. SNORT is such a most widely deployed open-source intrusion prevention and detection technology to defend the large and congested enterprise networks in the world [14].

A software firewall is an approach for protecting networks instead of hardware device solutions. Microsoft Internet Security and Acceleration (ISA) Server has the combined functionality of a software router, firewall, and host-based IPS working on the Windows operating systems [8].

In addition, applying operating system and application patches along with service packs will increase the operating system's security and stability. It should provide an easy way to deploy updates and security fixes from the server to the users' computers and provide administrative reports that track patching results across all campus computers in the network. Microsoft WSUS (Windows Server Update Services), for example, is such a patch management tool [8]. System administrators can use it to centrally update released critical patches for all campus computers running Microsoft Windows and other

Microsoft software. The updates will be applied properly and in a timely manner. Another software tool, Microsoft MBSA (Baseline Security Analyzer) [8], can help analyze security problems on a machine by comparing the current Windows security configurations on it to the baseline setup. The software will generate a report that shows recommended adjustments for unsecured security configurations.

Many other useful software tools are also available to scan or check up the security issues on the servers or client computers running different operating systems. [17]. For Example, Nessus is a popular UNIX vulnerability assessment tool [9]. It helps to identify potential security problems on the tested systems for vulnerabilities, misconfiguration, weak passwords, and denials of service. A Windows version is also available with it.

Second, protecting a stored or transmitting data or information is one of the most important works of campus network security. As wireless hotspots, or locations where Wi-Fi data services are available, are deployed more widely in campus, wireless LANs (Local Area Networks) are increasingly being used to provide students and faculty members flexible network connectivity. Wireless networks are becoming an important part of a campus 's infrastructure.

Because the transmissions occur over an unbounded medium, a wireless network is more exposed to intrusion, jamming, and drive-by hacking of the information. As a result, data protection is becoming essential during its transmission over campus wireless network systems. Data encryption is the process of scrambling stored or transmitted information so that it is unintelligible until the intended recipient unscrambles it. Key-based cryptography algorithms are usually used to scramble data into cipher text to protect its confidentiality. It is also used to protect the integrity and authenticity of the information.

The open nature of wireless transmission makes the campus wireless networks prime targets for information security threats and requires an increased level of authentication and authorization. Authentication is proving a user being whom he or she claims to. Authorization is determining whether a user can access a certain resource or running a particular program. Users must be authenticated before performing the activity they are authorized to. Today, many corporations adopt IEEE802.11i standard to provide a robust security for their wireless LANs. 802.11i makes use of the extremely strong Advanced Encryption Standard (AES) block cipher encryption with 128-bit keys to provide data confidentiality and integrity [6]. The framework provides centralized authentication and dynamic key distribution using IEEE 802.1X [5] and Extensible authentication Protocol (EAP) [4]. Mutual authentication is implemented between clients and a central authentication server. Most authentication servers use the RADIUS (Remote Access Dial-up User Service) standard [12], illustrated in Figure 1.

An integral part of security dealing with data protection is virtual private network (VPN) [2]. VPNs provide an economical yet secure way to let the remote users to create a campus network connection over the public network such as the nonsecure Internet. It maintains the data privacy through the use of a tunneling protocol that effectively conceals the private data information by encapsulating it within the Internet transmission packets. Further security is provided through the use of data encryption and decryption procedures. The most sophisticated VPN tunneling protocol is IP security (IPsec) [13], which provides security to the internet and upper layer protocols by mutual

authentication, encryption of each IP packet in a data stream, and cryptographic key establishment. The VPN’s transport mode gives a strong end-to-end security for two computers between the remote user and the campus’s fire server that are communicating using IPsec. To achieve this, a public key certificate authentication, which involves the digital signature, must be installed on both computers. Digital signature is a public key encryption scheme: the receiver uses the sender’s public key to decrypt a message (verifying the signature) that was encrypted by the sender using the sender’s private key (signing the message) [11].

III. Applying Practical Security to Campus

The appropriate security resources should be allocated to protect against particular types of attacks or threats. For example, a well-designed network security system should be deployed to any locations in campus where a computer is connected to the networks. It is illustrated in Table 3.

Applying Integrated Security		Threats of campus		
		(1) Network system	(2) Physical activity	(3) Disaster happens
Security Aspects	(1) Location / Place / Area	Equipment / facility System / Software	Danger	Danger
	(2) Special Situation / Event / Activity	Danger	Danger	Danger
	(3) Organization / Responsibility	Danger	Danger	Danger
	(4) Continuous Service	Danger	Danger	Danger

Table 3. Applying network system security

To reinforce the campus security including the computer network security infrastructure, adopting new means of reliable security steps will fulfill teaching and research requirements and people and information protection. Today, rather than wireless LANs, more wireless network technologies are becoming important part of a campus’s infrastructure. Figure 2 shows an anti-intrusion system using new wireless technology with existing campus monitoring systems. Campus security monitoring system is the solution for all day monitoring, usually equipped with cameras. With high resolution and night capturing capability, image recognition functionality, and network transportation ability, the system can act much more than simply recording the video but analyzing the video contents. As shown in the illustration, the network data transportation of this anti-intrusion system depends on WiMax [1], a broadband technology providing wireless communications over much longer distance than wireless LANs used by campuses mostly for wireless access and predicted to be widely deployed shortly. In the example, security services are provided and depicted in Table 4.

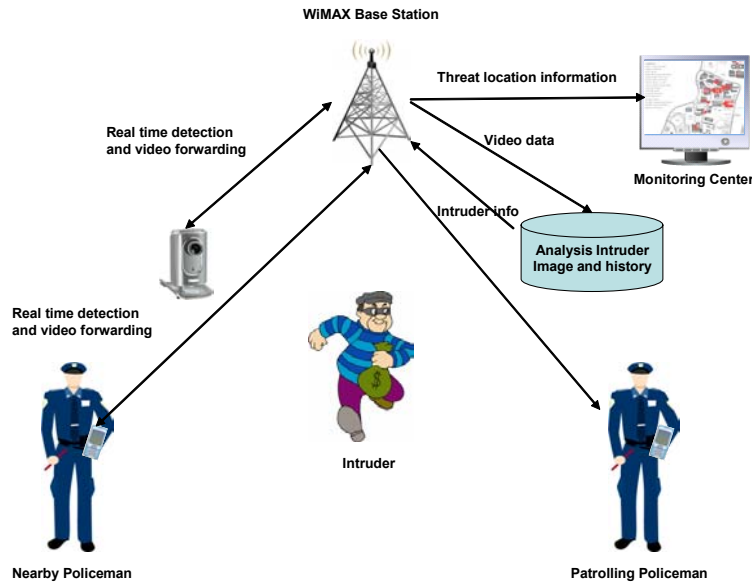


Figure 2. An anti-intrusion system with WiMax networks

Applying Integrated Security		Threats of campus		
		(1) Network system	(2) Physical activity	(3) Disaster happens
Security Aspects	(1) Location / Place / Area	Danger	Equipment / facility System / Software Group / people	Danger
	(2) Special Situation / Event / Activity	Danger	Equipment / facility System / Software Group / people	Danger
	(3) Organization / Responsibility	Danger	Danger	Danger
	(4) Continuous Service	Danger	Danger	Danger

Table 4. Security services provided by an anti-intrusion system

IV. CONCLUSION

Today’s campus heavily relies on computer network systems to function and perform most of their activities in teaching and research. As campus resources are connected to the Internet, the campus communication network systems come under constant threat from outside attackers. The biggest threat to campus comes from the network attackers and their intentionally trying to gain entry into the campus information systems and their resources. Campuses must be fully aware of the risks of their systems and all security countermeasures available in the network and communication technologies. As a result, the implementation of an effective security system becomes imperative.

This paper outlines the risks of campus systems and suggests ways of eliminating them. It has presented a survey of examining the practice of intrusion detection and prevention, which encompasses possible security aspects in a campus and the technologies to protect the communication systems from increasingly sophisticated attacks. Specifically, an integrated security framework has been introduced for protecting people, systems and information in the campus. With the way to view the campus security as such an integrated framework, it is more organized and easier to strengthen the whole campus system with new technologies.

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Developing Protective Strategies for Critical Building Infrastructures

Potentially Subjected to Malevolent Threats*

By

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ABSTRACT

The “events of nine-eleven” were an awakening for all professionals involved in design and construction of buildings and infrastructures and emphasized the importance of providing a safe and secure accommodation for their occupants, their functions, and their missions regardless of location. Building codes and standards are currently limited in providing adequate guidance for the managers, engineers, and architects who are responsible and ultimately liable for protecting and assuring the integrity of the built environment. However, noble attempts have been made in more recent years to identify critical assets and to determine cost-effective measures for minimizing the impacts of a potential adversarial attack. In order to be assured that viable protective means have been taken, a systematic risk assessment and management approach has been developed that has been applied successfully to numerous critical buildings and major supporting infrastructures. This paper describes and demonstrates the application of a validated methodology by providing typical examples where the systematic approach has been used for high-rise buildings, for its supporting primary and backup utility systems, for communications and data centers, for adjoining parking facilities and building access systems, for security and fire protection systems, and for other critical assets that were determined as a requirement to maintain operations. A systematic building characterization and functional inventory initiates the procedure that establishes the usually numerous most-critical-assets. Subsequent steps further detail vital information such as: 1) how the threat scenarios are determined, 2) how the impacting consequences are evaluated, and 3) how the infrastructure vulnerabilities are identified so that justifiable and cost-effective protective counter-measures can be recommended for subsequent management decisions. The primary objective of this robust decision-support process is to evaluate and provide the “real data base” so that owners and managers become fully cognizant of all the technical and operational details that are vital for their continued building operation. The risk assessment information can then assist with determining where the appropriate protective strategies indicate that both a) the calculated risk reduction potential is possible, and b) an assurance that consequences (including the loss of partial or total mission, loss of life and injuries, and owner income-revenue) can be adequately mitigated in the event of a potential malevolent attack. Alternatively, all practical and conceivable contingency operations and recovery plans are revitalized and upgraded to minimize the impacts and damaging effects of any losses.

* Underline used only to indicate the possible topical area for this paper at the AEI Denver Conference (Sept 25-27, 2008).

1. INTRODUCTION

1.1. Background

As we progress into the 21st Century, we want to recognize that the natural, malevolent, and environmental (aging) events that often cause major property damage, severe injuries, and unnecessary death are not completely “acts of God”. As professionals we can and must do everything possible to prevent these catastrophic losses by changing the way we think about analysis, design and construction projects. New tools, technologies, and techniques can improve facility safety, security, and reliability and protect owners, occupants, outside users and stakeholders against major losses.

Hurricane Katrina; the ice storms of several winters in the northeastern US and border regions of Canada; flooding, wild fires, and earthquakes in California and the Southwest; tornadoes and flooding in Florida; the Oklahoma City bombing; the African embassy bombings, and the airplane attacks of 9/11; and the relatively recent failure of the Minnesota Interstate Bridge are just a selection of vivid examples of threats to the safety and security of the public. They continue to challenge the reliability and resilience of our constructed environment. Today’s architectural-engineering design community must continue to recognize all of these threats and consider them in our standard guidelines, codes, and final designs. We will continue to strive for excellence in predicting the likelihood of disasters, and we will utilize or develop the tools to assess risks, measure and account for the impact of disasters, and implement corrective actions where warranted. We will continue to demand and create innovative solutions to the problems of structural analysis, consequence mitigation strategies, and emergency response to disasters.

There are questions we can ask ourselves as we seek to meet the new responsibilities of our profession.

- Why do we evacuate buildings and infrastructures during natural disasters or acts of violence? Buildings have been designed to protect lives and property investments.
- Why do we wait for the next disaster to test the effectiveness of a building retrofit or facility enhancement that resulted from studying the previous disaster? There are ways to evaluate an upgrade that do not put people or facilities at risk:
 - Use scale-modeling and simulation techniques.
 - Use massive parallel computers and full structural models.
 - Predict component material performance
 - Apply/develop testing procedures to evaluate effective fixes
 - Use non-destructive health monitoring systems throughout the life of a building
 - Instrument facilities during construction for monitoring life-cycle performance
- Why don't we insist that A-Es stay abreast of life-safety-security-reliability concerns? The changes in our role that result from the changing world can be formalized. We must find a way to take responsibility for doing our part to reduce loss.

These suggestions for reducing the unacceptably high costs of the losses we currently incur in the as-built environment are not necessarily the best suggestions, but we cannot afford to ignore the impacts to society if we do not meet our challenges. A partnership of government,

industry, academia, and the profession is a way to clarify the problem and select appropriate solutions. The many national science and engineering laboratories are an excellent repository of technologies, tools, and techniques developed to serve and protect national high-consequence missions. Numerous programs developed at the national laboratories, universities, and research industries intended to bring rigorous scientific and engineering efforts to bear upon improving the response of engineered structures to environmental and other threats. Risk management tools, advanced modeling and simulation capabilities, nondestructive evaluation techniques, and structural health monitoring capabilities are a few of the tools available to improve infrastructure performance in such hazardous and malevolent environments.

Two such programs that the authors have been involved in include the development and applications of the principles that govern Architectural Surety® (Matalucci, et al, 1997, 1998, 2000, 2002a) and the security Risk Assessment Methodology (Matalucci 2002b; Biringer, Matalucci, O’Connor 2007). The elements of these two significant developments will be reviewed only in summary and only as they pertain to the issues of this paper. The references noted can be reviewed for details and for further applications.

1.2. Architectural Surety®

The concept of Architectural Surety® addresses the need for the integration of *safety*, *security*, and *reliability* concerns for any facility by ensuring that the performance of a building system is evaluated in a balanced and more comprehensive way. This procedure is designed to reveal to the owners and stakeholders all of the issues and concerns that have been evaluated and expresses the levels of risks that are associated with natural occurring events (*abnormal environments*), deliberate man-imposed attacks (*malevolent environments*), and functional reliability regarding aging and deterioration impacts (*normal environments*). These elements are illustrated in the Venn diagram below where they overlap and define the concept of “surety engineering”.

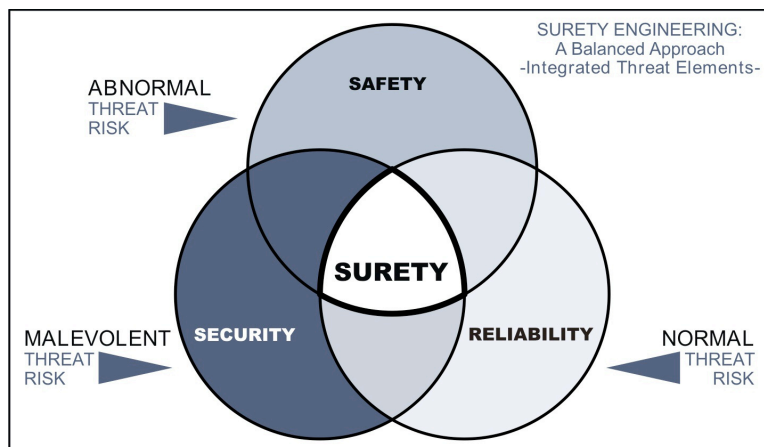


Figure 1. Surety Engineering Venn diagram - Integration of Three Elements of Surety Design

This paper will focus on the security element of surety as the areas of safety and reliability for abnormal and normal environments, respectively, are considered to have already been addressed by analysts and designers who specialize in each of these two specific areas. However, the importance of ensuring that these three elements are each evaluated with equal attention is stressed. Where there are security risk assessments performed that result in calculated risks to the public and stakeholders, the necessary corrective actions are pursued in the design with the emphasis placed where the likelihood of occurrence and impacts result in the highest levels of risk.

Infrastructure and Architectural Surety® is therefore concerned with the areas where safety, security, and reliability overlap. The scope of this paper has been intentionally limited to security issues. To this end, we have made the assumption that Safety and Reliability concerns have already been thoroughly addressed, and reported by other documents in the profession.

1.3 Risk Assessment Methodology (RAM)

The security Risk Assessment Methodology (RAM) that will be described and applied in this paper is divided into four basic parts that will be illustrated later and are identified as follows: 1) preliminary screening, 2) risk assessment, 3) risk management, and 4) emergency management. The preliminary screening is accomplished to determine whether there is a need for a full risk evaluation. Where the mission of a facility is deemed critical and the consequences that would result from a successful adversarial attack are severe. The high-level screening would clearly identify the requirement for further risk analyses. The risk assessment procedure is followed in a systematic step-by-step manner as indicated and will be described in more detail in a subsequent section. The risk management procedure is critical for the owner/stakeholder because decision makers are involved and policies, budget, and priorities are addressed regarding system upgrades strategies and consequence mitigation measures. Finally, where emergency management issues become evident, the necessary plans for evacuations, recovery, contingency and continuity of operations are engaged and placed into motion.

1.4 Demonstrate Value for Building Owner

Security risk assessments must be performed in such a way to provide obvious benefits to the owners and stakeholders regarding protection of mission, life, property, revenue, and adequate return on capital and other investments. Buildings and supporting infrastructures that have addressed these security risk concerns through upgrades, consequence mitigation, and/or protocol modification may be more attractive to occupants and prospective tenants. Individual tenants are often limited to suite protection or by the terms of their approved suite improvement. Obviously their mission security is contingent on building security and the protection of many utility services. Proactive building owners and stakeholders can mitigate loss of revenue from either failed lease agreements, or unanticipated vacancies due to a failed company mission, destruction of property or bankruptcy dissolution stemming from failure to meet consumer

expectation or contractual service. Building and infrastructure ownership is inexplicably tied to the financial success of its tenants and therefore, to the security of its tenant's critical assets.

1.5 Scope of Paper

It is necessary that this paper be concise regarding the security risk assessment approach that it proposes for buildings and infrastructure protection against malevolent attacks. We recognize the many elements that are required for any thorough risk analysis project, and some of them will be addressed herein. However, our purpose is to focus only on the basic security risk assessment and management approach and its application to a generic setting regarding the required protection of the infrastructure that supports a typical multi-storied and multi-tenant commercial office complex. Other indicated references can be consulted for more detailed information concerning other important issues. Therefore, after we describe a typical building and its facility infrastructure, we will present the risk assessment and management approach and its practical implementation for a generic high-rise structure. We will conclude with some points of value that further refine and re-enforce the major benefits that would be realized by the owner and stakeholders in the event of an adversarial attack.

2. GENERIC BUILDING MODEL

2.1. Building Program

There are many factors that can lead a building owner to request an analysis of a commercial property. For this discussion we have established the building program as a mixed use, multi-tenant, administrative financial complex with large data centers. The first floor is open to the public. There is also a private Day Care Center on the second floor for shared use by the tenants. The building owner has requested this phase of the analysis to protect the interests of the building tenants and has thus classified it as a 'critical mission' building. The tenants themselves have also asked for assurances that the facility can meet their individual corporate missions. As building programs change over time and tenant improvements modify utility and security requirements, often the 'existing' or 'improved' conditions have diverged from the Architect's and Engineer's original design criteria. Building ownership and management may also change and liability and licensure give way to due diligence and professional ethics.

2.2. Building Services and Utility

A generic building model description can provide basic information that is needed for preparing a full security risk assessment process. This building model description will also be applied as the guideline for obtaining pertinent details that are usually important when initiating a risk assessment process of a significant building and its supporting facilities. The descriptive categories and the extent of the building details are considered minimal and generic so that the topical areas and narrative can form the basis for an initial building survey "check-sheet" when a risk assessment study is required. The building description details that are provided will focus primarily on the critical assets that are normally evaluated for life-safety and mission oriented

concerns. Some of the other building descriptive details that are included might be more for occupant convenience and comfort, but are considered nevertheless assets that the management and decision maker could determine to be important for other reasons and might warrant protection.

The building itself is a 15 story reinforced concrete and steel frame building, with pre-cast exterior panels and curtain wall glazing. It has a large open lobby glazed on all sides with centralized vertical circulation. Data/Telecom utilities enter the building below grade with the main distributions housed in separate vaults in the basement. The main electrical distribution and transformers are above grade in the corner of the site, surrounded by a block wall and gate. From there, the power enters the building below grade and distributed to the floors via a central utility shaft located in the building's core. Also in this core are a wet standpipe, water and sewer plumbing, which also enter below grade near the front of the building. Chillers, heaters and main HVAC components are located above grade, secured by a fence and screened from sight. Minimal security exists in the main lobby where card readers permit building occupants entry without any form of sign-in. The unarmed receptionist/security officer verifies all visitors without badges, requires a sign-in process, and informs employee to escort visitor from the lobby to the meeting location. There are no intrusion detection systems located anywhere nor surveillance cameras to monitor activities in the entry or in the parking areas. Hours of operations for the reception area are 7:30 am to 5:30 pm, Monday thru Friday. The building is locked otherwise and entry via a card reader is permitted for non-operating hours.

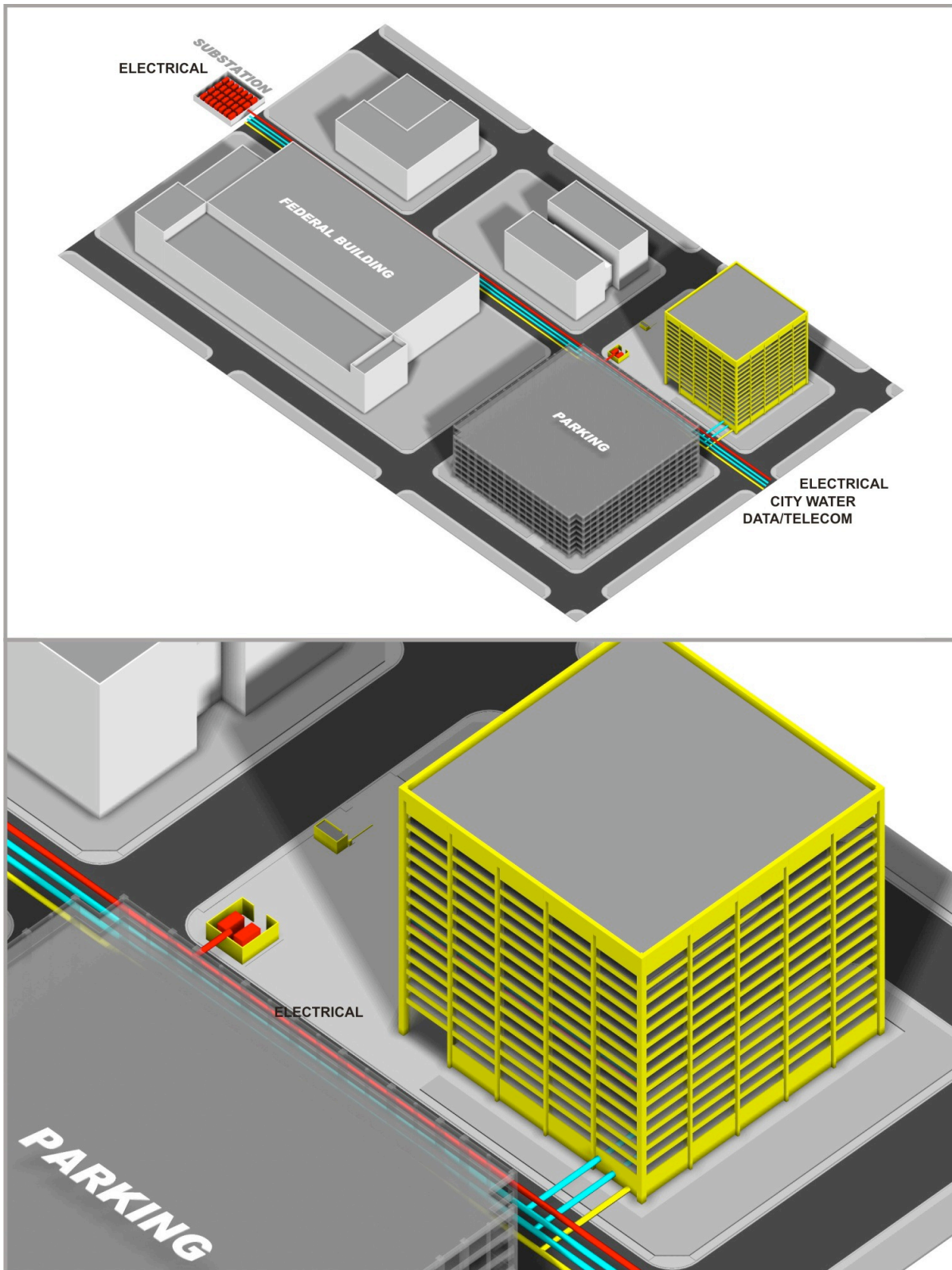


Figure 2. Infrastructure and Building Model for a Risk Assessment Project

2.3. Ownership and Responsibility

Some corporate communities have chief competitors as neighbors. An increasingly mobile workforce further increases the risk of corporate sabotage or collateral damage from disrupted utility service. Within such communities, secrecy is large component of the success of the corporate mission. This extends to protective strategies. However, security personnel can be encouraged to identify commonalities within its standard operating procedure or security protocol and exchange with its neighbors charged with building security. Such commonalities can be discussed without violating confidential agreements or divulging specific corporate vulnerabilities and have an inherent value to all parties involved beyond a simple awareness.

Corporate rivalry is clearly not limited to separate facilities, however. In the case of our model multi-tenant corporate building, competitor adjacency or superjacency (tenant above or below) may be within the same building. Building owners are bound by separate agreement to each of its tenants but are required to maintain certain minimum comprehensive services to all of its tenants. In such a case, disruption of utilities from a malevolent attack may cause failure of multiple critical missions either through direct means, or indirect collateral damage.

Early in the risk analysis process, determining personnel and positions responsible for review, approval and implementation is of paramount importance to the success of the risk management project. Without a legislative mandate, accountability for decisions and support for a rigorous risk analysis relies on professional ethics both on the part of the analyst and of the parties charged with review. In some cases certain protocols for review of security procedures are in place, but are limited by the scope of enforcement. Inter-agency communication is therefore limited if not discouraged. Establishing lines of communication amongst adjacent building owners and their respective security personnel allow continued analysis of shared vulnerabilities and deficiencies beyond the property boundary. This is especially important if the property under risk analysis is in proximity to a potentially high value target, such as a federal building, a critical data center, or a prison complex.

3. METHODOLOGY

3.1. Security Risk Assessment Methodology Overview

The security Risk Assessment Methodology (RAM) is well described and demonstrated by practical examples, elsewhere. (Biringer, Matalucci, and O'Connor, 2007). The baseline summary risk assessment process specifically for dams and hydroelectric power transmission systems is contained in another publication (Matalucci, 2002). The basis for these security risk assessment process approaches is contained in the application of the fundamental security risk assessment equation that is expressed as follows:

$$R = f \{ P_A , (1 - P_E) , \text{ and } C \}$$

R represents the relative risk associated with an adversarial attack, as a function of the three risk factors, where:

P_A denotes the likelihood of the attack occurring,

P_E denotes the physical security protection system effectiveness in sustaining the attack,

$(1 - P_E)$ denotes the physical protection system in-effectiveness in sustaining the attack, and

C denotes the consequences estimated from the attack.

To calculate the relative (conditional) risk at a particular facility project, the methodology permits the use of data collection forms to address and document the above described factors of the security risk equation. These developed data collection and analysis forms, and the contained information therein, provide upon completion the guidance for the security risk analysts as they progress through the structured screening, risk assessment, risk reduction, and risk management sequences. The information required to fulfill this risk analysis process is obtained from existing documented information, from site surveys and observations, expert security judgment and background experience, and from the required interviews conducted with project participants. The flow diagram in [Figure 3](#) provides risk analysts the major parts of the security risk evaluation approach and delineates the steps that are followed when a full risk assessment is deemed a requirement by the owners and stakeholders.

The risk assessment methodology is initiated with a screening process that is based primarily on the magnitude of the consequences of undesired mission disruption events. If potentially high consequence events are identified, the procedure suggests that a full risk assessment be fulfilled, and that the risk analysis be required to proceed by addressing each identified facility on a prioritized basis determined by the screening procedure.

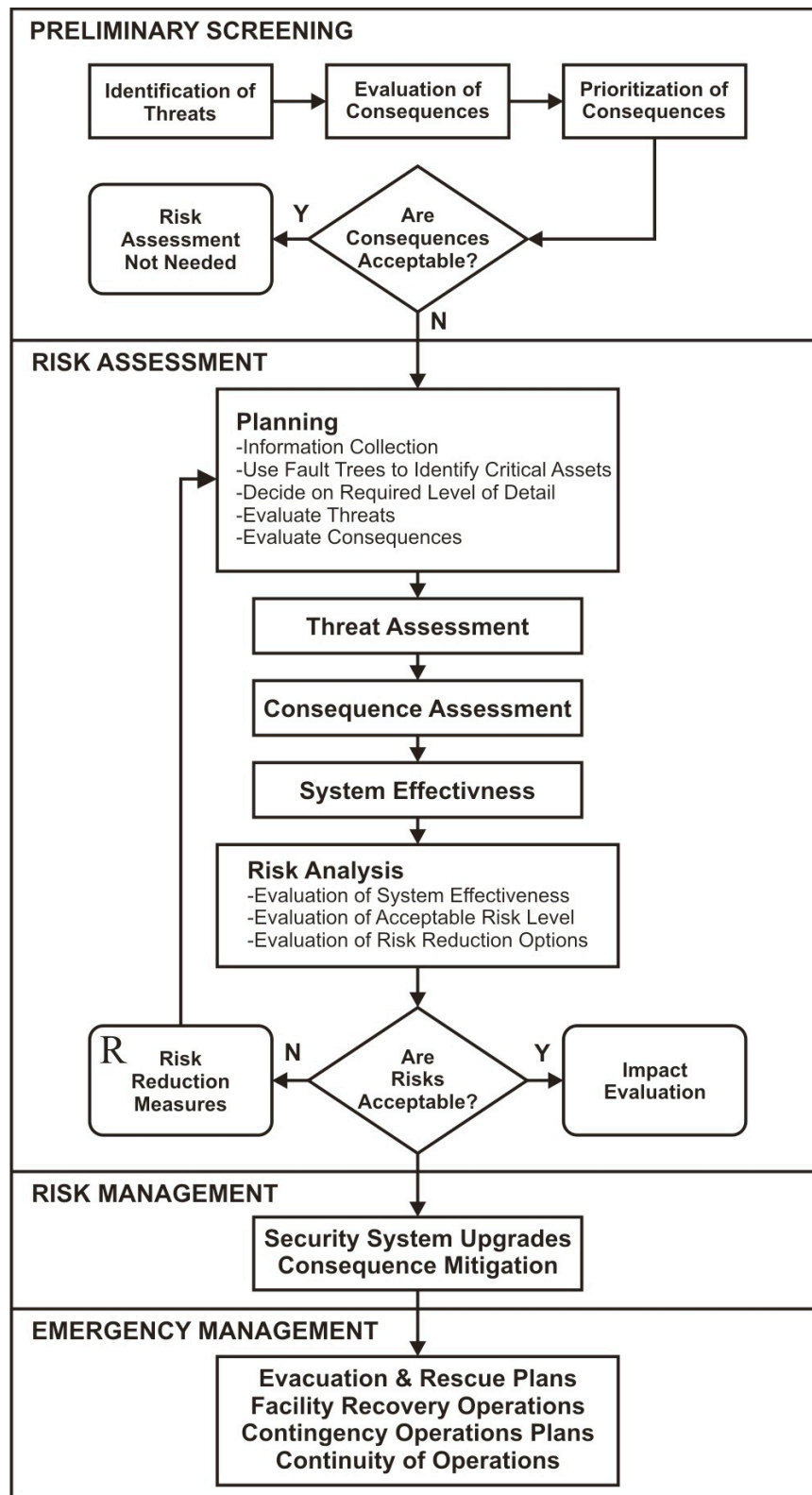


Figure 3. The Flow Diagram that Illustrates the Four Parts of a Risk Analysis Project

The risk assessment process then moves into a planning phase. This phase includes data collection and the development of site-specific fault trees to assist with the identification of those critical assets and sub-systems within the facility that need to be analyzed more thoroughly. A further advantage of the fault tree analysis is in the case in which potential adversarial attack paths can be recognized and evaluated for determining the attack scenarios that result in the largest impact to the owner/operator.

This planning phase procedure is particularly important because it systematically sorts for the critical assets and sub-systems (components) that could justify the protection of the project mission. This procedure also assists the owner with establishing the scope and required level of effort of completing the risk assessment. The completeness of the entire risk assessment that follows the planning phase is totally dependent on adequately addressing foremost the selection of these critical assets and sub-systems (components) that will cause a catastrophic facility failure under an adversarial attack.

The risk assessment process that follows the planning phase consists of the determination of the likelihood of the adversarial threat (P_A), the analysis of the effectiveness of the physical security protective systems (P_E), and the evaluation of the consequences (impacts) to the owner if the facility is disrupted/destroyed (C) following the attack. Once the risk level is determined as an integrated function of each one of these factors, the owner is consulted by the security risk analysts to provide the decision on the acceptability of the determined risk.

Where the risk level is not acceptable and risk reduction is required, the owner provides further guidance on the design basis threat, the level of additional protection desired, and the budget that is reasonable for the facility risks involved. Security specialists and engineers are then instructed to reach consensus on the suggested alternatives that are cost-effective and provide the amount of risk reduction desired. The owner is then in a better position to make an “informed” decision on the risk management efforts to implement. These alternatives include combinations of: 1) security system upgrades, including access control and intrusion detection, 2) consequence mitigation measures, including evacuation, hardening, and safe-havens, 3) enhanced security operations procedures, 4) facility redundancy designs, 5) contingency recovery plans, and 6) possibly optional emergency action plans.

3.2. Security Risk Management Alternatives

Upon completion of the security risk assessment, the analysts will report the results to the decision maker(s) by presenting the factors addressed and the level of risk estimated from the available information. Figure 4 represents most of the alternatives that are available for consideration by the risk assessment team and the owner and decision makers. Based on the data that is documented, some recommendation regarding the risk reduction strategies can be assembled if the level of risk determined initially is unacceptable.

These risk reduction strategies require an understanding of the operating policies that prevail, the likelihood and impacts that would result from a malevolent attack, the vulnerabilities that exist based on the size of the threat, and the budget that is available to implement any security system upgrades or consequence mitigation measures. The decision maker at this stage of the project, having access to some initial cost-effectiveness estimates for the upgrades, would be in a better position to identify the design basis threat (design criteria) that would be used for any enhancement to the physical security systems, hardening requirements, recovery strategies, redundancy options, and emergency action plan modifications that provide the most realistic benefits for the stakeholders. In summary, the information in Figure 4 provides some of the alternatives that can implemented if the risks involved for the public safety and to the project mission are considered unacceptable.

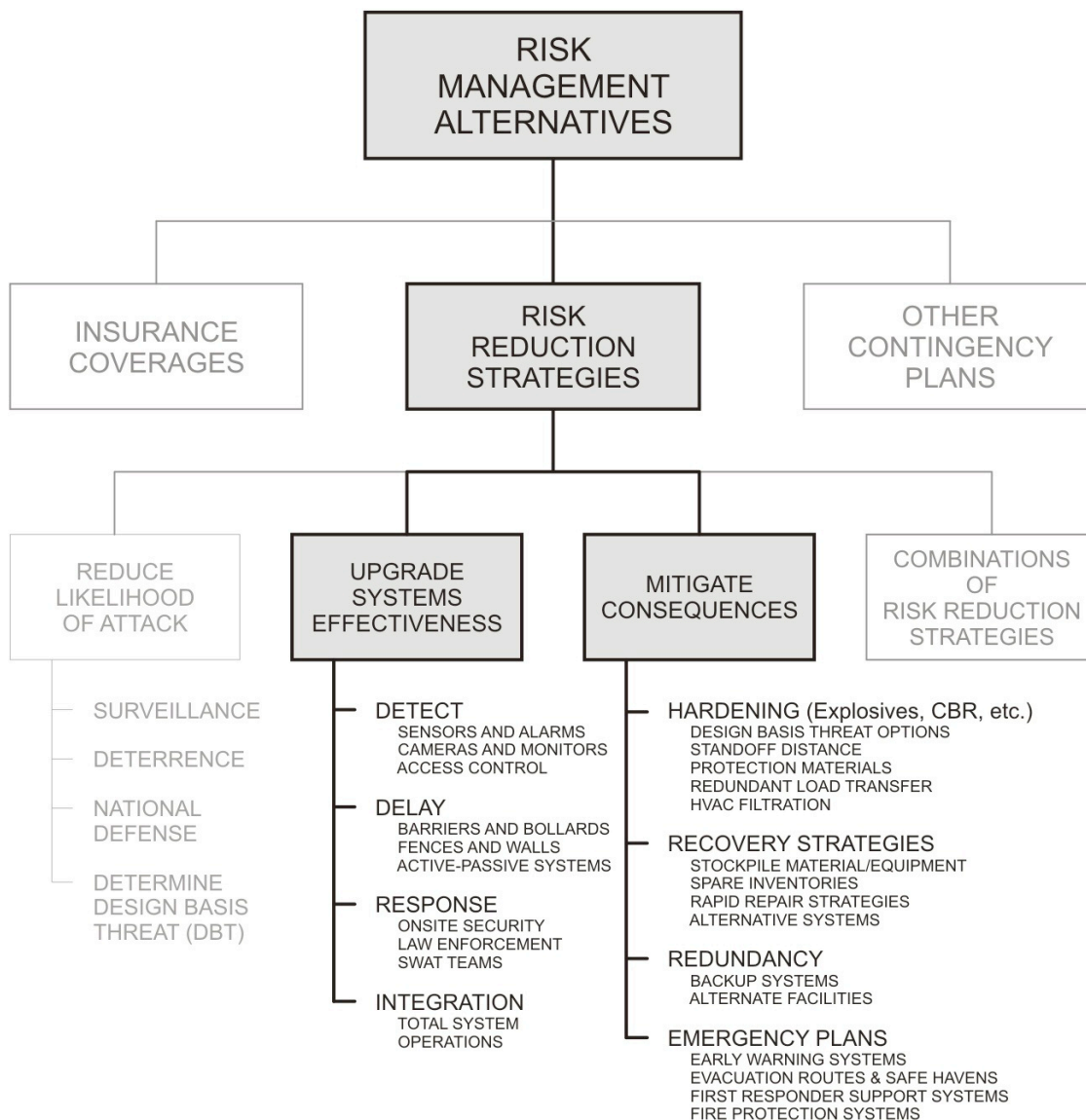


Figure 4. Risk Management Alternatives Illustrating Possible Risk Reduction Strategies

4. APPLICATION AND RESULTS

4.1 Security Risk Analysis

The security risk analysis process is usually initiated from a management decision to determine an evaluation regarding threats, consequences, and vulnerabilities that pertain to a building and its supporting infrastructure under examination. A screening effort usually establishes the justification that a particular facility is a high value property whose mission and occupants are critical for certain operations contained therein. The decision maker(s) is convinced that a full risk assessment is required to permit an evaluation for required security upgrade strategy and consequence mitigation in the event of a high likelihood of a malevolent attack. An assessment team is organized, a leader is identified, and the full mission and characteristics of the facility is examined thoroughly.

The risk assessment team through a fault tree analysis identifies the various critical assets that require some level of protection to sustain operations. For the example problem, the following critical assets were determined to most important by consensus: a) the agency information data center; b) the data files and storage vault areas; c) the safety features installed to protect the lives of the occupants (fire alarms, escape routes and suppression, security alarms and access controls, air circulation and controls); d) the child care center; and e) the electric power, water, and communication utilities. These assets are then considered the most critical and would receive the focus of the assessment. For the purpose of this example application, the selection of these critical assets does not necessarily mean that the other remaining building and infrastructure facility assets might not also be important. However, these are included in the subsequent risk analyses, as the ones deemed important for this facility by the assessment team.

Having reached consensus about these critical assets, the consequences that would result if any or all of these assets were destroyed by explosives or other attack scenarios are then determined. For this example, if failure of the infrastructure systems that support the building is possible, it is calculated from actual data estimates that the consequences would amount to the following:

- a. Loss of life, estimated to be > 100
- b. Loss of electronic stored data, estimated to be > \$20 M
- c. Loss of significant agency documents and files, estimated > \$10 M
- d. Loss of children, estimated to be > 50
- e. Loss of data control mission, estimated to be > \$10 M
- f. Loss of revenue for up to 6 months, estimated > \$100 M
- g. Loss of customer confidence and reputation, estimated to be > \$10 M
- h. Loss of property and reconstruction costs, estimated to be > \$100 M

Based on these consequence estimates, that are derived from actual agency records, available financial documents, and with assistance from engineers who offer analyses of damage levels, loss of life, and costs for reconstruction, a summation of the impacts of an attack can be estimated. These estimates are by necessity only a rough order of magnitude so that

management can evaluate with the assessment team the level of consequences that would result; these are then given a relative value of high, medium, or low (H, M, L) base on the tolerance levels of the agency involved. These values are also subject to revisions when the most likely threat is also determined by the risk assessment team. The decision maker contributes to the evaluation of the ultimate consequence level as a function of the agency's market value, financial situation, and value of potential loss of employees. For this example, an estimated accumulative consequence of greater than \$100 M loss is considered a high consequence (H).

The threat assessment is carried out with valuable assistance from law enforcement and intelligence agency estimates and records of past events and/or anticipation of future events based on historical evidence and targeting information. Although this element of the risk assessment equation is a difficult one to achieve with certainty, the best analysis possible is required with consultation from as many government and municipal agencies participating as possible, including the Department of Homeland Security (DHS) liaisons, FBI, and regional, county, local, and state investigative authorities. For this example, the facility in a large city has been recognized by competent authorities to have attracted attention from: 1) international extremist groups, 2) regional ecological protesters, 3) former disgruntled employees, and 4) militia. This information, although sensitive, requires confidentiality clearances, control and protection, and agency trust in order to gain sufficient and reliable data. From further threat analysis, the likelihood of attack is evaluated by evaluating adversary characteristics and capability, history and intent, and relative attractiveness of the asset. A high, medium, and low (H, M, L) value is then assigned by consensus for each of the most likely adversaries listed.

From this type of systematic and consistently rigorous threat assessment, the hypothetical likelihoods of the following threats were determined:

- a. International extremists, attacking using explosives; M
- b. Regional ecological protesters, attacking with hand-tools; L
- c. Disgruntled-insider former employees, attacking with hand-tools; M
- d. Militia; attacking with firearms and hand-tools; L

Applying the results of this threat assessment, the attack scenario that is considered most effective for the most likely threats (medium, M) above, the descriptions for use in the assessment is the following:

Scenario # 1: An international extremist group of three individuals attacks the facility infrastructure focusing at the point of utility entry into the facility and near the front entrance to the building below where the child care center is located (on the second floor). One individual attacks the building electric power substation using a 20 # satchel charge, the second attacks the intake of the ventilation system near the fans and chiller with a 20 # satchel charge, and the third ram s a van over the curb to the entrance where all the utility lines enter the building and detonates 500 # of high explosives.

Scenario # 2: One disgruntled former employee, in collusion with an employee insider, disrupts the electric power and ventilation/cooling systems by causing several short circuits to the controls of the cooling system of the data center, electric power, and communication systems simultaneously.

The third element of the risk equation considers the vulnerabilities from an attack of the various physical security systems and the other systems that would allow for full building operations. In the example problem where the consequences are noted and the most likely attack scenarios are considered, an analysis of the effectiveness of the existing security systems, and any other protective systems that are in place are evaluated.

Reviewing scenario # 1, there is only a locked steel gate and a 6-foot wall around the electric substation and no security monitoring systems. The ventilation intake is at ground level and not protected except for a 6-foot high chain link safety fence around the chiller and HVAC systems. The front entrance is in clear view from the 6-inch street curb and a vehicle can easily ram the doors and partially penetrate the entrance lobby. The utility services to the building are also near the front entrance, and buried about 3-feet underground and the meters and control valves are unprotected from any mode of attack, and are also within easy access either by foot or vehicle. An explosive attack at a strategic point near the entrance will destroy all utilities that supply the building and collapse the second floor where the child care center is located. Security personnel are located inside the building in the lobby near the front entrance where minimal access control is provided 5-days per week, 10 hours per day. No security is on duty otherwise throughout the remainder of the week, and the front doors can be unlocked using a card-reader system, only.

The protective system effectiveness is determined by evaluating detection, delay, and response systems and how well the physical security system is integrated into the facility operations. In view of no intrusion detection, minimal system delay measures, and no response force except the local police and personnel inside the building who are not given an alarm signal during any attack, the effectiveness of the security system is considered nil, and is assigned a value of 10^{-6} . There is also minimal hardness estimated for the shallow buried utility systems because they can be ruptured by the force of an air blast where exposed, and by forming a ground crater by the explosives in the van. The child care center is expected to collapse after an explosive attack because there is little blast protection to the floor and supporting columns.

Reviewing scenario # 2, there is a card-reader building access control system in place for employees only. Visitors are required to be escorted by an employee after a formal sign-in procedure is followed. Non-employees have access to all the facilities exposed outside of the building, including the electrical and data communication relays and controls that are inside a 6-foot wall or chain-link fence. The automatic-start electric power back up system is protected inside the building, however, the controls can be compromised from inside by short-circuiting some of the systems. Depending on the attack mode, and the knowledge of the attackers, these controls can be compromised and disrupted for up to two months by damaging critical systems with long-lead time for replacements.

The protective system effectiveness for this scenario is determined by the same measures of detection, delay and response. An insider can assist the former employee with intelligence data about the electrical, ventilation, and communication systems and where to best attack. There is no alarm system to alert the security personnel if an insider is attempting to compromise any circuits or control systems, prior to the external attack. The security systems, therefore, do not provide the protection against the disgruntled former employee who has some knowledge and is supported by the insider. The disruption may be of sufficient duration to severely impact service to customers and furthermore to the electronic information that is in the data center. The system effectiveness is considered nil for this scenario and is assigned a value of Low, L.

The risk analysis is performed by evaluating the values of the three elements of the risk equation and reaching some consensus with the decision maker about its acceptability. If the risk level is considered too high to accept for the mission involved and expertise of the occupants and the key customer base, a decision can be made to find alternative upgrades that will reduce the risk to a more acceptable level. The tentative and relative risk values are summarized in Table I below for comparative purposes. Additional scenarios can be included to provide a review for a broader range of likely adversaries. If the risks levels are determined to be unacceptable, a selection of security upgrade alternatives will assist management in deciding on the most effective measures for risk reduction. Other options are also suggested to include consequence mitigation through redundant systems, safe havens for occupants, protected evacuation routes, and alternative facilities off-site.

Scenario	Consequence C	Likelihood of Threat P_A	System In-Effectiveness $(1 - P_E)$	Relative Risk R
Scenario # 1 International Extremist	High (H)	Medium (M)	(1- Low) = High (H)	Medium to High (M to H)
Scenario # 2 Disgruntled Former Employee with Insider Assistance	Medium (M)	Medium (M)	(1- Low) = High (H)	Medium (M)

Table I. Comparative Analysis Values for the Elements of the Risk Equation.

5 INFORMATION TRANSFER

5.1 Licensure/Liability/ Negligence/ Due Diligence

In the absence of a legislative mandate, we have discussed just a few of the incentives for building owners. With the rapid cycling of capital constraints it is often difficult to justify the “do the right thing” approach on its own merit. The owner must also be given a set of tools to demonstrate the value of such an analysis and the cost of upgrades to board members, shareholders, lenders etc. Acknowledging that a privately owned building is very different than a public-owned federal building, the practitioner cannot rely strictly on the thoroughness of the risk analysis as justification for a project. As stated before, often the perceived value of a security upgrade is seen strictly as a cost and not as a means to secure an uninterrupted revenue stream and to save lives. If the purpose of a security risk analysis is to just locate vulnerabilities or how to secure critical assets, the risk analysis itself is unsuccessful if operational procedures and tools beyond a Risk Assessment Methodology are not employed; for example, intelligence gathering and threat identification, blast analysis, access control techniques, coordination with infrastructure owners and utility service suppliers.

Due diligence is a subjective judgment. In the case of civil litigation, it is often in the terms of the contract. Its denotation can even be outsourced to specialized companies. When it becomes a matter of independent inquiry, this subjective quality becomes objective and failure to meet this is deemed negligence. This inversion is, at its best, defined more by a list of trials and what not to do and at its worst, left to the Professional Ethics of the party involved. Due diligence as it applies to this paper may be described as beyond the competency implied by licensure of the professional. It rests both with the analytical rigor of a methodology and the integrity of its communication of intent.

5.2 Communication

Often the greatest benefit to the owner and stakeholder regarding security protective measures and the success of a comprehensive risk assessment project is in the information transfer both from the risk analysts and from the subsequent communication of the results and recommendations. For public works projects, there is frequently a legislative mandate that holds management and operations personnel accountable for the success of the facility to carry out the mission. In commercial projects, however, interests are often limited to facility and financial protection and insurance of those interests in the event of an adversarial attack. This can be classified as a Business and Properties model. In a Business and Properties model, final decisions with respect to resource allocation, or action on the security recommendations, rest with owners and stake holders, and most often, not with Operations and Security personnel. Without adequate tools for dialogue, these interests are frequently in direct opposition to each other. For example, recommending a vehicle standoff distance that effectively lowers the number of revenue producing parking spaces is an impact on a commercial business. In a public works project, the emphasis would be on protection of the public and not on revenue intake.

Much of the rigor in a comprehensive risk assessment requires a large amount of technical knowledge, often beyond the expertise of decision makers. If a technical model cannot be explained adequately to the occasional non-technical decision maker, information transfer is stopped at these upper levels, and is prevented from being acted upon by Operations and Building Security Personnel. Further, often a return on an investment is not quantifiable if the modifications remain untested until an attack by a malevolent threat.

Infrastructures such as transportation systems and utility services are supplied by a mix of Municipal and Commercial Corporations. Modifications or security upgrades to these infrastructures require extensive coordination and permitting. The line of demarcation is usually the property boundary. If recommendations for security modifications are beyond this property boundary, information transfer again may become problematic without sufficient coordination and appreciation between all parties affected by a disruption or destruction by the potential threat.

6 SUMMARY

The importance for providing and applying a security risk assessment process for all professionals involved in design and construction of buildings and infrastructures is emphasized. A safe and secure environment and accommodation for building occupants, their operations and functions, and their missions is part of the responsibility shared with decision makers, managers, architects, and engineers alike. It has been shown that viable protective means can be implemented where a systematic risk assessment and management approach is applied successfully to evaluate all critical building assets, including major supporting infrastructures. The example application of a security risk methodology described and demonstrated herein can be useful for high-rise buildings, for its supporting primary and backup utility systems, for communications and data centers, for adjoining parking facilities and building access systems, for security and fire protection systems, and for other critical assets that are required to maintain a functional operation.

After a systematic building mission characterization and functional user inventory is completed, the critical assets are determined. This paper subsequently illustrated how the threat scenarios can be determined, how the impacting consequences are evaluated, how the infrastructure vulnerabilities are identified to permit management decisions for the most cost-effective protective counter-measures. This information can assist the decision maker(s) with determining where the appropriate protective strategies can be applied to reduce unacceptable risk levels and/or implement consequences mitigation measures in the event of a potential malevolent attack.

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The Role of Foundation Design in Progressive Collapse of Buildings

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ABSTRACT: While statistical data indicate that risk of progressive collapse in buildings is very low, loss of human life and severe injuries would be significant when a fully occupied multi-story building encounters partial or total failure. As a result of recent terrorist attacks on buildings throughout the world, particularly U.S. owned and occupied buildings, and recent natural hazards like Katrina Hurricane; several U.S. government agencies with large construction programs have developed their own design requirements (GSA 2003; DOD 2005) to provide resistance against progressive collapse. Each agency, however, with its own mission, has adopted different performance objectives for buildings subjected to abnormal loads. Foundation and geotechnical design considerations to provide resistance against progressive collapse are important components of the overall building performance under abnormal loadings. This work discusses the role of geotechnical and foundation system design considerations to reduce the likelihood of progressive collapse of buildings in the event of anomalous loadings. This includes outlining of acceptable risk approach to progressive collapse along with definitions of threads, events control, risk mitigation and practical recommendations for enhancing foundations resistance to progressive collapse.

INTRODUCTION

The 1995 attack on the Oklahoma City Murrah Building was a major thrust to raise government interest in explosion protection for its facilities in the United States and overseas. In response, the federal Interagency Security Committee (ISC) addressed the issue promptly by developing a blast-resistance standard outlining new criteria for design. Subsequently, the horrific structural collapses of Sept. 11, 2001 and the catastrophic damages caused by hurricane Katrina 2005, refocused attention and emphasis on design for extraordinary loads.

In light of these events, two major building owners, the General Service Administration (GSA) and the Department of Defence (DoD) are requiring engineers to consider building security as additional criterion. Even private sector owners and

developers of high profile buildings are taking a serious look at security risks as their buildings may be considered as target of both domestic and international terrorists.

The primary design objective is to save the lives of those who visit or work in these government buildings in the unlikely event that an explosive terrorist attack occurs. In terms of building design, the first goal is to prevent progressive collapse which historically has caused the most fatalities in terrorist incident targeting buildings. Beyond this, the goal is to provide design solutions which will limit injuries to those inside the building due to impact of flying debris and air-blast during an incident. In some cases, secondary objectives may need to be considered such as maintaining critical functions and minimizing business interruption.

Progressive collapse is defined as a situation where local failure of a primary structural element(s) progresses to adjoining members, which in turn leads to additional collapse. Hence, the extent of total damage is disproportionate to the original cause. Different standards describe the term in slightly various ways. ASCE 7-05 defines the term as: "the spread of an initial local failure from element to element, eventually resulting in the collapse of an entire structure or disproportionately large part of it." On the other hand DOD gives the following definition: "A progressive collapse is a chain reaction of failure of building members to an extent disproportionate to the original localized damage. Such damage may result in upper floors of a building collapsing onto lower floors." The GSA 2003 defines the expression as "Progressive collapse is a situation where local failure of a primary structural component leads to the collapse of adjoining members which, in turn, leads to additional collapse. Hence the total damage is disproportionate to the original cause."

Regardless of the definition, blast loading or other abnormal events can cause progressive collapse due to damage of some key element(s) which can either make the structure unstable or trigger the failure of the main portions of the structural system. Explosion generally results in a high-amplitude impulse loading which lasts for a very short period of time and produces high pressure loading. The loading in many situations is local in the sense that only those elements closest to the blast may be directly impacted. Elements far from the blast site may experience little or no direct impact due to sharp dissipation of blast energy with distance. The forces experienced by structural components depend on the size, geometry and proximity of the explosion. Because all of these parameters can vary, it is not easy to accurately predict the force level that a particular structure could experience as a result of an unexpected blast.

Risks of these events cannot be totally eliminated; rather it must be controlled. Building codes are key tools for engineers to manage risk in the interest of public safety. The provisions for foundation and structural design in codes and standard for load combination and safety and partial safety factors addresses risks in building performance. However, risks of blast events have not been part of limit states in previous codes and quite often managed judgmentally. However, the aftermath of recent natural and terrorists disasters has made it clear that judgmental approaches to risk management are not sufficient. Rational approaches to progressive collapse mitigation require risk-informed analysis and assessments.

THREAD DEFINITION

Federal guidelines define three threat levels that delineate blast protection of building structures:

- A high threat level entails a verified high threat of attack. These projects typically are buildings of high importance, buildings whose loss will have high consequences or those that are cultural icons.
- A medium threat level consists of a verified threat of attack. These buildings may be regional symbols, or their loss will highly impact governing powers.
- A low threat level constitutes a suspected threat. These buildings may be regional symbols, or their loss will have moderate consequences.

To gain a systematic approach of investigating terrorists threads, FEMA 427 classifies terrorist threats into the following groups:

Explosive Threats:

- Vehicle weapon
- Hand-delivered weapon

Airborne Chemical, Biological, and Radiological Threats:

- Large-scale, external, air-borne release
- External release targeting building
- Internal release

Although the dominant threat mode may change in the future, bombings have historically been a favorite tactic of terrorists. Ingredients for homemade bombs are easily obtained on the open market, as are the techniques for making bombs. Bombings are easy and quick to execute. Finally, the dramatic component of explosions in terms of the sheer destruction they cause creates a media sensation that is highly effective in transmitting the terrorist's message to the public, as was shown in the UK's car bombs in London and Glasgow June 2007.

The primary threat is mostly a vehicle weapon located along a secured perimeter line surrounding the building (see Figure 1). Depending on the accessibility of the site to vehicles there may be more than one line of defense to consider. The outermost perimeter line is often a public street secured against vehicular intrusion using barriers and with limited secured access points. The size of the vehicle weapon considered outside the perimeter line may vary from hundreds to thousands of pounds of TNT equivalent depending on the criteria used. Weapon sizes vary depending on the specific criteria used and may be obtained from the federal agency client on a need to know basis.

This threat is to be considered on all sides of the building with a public street or adjacent property lines along the secured perimeter line. Because air-blast loads decay rapidly with distance, the highest loads are at the base of the building and decay with height. Benefit of these reduced loads is usually not realized in terms of reduced design requirements except for high rise structures.

Because explosive attacks are expected to remain the dominant terrorist threat in the near future, this work focuses primarily on bomb (explosion) threats, likely targets, and likelihood of occurrence.

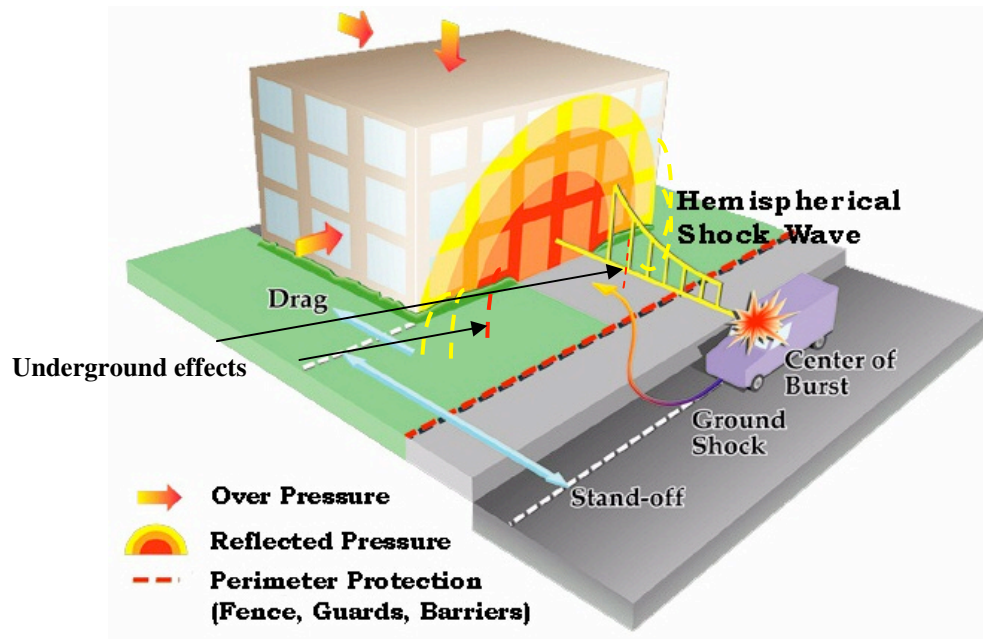


Figure 1. Vehicle Weapon Threads (FEMA 427)

DAMAGE MECHANISMS

Building damages due to a blast event can be categorized into the following groups (see Figure 2):

- Non structural damages; generally taking place on the building envelope.
- Superstructures damages: beams, columns, slabs, ... etc
- Substructures damages: footings, raft, pile, and soil failures.
-

It is notwithstanding that these hazards are interrelated during an explosion event and the occurrence of one may lead to the other with the likelihood of a progressive collapse. Figure 2 below illustrates the relationships among these groups.

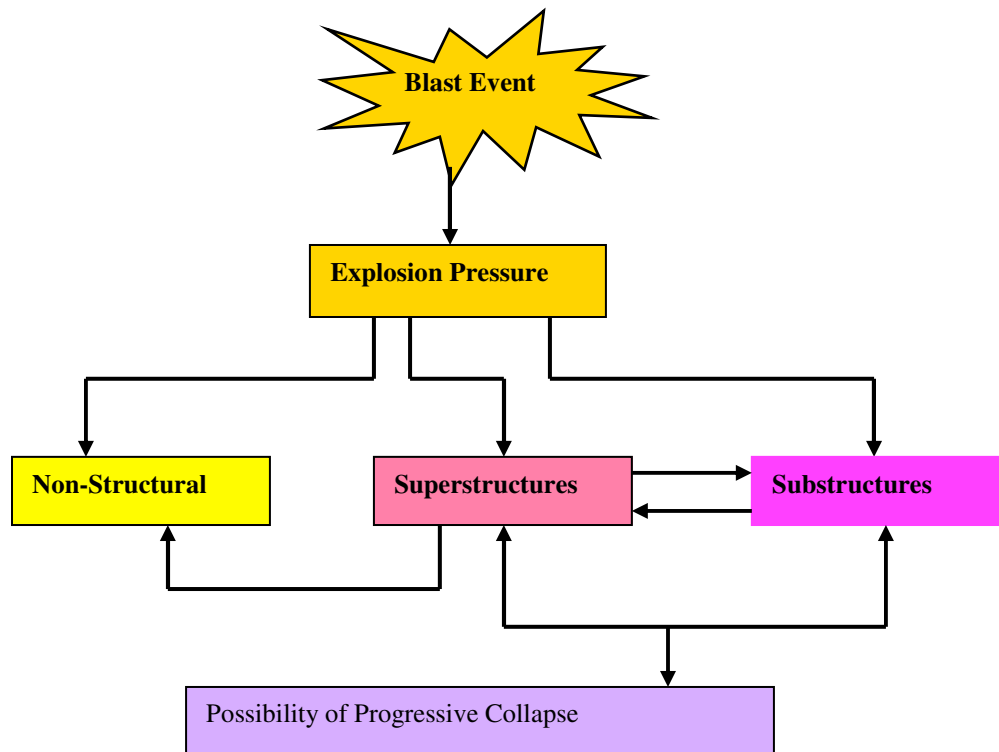


Figure 2. Building's hazards due to Blast Event.

Superstructures

The response of a structure to blast loading is different from its response to typical static and dynamic loads because of the very short duration and extreme pressure loading caused by explosion. According to FEMA 427, the structural damages caused by large exterior explosion can be summarized as follows:

- The pressure wave acts on the exterior of the building and may cause window breakage and wall or column failures;
- As the pressure wave continues to expand into the building, upward pressures are applied to the ceilings and downward pressures are applied to the floors;
- Floor failure is common due to the large surface area upon which the pressure acts; and
- Failure of floor slabs eliminates lateral support to vertical load-bearing elements, making the structure prone to progressive collapse

All of the damages mechanisms described by FEMA(2003), DOD(2005) or GSA(2003) are primarily focused on superstructure effects (Figure 3). Foundations and geotechnical aspects were not considered in these reports. The next section examines different foundation and geotechnical damage scenarios caused by blast effects.

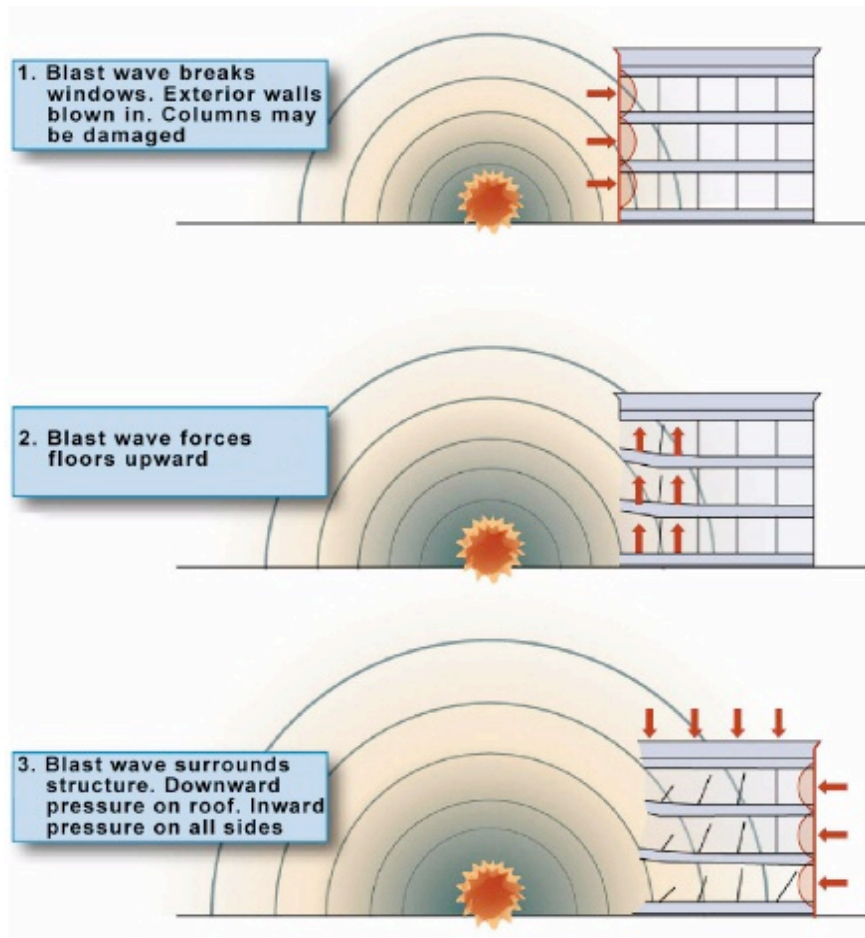


Figure 3. Sequence of Building Damage due to Vehicle Weapon. (FEMA 427)

Substructures

Because there are many potential means by which a local collapse in a specific structure may propagate from its initial extent to its final state, there is no universal approach for evaluating the potential for progressive collapse in buildings. This case specific behavior differentiates progressive collapse from other well defined structural engineering concerns, such as design to resist gravity, wind, seismic or vibration loads. The following general statement can be made, however, of all progressive collapse scenarios: When an initiating event causes a local failure, the resulting failure front will propagate through the building structure until specific structural conditions in its path arrest the progression of failure, or until the remaining structure becomes statically unstable and the entire building collapses. Because progressive collapse is a dynamic event, the failure boundary divides the structure into a zone that has not yet experienced the effects of the progression of failure and the failed portion of the structure. A failure front may propagate laterally, vertically, or both. Blast affects foundations either directly or indirectly or both. In the first case

explosion reaches part of the foundations and causes damages on the footings or piles foundations (see Figure 4). In addition to these damages, excessive dynamic forces impose additional stresses in other existing foundation structures.

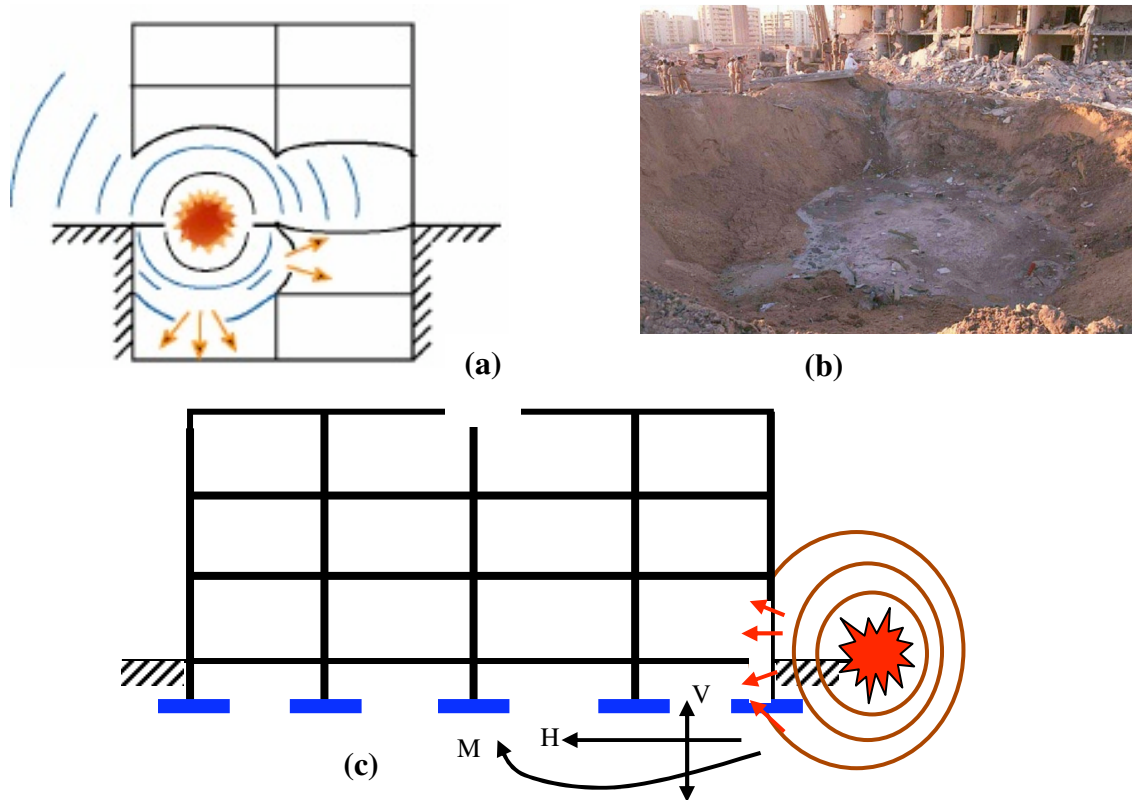


Figure 4. Explosion Affecting directly both Super- and sub-structures.

In figure 4 above, detonation of explosion inside (figure 4a and 4b) or outside (Figure 4c) the building caused damages to the framed superstructure and at the same time foundation was directly affected by the explosion. Furthermore, additional vertical, lateral and vibration forces in the foundation domain due to the blast are generated. As a result, these forces may cause additional drift of the structural frame. The magnitude of the drift and the associated stability issues depends upon the type of the frame (rigid or flexible), type of foundation (single, combined footings, or piles) and the geotechnical properties of the foundation soils (Figure 5) along with the strength of blast.

The second scenario is when explosion damages only parts of the superstructure. Figure 3 illustrates some possible collapses of the superstructure. The failure of columns, beams and slabs will generally be associated of load redistribution and collapse may progress if the remaining elements are at the stage of reaching their ultimate limit states. As a consequence, foundation structures (single footing, combined footings, raft, or pile foundations) will be experience to additional loading conditions. For instance, figure 7 shows the redistribution of forces after the loss of an exterior column. In the next sections, damage mechanisms are discussed for single and combined footings. Next article will investigate damage scenarios for raft and

pile foundations.

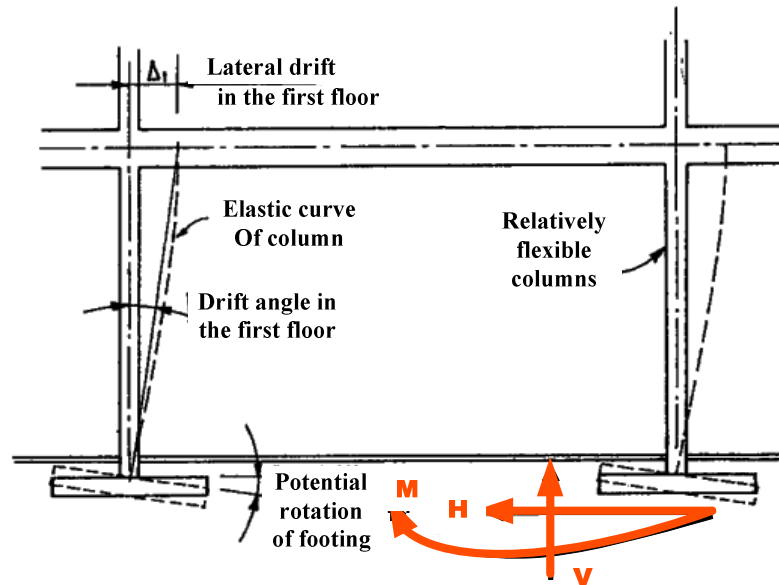


Figure 5. Drift due to blast

Single Footing Foundations

In the case of loss of any external or internal column due to blast, loads on the other adjacent remaining footings will increase due to the load redistribution and changes in tributary areas (see figure 6). Thus, changes in the applied compression (P), shear force (H) and bending moment (M) may exceed the design values causing structural failure of the footing.

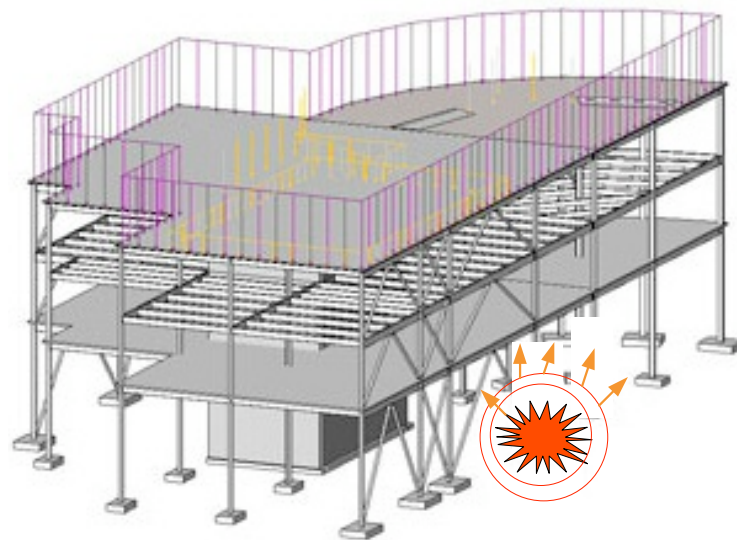


Figure 6. Removal of exterior Columns by detonation of explosives.

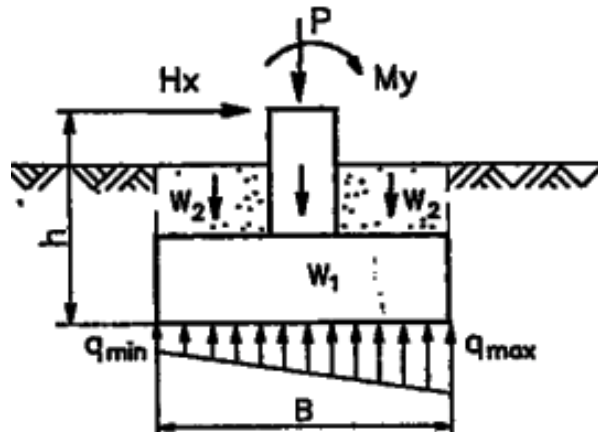


Figure7. Forces acting on a pad foundation.

Other risk of this abnormal loading is excessive settlement or bearing capacity failure. For example, in figure 7 if the value of the soil pressure q_{max} after the load redistribution surpasses the safe contact bearing pressure soil bearing failure takes place and the support provided by the spread footing is practically lost. Soil liquefaction and collapses may also be experienced by the supporting subsurface soils.

Combined Footing

Removal of a column supported by a combined footing due to blast event would cause redistribution of forces on the remaining adjacent columns in the combined footing which in turn affect the contact pressure distribution. Figure 8a illustrates a combined footing designed such that a uniform soil pressure would result in the contact area.

Under an abnormal load case of a blast, the soil pressure distribution after the removal of one or more columns is entirely different than the uniform pressure assumed during the design phase.

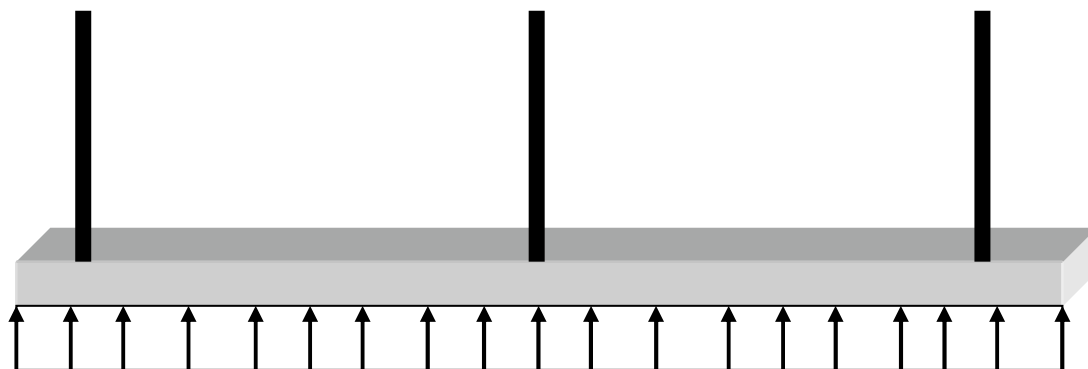


Figure 8a. Soil pressure distribution under a combined footing.

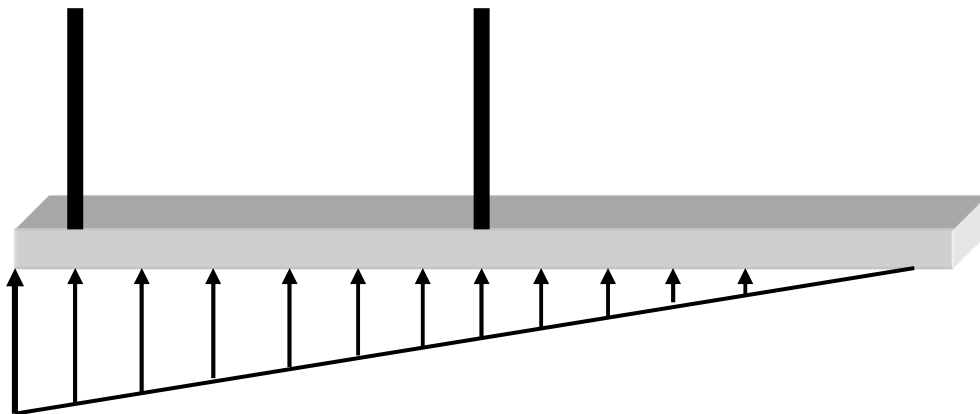


Figure 8b. Redistribution of soil pressure after the loss of an exterior column.

Figure 8b depicts the redistribution of the soil pressure as a result of an exterior column failure. This change in the pressure diagram at one edge may lead to excessive rotation and/or differential settlement of the footing particularly in case of low and medium soil strengths. The redistribution in the contact pressure will also lead to changes in the bending moments and shear forces acting on the footing, that are not accounted for during the design. This creates additional risks of bending and shear failure of the combined footing.

Another abnormal loading case for the combined footing is under applied pressure reversal during explosion events near the foundation. Figure 9a and 9b illustrate the bending surface of the footing before and after such case respectively.

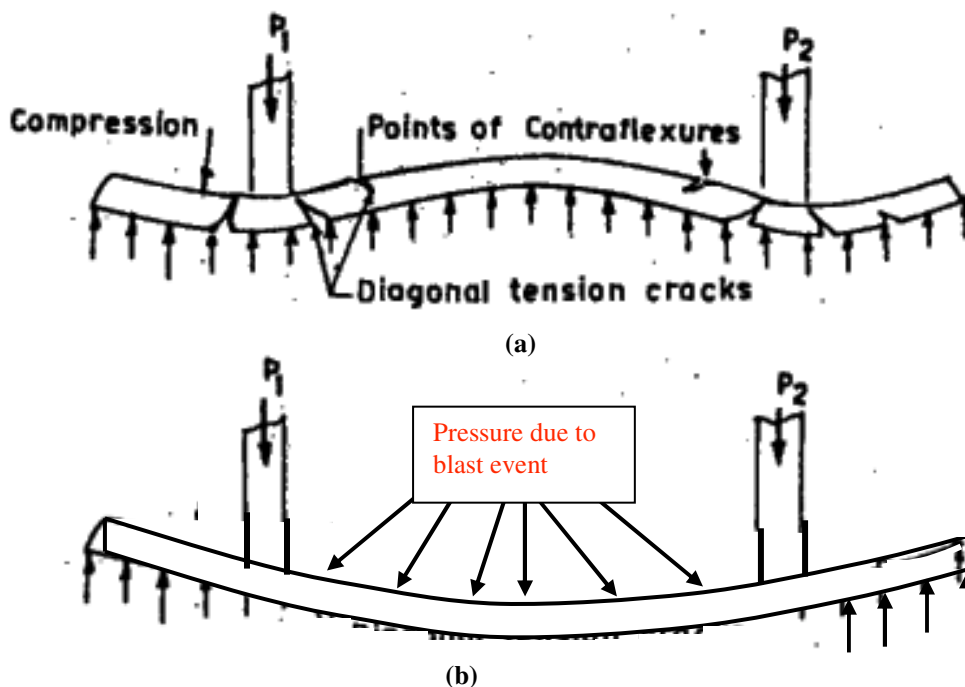


Figure 9. Load reversal effects on combined footing

Basic progressive collapse risk mitigation strategies must be aimed at three basic levels: (i) to prevent the occurrence of intentional abnormal loads through social or political means; (ii) to prevent the occurrence of local significant structural damage that is likely to initiate a progressive collapse; and (iii) to prevent structural system collapse and loss of life through structural design, compartmentalization, development of alternate load paths, alternate exit-ways, and other active and passive measures (Ellingwood et al., 2007). According to the ASCE Standard 7-05, the basis of treating abnormal load combination is given by:

$$P[\text{Collapse}] = P[\text{Collapse}|\text{LD}] P[\text{LD}|\text{H}] P[\text{H}] \quad (1)$$

Where $P[\text{collapse}]$ = probability of structural collapse event, $P[\text{H}]$ = probability of hazard H, $P[\text{LD}|\text{H}]$ = probability of local damage, given that H occurs, and $P[\text{Collapse}|\text{LD}]$ = probability of collapse, given that hazard and local damage both occur. The term $P[\text{Collapse}]$ must be limited to some socially acceptable value through a combination of professional practice and appropriate building authority.

This breakdown of the collapse probability into three components is instructive for focusing attention on appropriate strategies for hazard prevention, withstanding local damage, and absorbing local damage without progressive collapse. Reductions in $P[\text{Collapse}]$ can be achieved by reducing any one, or all three, of the terms in equation (1). The most cost-effective strategy for most buildings is likely to involve some combination of the three. Controlling $P[\text{H}]$ require action such as changes in the building site or access to it (e.g., imposing a minimum stand-off distance through placement of physical barriers and similar devices (Conrath, et al. 1999), or preventing access to certain building zones), by controlling hazardous substances within the building (e.g., limiting the use of natural gas), and by educating the building occupants on the need for caution with dangerous substances or unauthorized access. Often controlling the probability or mean rate of occurrence of the hazard is the most cost-effective route to risk reduction. The main goal of foundation design against the effect of abnormal loads is to mitigate progressive collapse and to obtain an acceptably low probability of a catastrophe involving loss of life and significant economic or social losses. In meeting these performance objectives, a certain amount of damage to the building structure may be incurred and tolerated. Building design is focused on the terms $P(\text{Collapse}|\text{LD})$ and $P(\text{LD}|\text{H})$ in equation (1). The design options include designing the structure to withstand specific abnormal loads or designing and detailing the building to withstand local damage without collapse (alternate load path design). In a “specific local resistance” design strategy, the focus is on controlling $P(\text{Collapse})$ by limiting $P(\text{LD}|\text{H})$, that is, to minimize the likelihood of initiation of damage that may lead to progressive collapse. Normally, this requires that a specific threat be identified in order to determine the stress placed on the structural member, component or subsystem. In an “alternate load path” design strategy, the focus is on controlling $P(\text{Collapse})$ by limiting $P(\text{Collapse}|\text{LD})$. This may require that the design team identify a specific threat (e.g., a detonation that would remove one or more load-bearing members from the perimeter frame). Alternatively, the design team might postulate certain structural

damage mechanisms, without regard to specific cause, and require that the system as a whole absorb such damage without progressive collapse (Ellingwood et. Al, 2007). In any event, it is in minimizing these two probabilities that the science and art of the engineer becomes dominant.

As shown previously, foundation and geotechnical considerations play an important role in minimizing $P[\text{Collapse}|LD]$ and $P[LD|H]$. For instance, establish criteria to ensure that both the soil and the foundations of load-bearing elements adjoining the removed columns were not overloaded. This is achieved by performing strength check on the remaining foundation structure, especially the adjacent columns using a realistic extreme event loads. Another approach to reduce these two probabilities is by the addition of an alternate load path. This means providing capability for the structure above the first level at grade on the exterior to overpass or redistribute loads after the loss of a column at a lower level. Also, introducing two-way behavior in the superstructures that have been designed with planar systems can create alternate load paths. Such modifications force foundation systems to engage the resistance of more components when one or more critical elements have been damaged.

RECOMMENDATIONS

Foundation and geotechnical engineers must determine which hazardous events and damage scenarios to consider and what are the acceptable probabilities and consequences. Collapse prevention begins with awareness by architects, planners and engineers that design of foundations to guard against collapse is vital enough to be routinely considered in structural design. Features to improve general structural safety against progressive collapse can be incorporated into common buildings at affordable cost. At a higher level, design for progressive collapse can be accomplished by the alternate path method (i.e. design for removal of specific elements) or by direct design of components for air-blast loading or by the indirect method of prescribing design features, which promote redundancy and ductility. The following practical considerations related to geotechnical and foundation design are suggested to enhance resistance against progressive collapse of buildings:

- Loss of a column and/or footing will increase distress to other adjacent columns as load redistributes, verify that ultimate bearing capacity is not exceeded.
- Excessive settlement and foundation rotation might be critical near blast location and need to be minimized by increasing footing thickness and width.
- Provide redundancy by using wider footings to reduce bearing pressure on ground foundation.
- Provide thicker footings to improve resistance to rotation and punching failure at column interface.
- Provide extra capacity for load reversal by adding adequate bottom and top reinforcement in footings to resist any load reversal during detonation of explosives (Figure 9).

- Tie footings together with strip footings or grade beams to improve load redistribution.
- Column connections to the foundation should be checked for additional flexure that might result from load redistribution as a consequence of the loss of a structural element.
- Avoid ground liquefaction during blast event by checking soil liquefaction properties to prevent settlement, tilting, instability and rupture of the structure.

Designers must also note that measures taken to mitigate explosive loads may reduce the structure's performance under other types of loads, and therefore an iterative approach may be needed to achieve an optimum solution. As an example, increased mass generally increases the design forces for seismic loads, whereas increased mass generally improves performance under explosive loads (FEMA 427). For more complex structures, the foundation engineer may employ finite-element numerical time integration techniques and/or explosive testing. Because the design process against progressive collapse is a sequence of iterations, the cost of analysis must be justified in terms of benefits to the project and increased confidence in the reliability of the outcomes.

CONCLUSIONS

The role of geotechnical and structural foundation systems considerations in minimizing the likelihood of progressive collapse of buildings in the event of anomalous loadings in form of explosion is shown to be profound. Thus, prevention of progressive collapse in case of impulse-type high-amplitude loading can be introduced as one of the overt imperatives in foundation engineering practice. Design and analysis of foundation and geotechnical systems against progressive collapse is recommended to include acceptable risk approach along with definitions of threads, events control, and risk mitigations. Furthermore, practical recommendations for enhancing redundancy and foundations resistance against progressive collapse of buildings are provided.

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NUMERICAL ANALYSIS OF CURVED THIN BEAMS ON WINKLER FOUNDATION

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ABSTRACT

This research, deals with the linear elastic behavior of curved thin beams resting on Winkler foundation with both compressional and tangential resistances. Thin beam theory is extended to include the effect of curvature and externally distributed moments under static conditions.

The computer program (**CBFFD**) coded in **fortran_77** is developed to analyze curved thin beams on Winkler foundation by Fourier series and finite difference methods. The results from these methods are plotted with other solutions to compare and check the accuracy of the used methods.

KEYWORDS

Curved thin beams, numerical analysis, and Winkler foundation.

INTRODUCION

The object of this research is to analyze curved thin beam using finite difference and Fourier series methods. The beam is resting on elastic foundation with Winkler frictional and compresional resistances, and loaded generally (both transverse distributed load and distributed moment).

The linear elastic behavior of curved thin beams on elastic foundations is considered. The governing differential equation of curved thin beams (in terms of w only) is developed and converted into finite differences. A computer program in (Fortran language) is developed. This program assembles the finite difference equations to obtain a system of simultaneous algebraic equations and then the solution is obtained by using Gauss elimination method. The deflections and rotations for each node are obtained. The shear and moment are obtained by simple substitutions of the deflections into the finite difference equations of moment and shear. Also, this program used Fourier series method to solve the governing differential equation for simply supported beam and obtain the deflection, moment and shear. The obtained solutions compared with available results to check the accuracy of the used methods.

Curved beams are one-dimensional structural elements that can sustain transverse loads by the development of bending, twisting and shearing resistances in the transverse sections of the beam. It's extensively used in engineering and other fields since such beams have many practical applications. The curved beam elements on elastic foundation would be helpful for the analysis of ring foundation of structures such as antennas, water towers structures, transmission towers and various other possible structures and superstructures.

These are review of early studies on curved beam.

Volterra (1952), analyzed the deflections of circular beams resting on elastic foundations. The beam was loaded by symmetric concentrated forces acting in a plane perpendicular to the plane of the original curvature of the beam. The foundation is supposed to react following the classical Winkler hypothesis, i.e. the reaction forces due to the foundation are proportional at every point to the deflection of the beam at that point.

Rodriguez (1961), solved the three-dimensional bending of a ring (curved beam in the form of a complete circle) of uniform cross sectional area and supported on a transverse elastic foundation.

Dasgupta and Sengupta, (1988), suggested a formula for the analysis of a horizontal curved beam by using three node isoparametric finite elements. The formulation presented was general and the method, therefore, may be utilized for straight beams as well. The beam was with or without an elastic base throughout its length.

BASIC DIFFERENTIAL EQUATION

According to the small deflection theory and linear stress-strain relationships, an equation for the bending of curved thin beams is derived using the following assumptions:-

- 1) Plane cross-section before bending remains plane after bending.
- 2) The cross-section remain normal to the neutral plane after bending

The governing equation of bending of curved thin beams on Winkler compressional and frictional resistances in terms of deformation functions (w) is;

$$\frac{d^4 w}{d\theta^4} + \frac{K_\theta R^2 h^2}{4EI} \frac{d^2 w}{d\theta^2} + \frac{K_z R^4}{EI} w = \frac{q \cdot R^4}{EI} \quad (1)$$

Exact Solution

The exact solution of the governing differential equation (putting K_θ and K_z equal to zero) for the case of simply supported beam under uniform load gives the following equations for deflection, moment and shear :

$$w(\theta) = \frac{q \cdot R^4}{192EI} (8\theta^4 - 8\pi\theta^3 + \pi^3\theta) \quad (2)$$

$$M(\theta) = -\frac{q \cdot R^2}{4} (2\theta^2 - \pi\theta) \quad (3)$$

$$Q(\theta) = -\frac{q \cdot R}{4} (4\theta - \pi) \quad (4)$$

Fourier Series Solution

Fourier series solution of the governing differential equation for the case of simply supported beam under uniform load and on Winkler Compression and friction foundation gives the following equations for deflection, moment and shear:

$$w(\theta) = \frac{4q \cdot R^4}{\pi EI} \sum_{m=1,3} \frac{\sin\left(\frac{m\pi\theta}{\phi}\right)}{m \left[\left(\frac{m\pi}{\phi}\right)^4 - \frac{K_\theta \cdot R^2 \cdot h^2}{4EI} \left(\frac{m\pi}{\phi}\right)^2 + \frac{K_z \cdot R^4}{EI} \right]} \quad (5)$$

$$M(\theta) = \frac{4q \cdot R^2 \pi}{\phi^2} \sum_{m=1,3} \frac{m \cdot \sin\left(\frac{m\pi\theta}{\phi}\right)}{m \left[\left(\frac{m\pi}{\phi}\right)^4 - \frac{K_\theta \cdot R^2 \cdot h^2}{4EI} \left(\frac{m\pi}{\phi}\right)^2 + \frac{K_z \cdot R^4}{EI} \right]} \quad (6)$$

$$Q(\theta) = \frac{4q \cdot R \pi}{\phi^2} \sum_{m=1,3} \frac{m \cdot \left(\frac{m\pi}{\phi}\right) \sin\left(\frac{m\pi\theta}{\phi}\right)}{m \left[\left(\frac{m\pi}{\phi}\right)^4 - \frac{K_\theta \cdot R^2 \cdot h^2}{4EI} \left(\frac{m\pi}{\phi}\right)^2 + \frac{K_z \cdot R^4}{EI} \right]} - \frac{K_\theta h^2}{4R} \left[\frac{4q \cdot R^4}{\pi EI} \sum_{m=1,3} \frac{\left(\frac{m\pi}{\phi}\right) \cos\left(\frac{m\pi\theta}{\phi}\right)}{m \left[\left(\frac{m\pi}{\phi}\right)^4 - \frac{K_\theta \cdot R^2 \cdot h^2}{4EI} \left(\frac{m\pi}{\phi}\right)^2 + \frac{K_z \cdot R^4}{EI} \right]} \right] \quad (7)$$

Finite Difference Equations For Curved Thin Beams With Winkler Friction and Compression

The governing differential equation for curved thin beams on elastic foundations represented by a Winkler model for frictional restraints is converted into finite differences and produced for an interior node i :

$$\left(\begin{matrix} i-2 \\ \textcircled{i} \end{matrix} \right) \left(\begin{matrix} i-1 \\ -4 \frac{K_\theta R^2 h^2 \Delta\theta^2}{4EI} \end{matrix} \right) \left(\begin{matrix} i \\ 6 \frac{K_\theta R^2 h^2 \Delta\theta^2}{2EI} + \frac{K_z R^4 \Delta\theta^4}{EI} \end{matrix} \right) \left(\begin{matrix} i+1 \\ -4 \frac{K_\theta R^2 h^2 \Delta\theta^2}{4EI} \end{matrix} \right) \left(\begin{matrix} i+2 \\ \textcircled{1} \end{matrix} \right))_w = - \frac{q_i R^4 \Delta\theta^4}{EI} \quad (8)$$

APPLICATIONS

Free Curved Beam

A ring foundation of ($E=20700000 \text{ kN/m}^2$, $\nu=0.15$), having a radius of ($R=7.62945 \text{ m}$), width ($b=0.762 \text{ m}$), thickness ($h=0.762 \text{ m}$). The ring beam carries four equal column loads (perpendicular to the ring), each column load ($P=667.5 \text{ kN}$). The ring beam is resting on an elastic foundation which is represented by Winkler model for compressional restraint with the coefficient ($K_z= 13500$

kN/m^3), as shown in Figure 1. This problem was solved by **Dasgupta and Sengupta, (1988)** by using finite element method. In the present study, the same problem is solved by using the finite difference method. The results of deflections are plotted with the results of **Dasgupta and Sengupta, (1988)** as shown in Figure 2. The figure shows acceptable agreement. The percentage of the difference between the maximum deflections for **Dasgupta and Sengupta,(1988)** solution and the present study is equal to (5%).

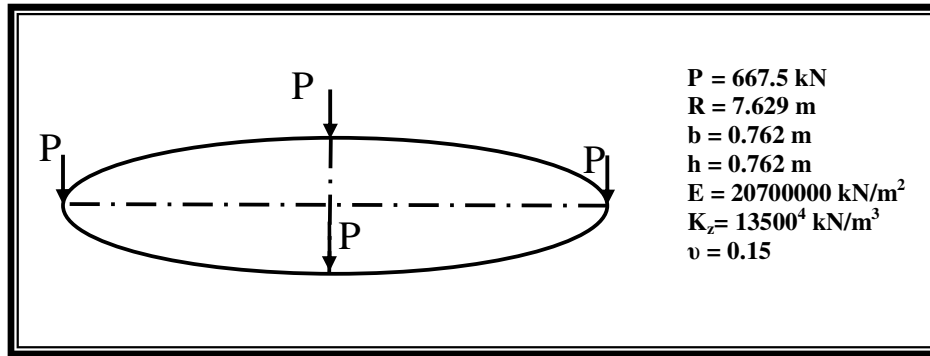


Figure 1. Ring beam on a Winkler Foundation under four concentrated loads.

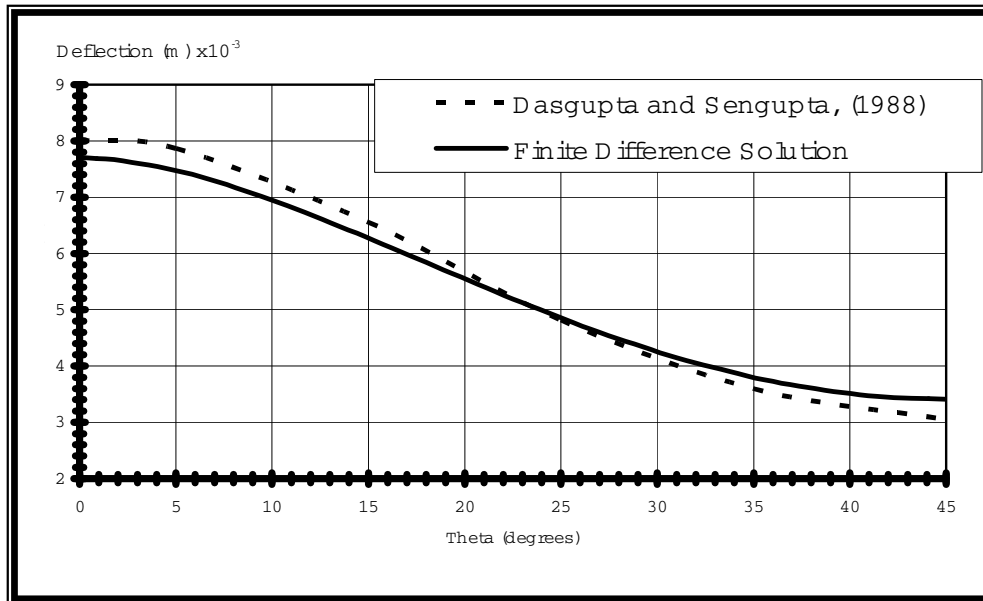


Figure 2. Deflection curves for free curved beam resting on a Winkler foundation.

Simply Supported Curved Beam

A curved beam of ($E=20000000 \text{ kN/m}^2$, $\nu=0.15$), having a radius of ($R=2.00 \text{ m}$), width ($b=0.4 \text{ m}$), thickness ($h=0.4 \text{ m}$) is considered. The beam carries a uniform load ($q= 35\text{kN/m}$). The beam is resting on an elastic foundation, which is represented by Winkler model for compressional restraint with the coefficient

($K_z = 10000 \text{ kN/m}^3$) and horizontal restraint with the coefficient ($K_\theta = 20000 \text{ kN/m}^3$), as shown in Figure 3.

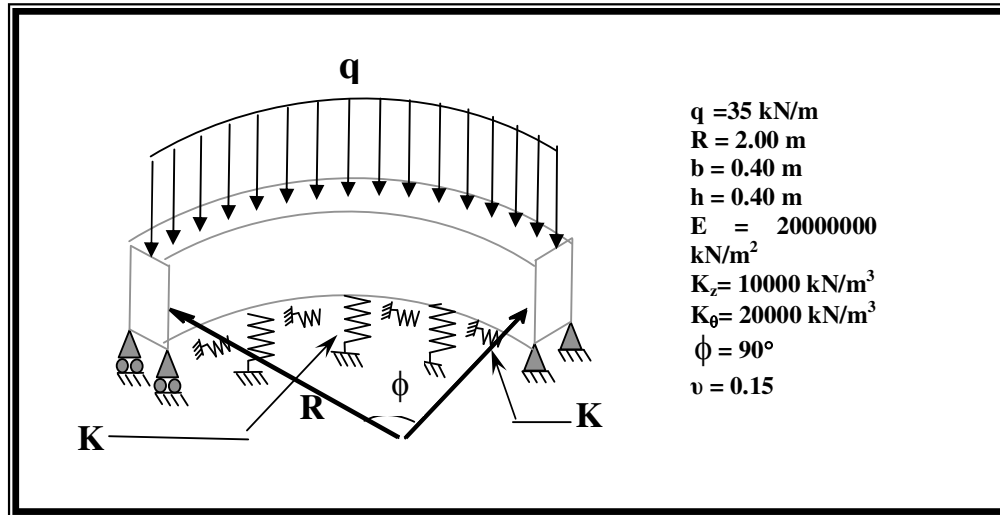


Figure 3. Simply supported curved beam resting on a Winkler foundation under a uniform load.

This problem was solved using exact solution and finite difference method assuming the beam without elastic foundation. The results of deflections, moments and shears are plotted with the results obtained by finite differences as shown in figures 4, 5 and 6. The figures show good agreement with percentage differences with deflection, moment and shear (0.2%, 0.02% and 5%) respectively.

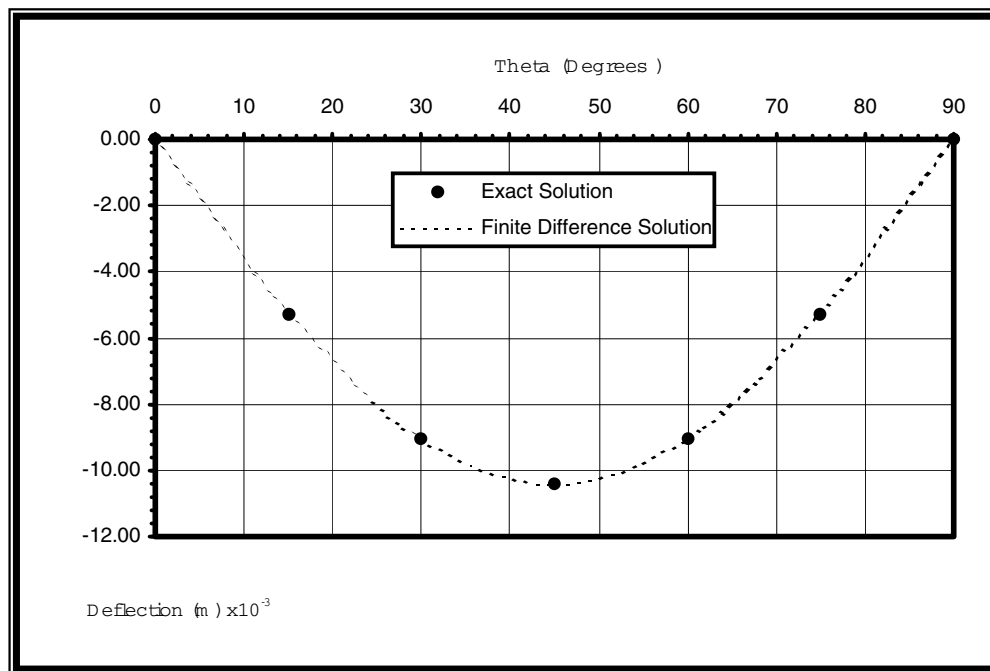


Figure 4. Deflection curves for simply supported curved beam.

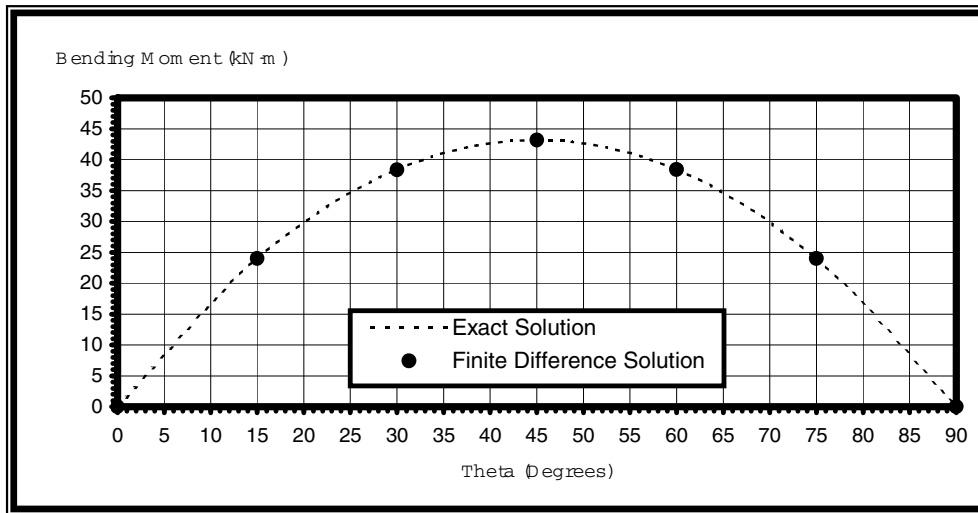


Figure 5. Bending moment diagrams for simply supported curved beam.

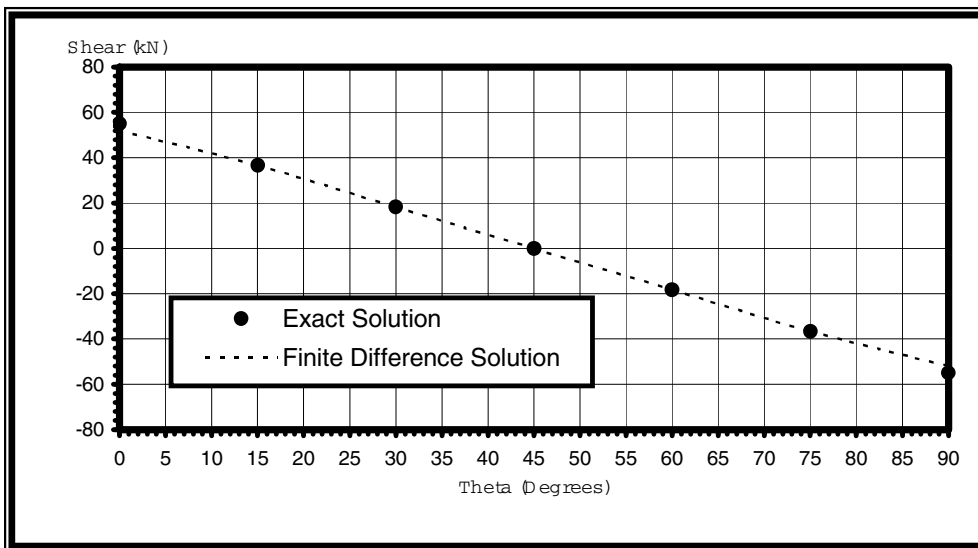


Figure 6. Shear force diagrams for simply supported curved beam.

The problem of simply supported curved beam on Winkler model for compressional restraint with the coefficient ($K_z= 10000 \text{ kN/m}^3$) and horizontal restraint with the coefficient ($K_\theta=20000 \text{ kN/m}^3$), was solved by Fourier series and finite difference methods. The results of deflections, moments and shears obtained by the two methods are plotted together as shown in figures 7, 8 and 9. The figures show good agreement with percentage differences with deflection, moment and shear (0.9%, 0.9% and 3%) respectively.

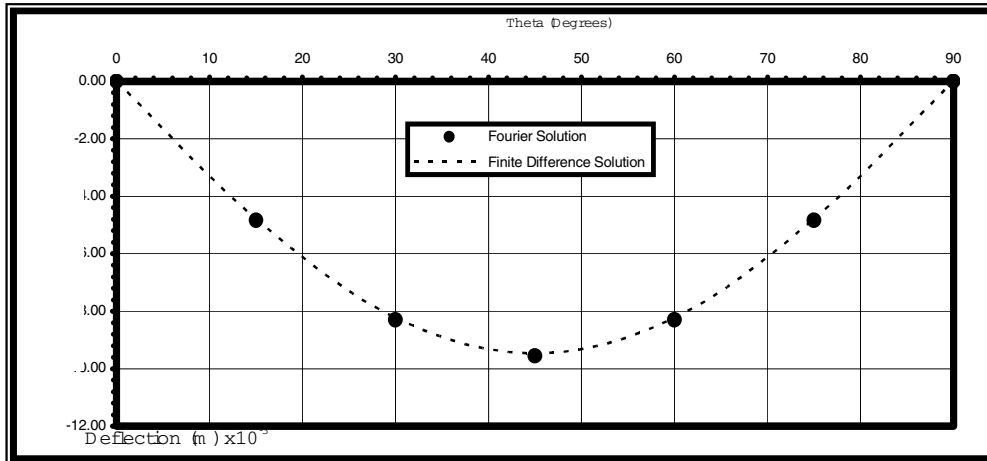


Figure 7. Deflection curves for simply supported curved beam.

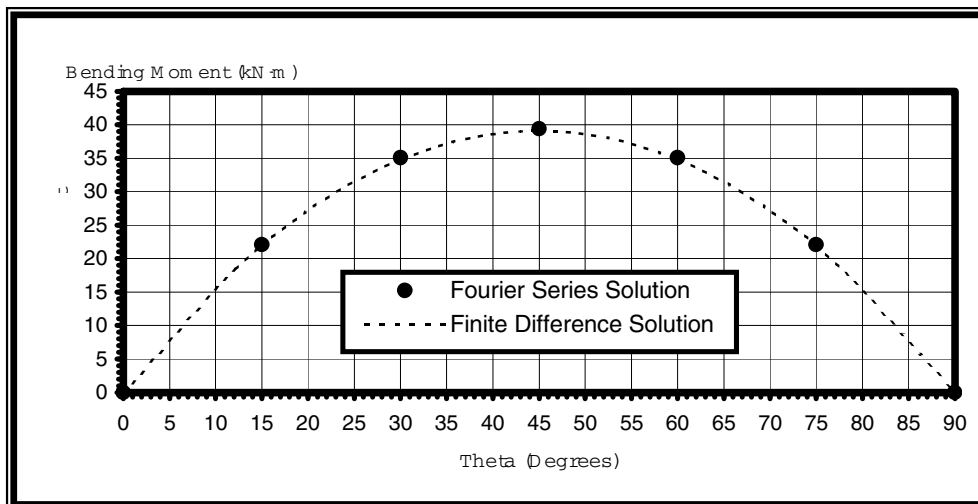


Figure 8. Bending moment diagrams for simply supported curved beam on Winkler foundation.

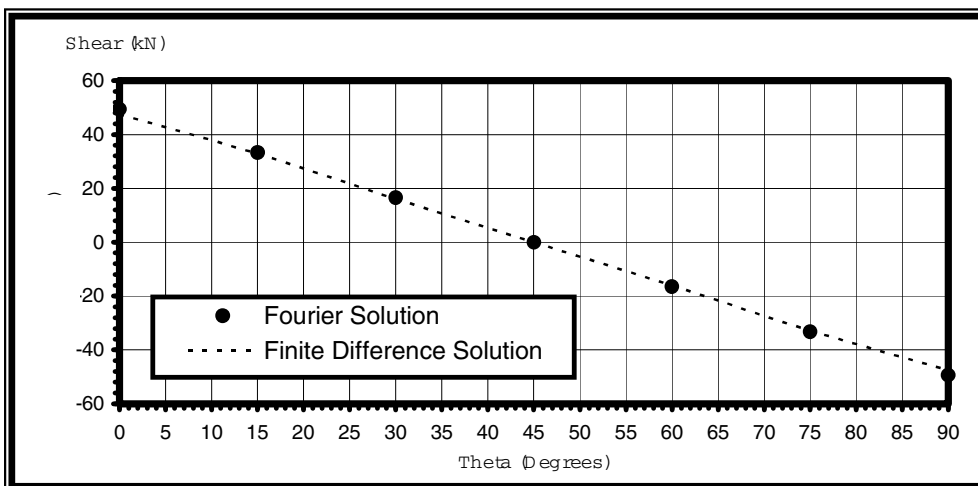


Figure 9. Shear force diagrams for simply supported curved beam on Winkler foundation.

Parametric Study

A parametric study is performed to show the influence of several important parameters on the behavior of simply supported curved beam. The values of the vertical and horizontal subgrade reactions (10000 kN/m^3 and 20000 kN/m^3) are considered.

The effect of increasing the depth of beam on deflection is shown in Figure 10. From this Figure, the mid span deflection will decrease at decreasing rate as the beam depth increased. It was found that by increasing the depth from (0.1m to 0.4m), the mid span deflection for the simply supported curved beam under uniform loading is decreased by (90%).

The effect of increasing the beam width on deflection is shown in Figure 11. From this Figure, the mid span deflection will decrease at decreasing rate as the beam width increased. It was found that by increasing the width of the simply supported curved beam from (0.2m to 0.6m), the mid span deflection for the curved beam is decreased by (66%).

The effect of increasing the vertical and horizontal subgrade reactions (K_z) and (K_θ) on deflection are shown in figures 12 and 13. From these figures, the mid span deflection will decrease at a decreasing rate as the vertical subgrade reaction is increased. While the mid span deflection will decrease almost linearly as the horizontal subgrade reactions is increased. It was found that by increasing the vertical and horizontal subgrade reactions for the simply supported curved beam from (10000 kN/m^3 to 100000 kN/m^3) for the vertical, and from (10000 kN/m^3 to 100000 kN/m^3) for the horizontal, the mid span deflection decreased by (49%), (4%) respectively.

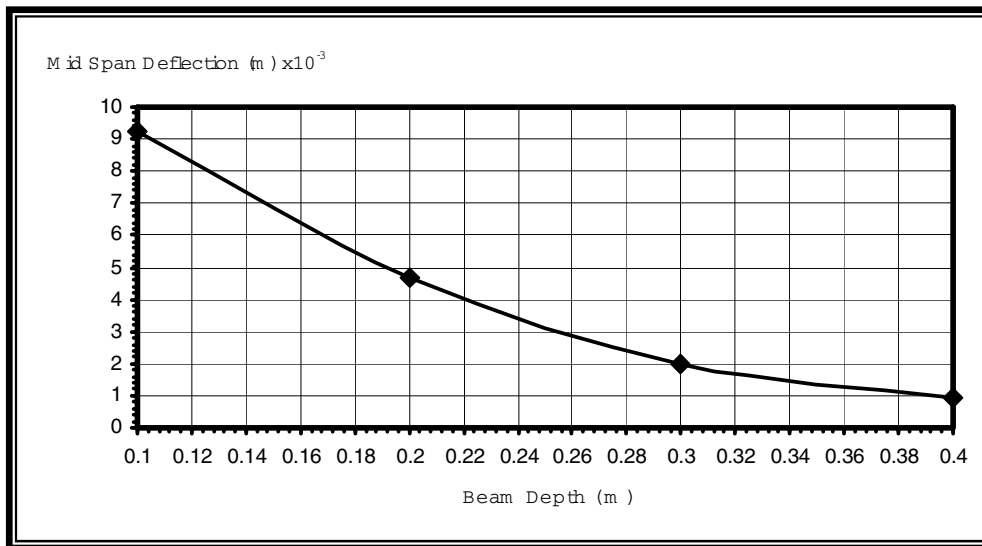


Figure 10. Effect of beam depth on the mid span deflection.

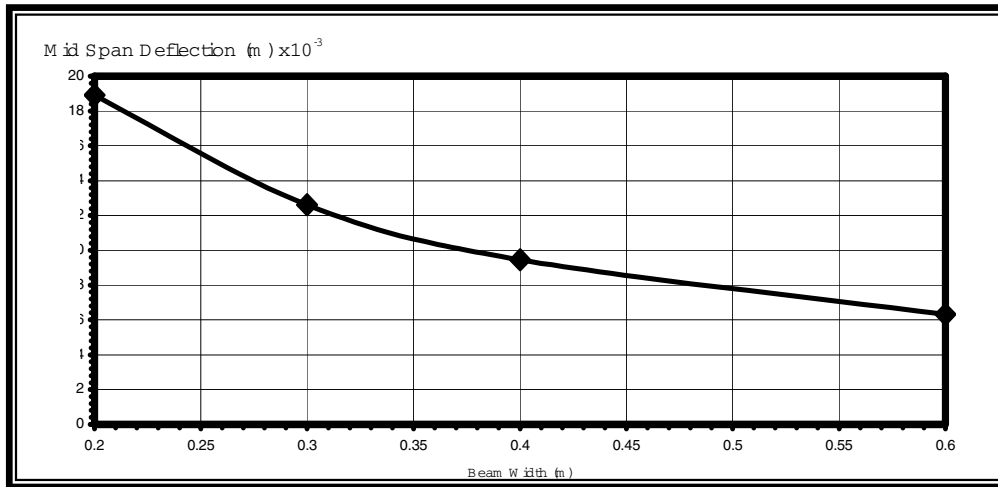


Figure 11. Effect of beam width on the mid span deflection.

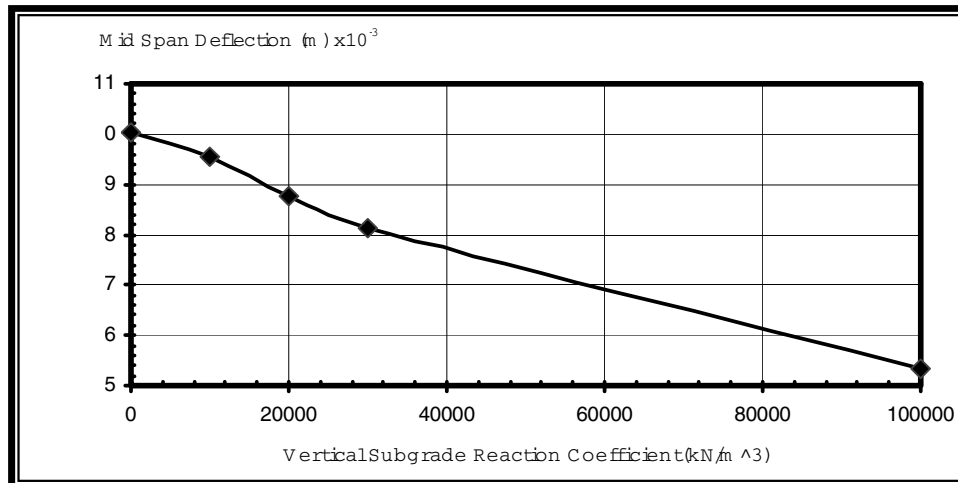


Figure 12. Effect of vertical subgrade reaction on mid span deflection.

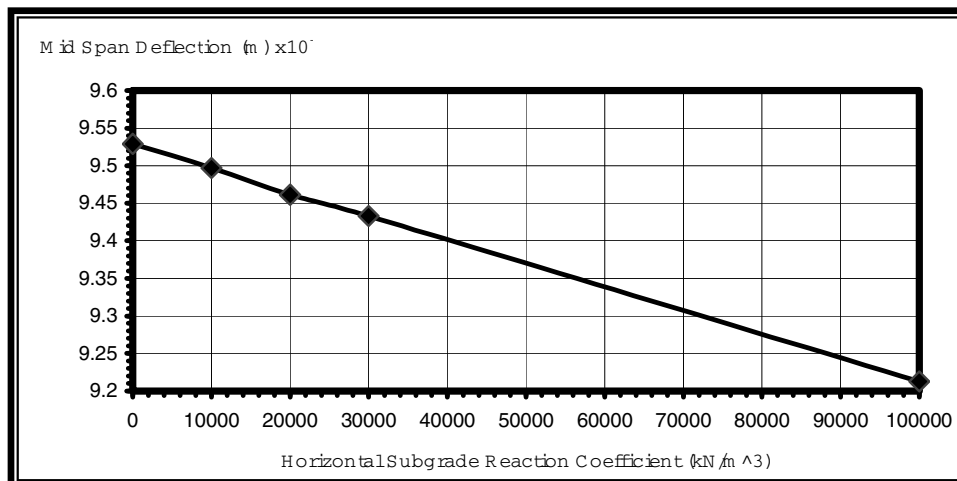


Figure 13. Effect of horizontal subgrade reaction on mid span deflection.

SUMMARY AND CONCLUSION

1. Original Euler-Bernulli thin beam theory is extended to include the effects of externally distributed moments along the curved thin beam and modeled by finite difference and Fourier series methods.
2. When the width of the beam increases, the deflection will decrease because the stiffness of the increased. It was found that by increasing the depth from (0.2m to 0.6m), the mid span deflection for the simply supported curved beam under uniform loading is decreased by (66%).
3. When the depth of beam increases, the deflection will decrease because the stiffness of the increased. It was found that by increasing the depth from (0.1m to 0.4m), the mid span deflection for the simply supported curved beam under uniform loading is decreased by (90%).
4. The deflection will decrease at a decreasing rate as the vertical subgrade reaction is increased. While the mid span deflection will decrease almost linearly as the horizontal subgrade reactions is increased. It was found that by increasing the vertical and horizontal subgrade reactions for the simply supported curved beam from (10000 kN/m³ to 100000 kN/m³) for the vertical, and from (10000 kN/m³ to 100000 kN/m³) for the horizontal, the mid span deflection decreased by (49%), (4%) respectively.

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Ambient Environmental Effects On Experimental Modal Analysis

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Abstract

Experimental modal analysis is an effective, nondestructive method to extract several important physical characteristics of a structural system, such as the mode shapes (deflected form), the natural frequencies of the modes that are excited, and an approximation for the component stiffness. Through past experiences, the authors have identified that the ambient environmental factors, such as temperature and humidity, have an effect on obtained natural frequencies. This finding of the authors agrees with findings published elsewhere. In order to further investigate the issue and quantitatively determine the effect of these factors on modal behavior, this study applies experimental modal analysis on a model timber vault. The model dome is constructed at the Peter Kiewit Institute-Structures Laboratory at the University of Nebraska, and stored outside where it is subject to fluctuations in ambient environmental conditions. Experimental modal analysis is performed over a four-month period on the model, and the ambient temperature and relative humidity are recorded. A database is created that includes temperature, relative humidity, measured mode shapes, and natural frequencies. The effects of temperature and humidity on the mode shapes are considered individually and collectively, and correction factors are presented.

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Introduction

A one-half scale model of a thin tile masonry tumbrel dome located in the Nebraska State Capitol is constructed at the Peter Kiewit Institute-Structures Laboratory at the University of Nebraska (Figure 1). The model dome provides a testable system for multiple applications of experimental modal analysis (EMA) with varying parameters. The model dome also provides the opportunity to increase understanding of EMA on complex masonry structures and allows for the development of more accurate *in situ* material property extraction.



Figure 1. Dome in Nebraska State Capitol (left), Model Dome Test Specimen (right).

During experiments carried out over the period of a year, it was noticed that the natural frequencies obtained from EMA display large variations on different days. With motivations from these observations, a parametric study is planned to quantitatively link the change in temperature and relative humidity to changes in experimental modal analysis. Experimental modal testing is repeated five times over a four-month span. At each test, along with the modal test data, the ambient temperature and relative humidity at the time of testing are recorded. The natural frequencies are plotted against the measured temperature and humidity, and a linear relationship between is observed for both parameters. Based on this relationship, modification coefficients for the natural frequency are developed.

Testing Procedure

The model dome is labeled using a 64 node radial quadrant system. Low frequency excitations are then applied to the structure on 45 of the 64 points with an instrumented hammer (PCB Piezotronics ICP[®] Impulse Hammer Model 086D20). The remaining 19 nodes are located along the lower boundaries and no impact is applied because they are support points and are not expected to move during modal deformations. The force sensor in the hammer measures the load input, and the vibrations generated in the structure are measured using a seismic shear

accelerometer (PCB Piezotronics model 393A03 seismic accelerometer with a sensitivity of approximately 1 V/g). A SIGLAB[®] dynamic signal analyzer is used to collect fixed response modal testing data. After the test data is gathered, the acceleration frequency response function (FRF) is plotted and analyzed using STAR Modal and system characteristics are extracted. Five sets of experimental modal data are obtained, with the fixed response measured from two points: a quarter-point and the crown. Three sets of data were obtained with the sensor at the quarter point (sensor location 1) and two sets were obtained with the sensor at the crown (sensor location 2). The placement of the sensor at both locations is necessary to capture different modes. The testing set up with the sensor locations is shown in Figure 2.

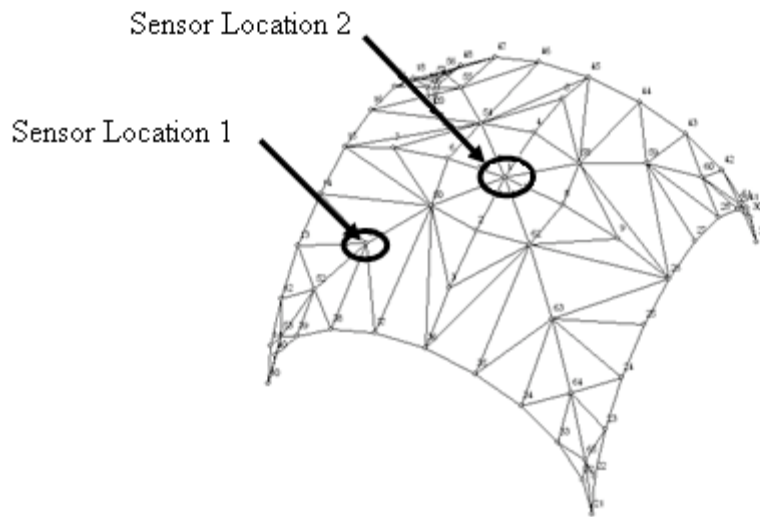


Figure 2. Modal Testing Node Configuration.

Test Results

The first mode natural frequencies from the experimental modal testing are shown in Table 1 along with the measured temperature and relative humidity.

Table 1. Experimentally Obtained First Modes.

Test Number	Frequency (Hz)	Temperature (°C)	Relative Humidity (%)
1	50.6	45.5	16
2	51.5	45.1	14
3	40.7	34.0	38
4	43.5	31.9	28
5	47.1	41.5	27
Average	46.7	39.6	25

The frequencies and corresponding temperatures and humidity are plotted for the observed modes. From the temperature plot (Figure 3) it is observed that a linear

relationship exists for the middle three data sets while the two end data sets do not appear to fit in this trend. The explanation for this may be in the inherent error of the modal data; however, it is also possible that temperature only affects the modal data for a range of temperatures. Utilizing the three middle sets of data, a temperature modification coefficient (C_T) of 1 Hz/°C is calculated.

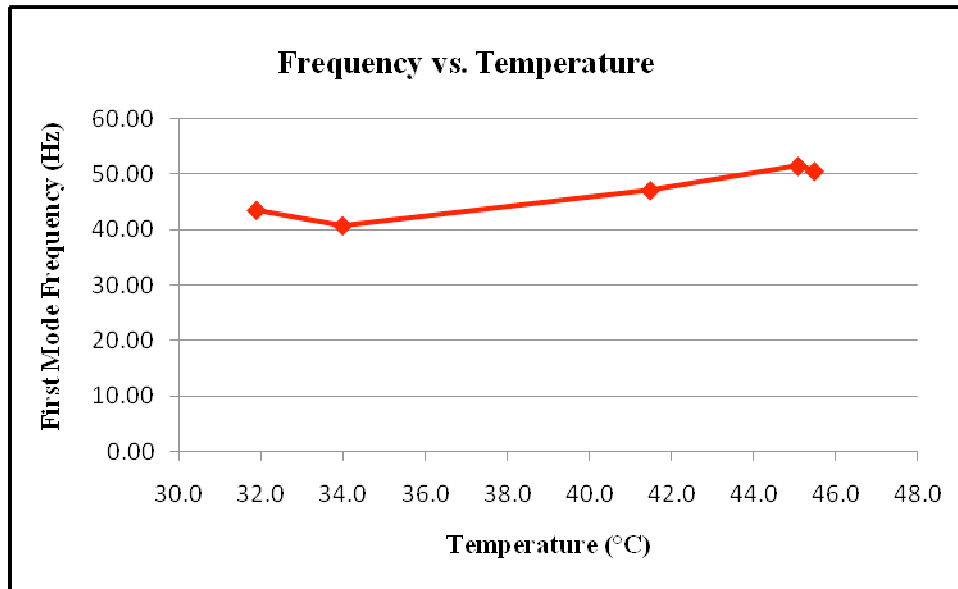


Figure 3. First Mode Frequency versus Measured Temperature.

Figure 4 illustrates the relationship between frequency and relative humidity, where a negative linear relationship between the two parameters is observed. Thus, as humidity increases, the measured natural frequency decreases. The linear relationship was determined by applying trend-lines of different types (linear, exponential, logarithmic) and evaluating the regression analysis of each line. The linear trend-line demonstrated a fit with the least regression. Calculating the slope of the line between the two end data points, a relative humidity modification coefficient (C_{RH}) of -0.5 Hz/% is derived.

Review of existing literature reveals supporting observations by other researchers. While testing segmental masonry retaining wall units, Chan *et al.* measured the natural frequencies of the units at different moisture contents, and found that as the moisture content of the brick increased, the natural frequency decreased (Chan *et al.* 2007).

The inversely proportional relationship between relative humidity and measured natural frequencies can best be explained using basic vibrations theory (Equation 1).

$$\omega = \sqrt{k/m} \tag{1}$$

Where, ω is the natural frequency, k is the stiffness, and m is the mass.

Due to the material properties of masonry, it is more likely that the humidity will affect the mass of the system as opposed to the stiffness. With increasing humidity, moisture content and the mass of the structure will increase. Assuming the stiffness remains constant; this increase in mass explains the decrease of the natural frequency.

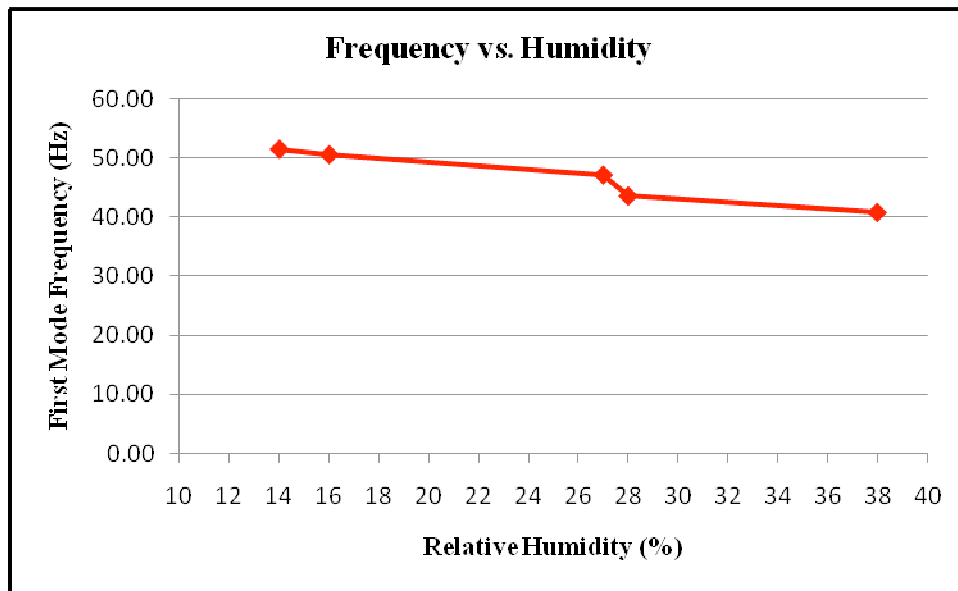


Figure 4. First Mode Frequency versus Measured Relative Humidity.

The proposed modification coefficients for temperature and humidity can be verified using the parametric data from test 2 to predict the natural frequency of test 5. This is done by applying C_T and C_{RH} to the differences in temperature and relative humidity and calculating a predicted natural frequency. The predicted frequency can then be compared to the measured frequency. The EMA results from the two tests are shown in Table 2.

Table 2. Experimentally Obtained Second Modes

Test Number	Frequency (Hz)	Temperature (°C)	Relative Humidity (%)
2	97.0	45.1	14
5	90.0	41.5	27

The difference in temperature and relative humidity between the two tests is - 3.6 °C and 13% respectively. The predicted natural frequency is then calculated as:

Temperature: $97 \text{ Hz} + 1.0 \text{ Hz}/^\circ\text{C} * (-3.6 \text{ }^\circ\text{C}) = 93.4 \text{ Hz}$ (3.6% error compared to 90Hz)

Relative Humidity: $97 \text{ Hz} + -0.5 \text{ Hz}/\% *(13 \%) = 90.5 \text{ Hz}$ (0.5% error compared to 90Hz)

*Temperature & Relative Humidity: 97 Hz + 1.0 Hz/°C *(-3.6 °C) + -0.5 Hz/% *(13 %) = 86.9 Hz (3.4% error compared to 90Hz)*

Based on these results, it can be concluded that it is best to only correct the measured natural frequencies for changes in relative humidity. This is a logical conclusion when the material properties of masonry are once again considered. Masonry is a porous material that is more susceptible to changes in moisture than to temperature.

Using the relative humidity modification coefficient (C_{RH}), a natural frequency normalized to 0% relative humidity, can then be obtained by equation (2):

$$f_{norm} = C_{RH} * RH + f_{meas} \tag{2}$$

Where, f_{norm} is the normalized first mode or fundamental natural frequency of the structure in Hertz, C_{RH} is the relative humidity modification coefficient, RH is the measured ambient relative humidity at the time of testing and f_{meas} is the measured natural frequency obtained from the experimental modal analysis.

Using equation (2), an average normalized first mode natural frequency of 59.0 Hz and a corresponding standard deviation of 1.2 are obtained. The normalized frequencies and their percent difference from the average are shown in Table 3 along with the measured frequencies and their corresponding average and standard deviation. The normalized frequencies show a vast improvement in limiting the deviation from the average, which in turn demonstrates that the measured frequencies can be considered to be valid data and that the difference is not due to some testing error.

Table 3. Normalize First Mode Frequencies

Test Number	Measured Freq. (Hz)	% Diff. from Average	Normalized Freq. (Hz)	% Diff. from Average
1	50.6	-8.31	58.6	0.71
2	51.5	-10.30	58.5	0.83
3	40.7	12.74	59.7	-1.27
4	43.5	6.75	57.5	2.46
5	47.1	-0.88	60.6	-2.73
Average	46.7		59.0	
Std. Dev.	4.6		1.2	

Calculation of a normalized frequency is useful for comparing the measured frequencies, obtained from tests performed under varying ambient environmental conditions. Comparison of normalized frequencies can be used in structural health monitoring to determine if a structure has actually lost strength, or if a decrease in the experimentally obtained natural frequency is due to an increase in the humidity. It should be noted, however, that the normalized frequency is not meant to be taken as the ‘true’ frequency.

As previously demonstrated, the measured frequency is a function of the relative humidity and therefore, at every relative humidity value, a different frequency would be gathered. A good approximation for a structures' 'true' natural frequency then, would be one calculated using an average relative humidity for the location of the structure. This average can be obtained from various weather services that provide annual relative humidity averages. If large quantities of data are taken throughout the year, the average of measured relative humidity taken at the time of testing could also be used. The equation to determine the average frequency can be obtained by slightly modifying Equation 2:

$$f_{aver} = C_{RH}*(RH_{ave} - Rh_{meas}) + f_{norm} \quad (3)$$

where f_{aver} is the average natural frequency, C_{RH} is the relative humidity modification coefficient, RH_{ave} is the average relative humidity, Rh_{meas} is the measured relative humidity and f_{norm} is the normalized frequency.

For this study, application of Equation 3 using the average relative humidity of 25% (shown in Table 1) yields an average frequency of 46.7 Hz. Inspection of this value reveals that it is approximately equal to the average of the measured natural frequencies (Table 1). This is expected due to the range of humidity at which the measured frequencies were obtained.

Conclusions and Future Work

Large variances in relative humidity affect the obtained natural frequencies of structures obtained from experimental modal analysis and should be taken into account.

It is determined that as humidity increases, the natural frequency decreases, most likely due to an increase in mass. The obtained frequencies can be normalized and averaged using the equations developed in this study. Temperature is also found to have a small effect on measured natural frequencies, but does not have as large an impact as relative humidity and can therefore be neglected.

Equation 2 of this study is useful in determining the validity of set of data obtained under varying environmental ambient conditions. Equation 3 is useful for modifying a single natural frequency obtained through EMA and should be used for applications such as finite element model updating.

The limitations of this study are evident in that only five sets of data were utilized to develop the relationships and that the relationships may only be valid for only one type of masonry structure. In order to verify these results, they need to be reproduced on other geometrical masonry configurations. For this purpose, a follow-up study is ongoing, where a second model dome is being constructed and the study will be performed again to insure replication of the data. Other geometries including a masonry wall and a single arch will also be studied. As more data is acquired, the modification factors can be evaluated and modified.

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Comparative studies of IMRF and SMRF.

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Abstract

All structural systems are not treated equal when response to earthquake-induced forces is of concern. Aspects of structural configuration, symmetry, mass distribution, and vertical regularity must be considered. The importance of strength, stiffness, and ductility in relation to acceptable response must also be appreciated. While considering the lateral force resisting systems we come up with so many options to have structural systems like Bearing wall systems, Moment Resisting frames, Lateral Bracing systems, designing the moment resisting concrete frame structures we have option to use IMRF, OMRF or SMRF.

The basic step in conceptual design is to find the best suitable framing system and than lateral load resisting mechanism, while designing structures in the field mostly engineers face problem about the decision of Response Modification Factor R which is a measure of ductility and over strength of the structures. It is used to find the base shear which is distributed on different stories. SMRF and IMRF being emphasized in the research and a detailed computer simulation of the different RCC structures in zone 2 B with different R values i.e., 5.5 and 8.5 given in UBC-1997 are used. Total 04 Structures with different heights of stories, Plans and No. of stories are modeled in software which uses the advanced finite element method to analyze the structure. The conclusions are drawn from the research for the approximation of the most suitable R values and to check the reliability of the values given in UBC.

Keywords: Beams, IMRF, SMRF, Response modification factor, Computer simulation.

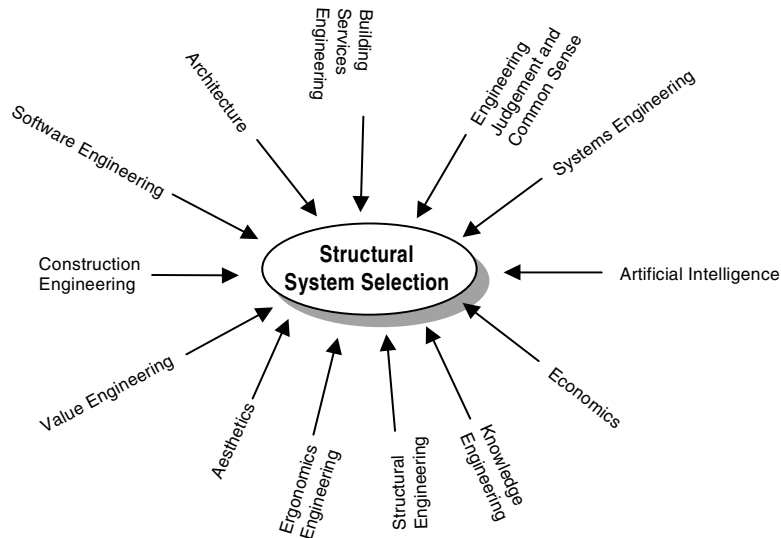
Introduction

In Pakistan no area is completely safe from earthquakes. However some areas are more vulnerable than others. The western and Northern Mountains are more susceptible to seismic activity than the Indus plain. Two high intensity seismic zones are located in the mountains north; The Karakoram region which is associated with Karakoram fault and second area located in the South Abbotabad. The earthquake of October 8, 2005 having magnitude of 7.6 caused widespread damage in Azad Kashmir and adjoining area of NWFP, Pakistan. This earthquake was felt for several minutes in Pakistan, Northern India and Afghanistan. Heaviest damage was recorded in the town of Balakot, Garhi Habibullah, Batal and Batagram in NWFP and Muzaffarabad, Bagh and Rawalakot in Azad Kashmir where entire population was affected. It necessitates the research on earthquake resistant design which will help to mitigate the damage done by them. Lateral-force-resisting elements must be provided in every structure to resist wind and seismic loads. The three principal types of lateral load resisting mechanisms are;

1. Introduction of Shear Walls
2. Braced Frames
3. Moment Resisting Frames

Several factors affect the choice of structural systems which can be summarized in the following flow chart,

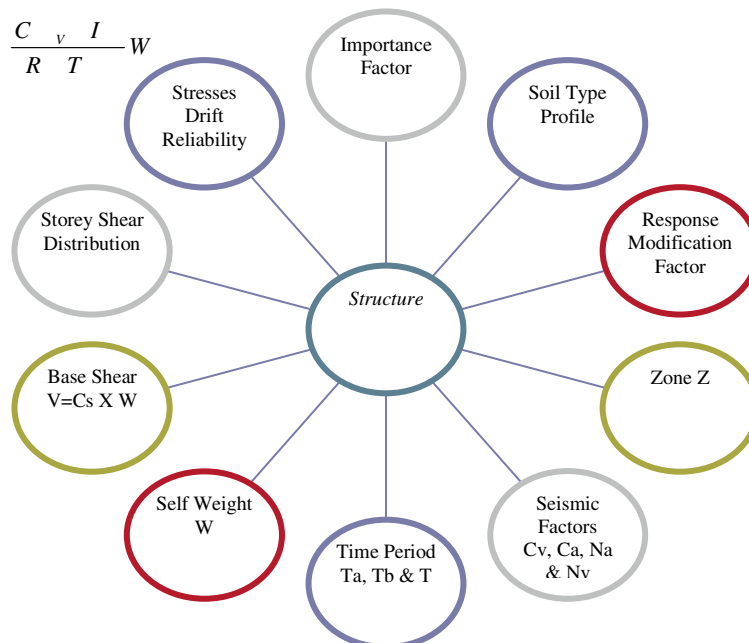
Fig. 1. Parameters for selection of structural system



Special moment resisting frames and Intermediate moment resisting frames will be discussed in this paper; difference is in ductility and over strength. Different parameters governing the stability of the structure are as under,

Fig. 2. Parameters used in seismic analysis

$$V = \frac{C_v I}{R T} W$$



Comparison of drifts, base shear, natural time periods, stresses, flexural steel required in individual elements and strength oriented comparison is carried out.

Objective

The objective of this paper is to find the structural behavior of special moment resisting frames and intermediate moment resisting frame, best structural system and effects of response modification factor on structural response.

Literature Review

Table 1. Detailing Comparison for beams in accordance with ACI-318-05 Manual

Sr. #	Item description	IMRF	SMRF
1	Class of concrete	Class B (min)	Class B (min)
2	Beam Size	8" Min. width	10" Min. width with width to depth ratio of 0.3
3	Differentiating stirrups spacing factor	Effective depth / 2	Effective depth / 4
4	Stirrups spacing in lap	Normal	Not more than 4"
5	Lap location	At column / support	Min. twice the depth from column / support
6	Lap length for top bars	48 diameter of larger longitudinal bar	55 diameter of larger longitudinal bar
7	Positive Moment Strength at joint face	1/3 of Negative Moment Strength at the joint face	1/2 of Negative Moment Strength at the joint face
8	Min. Beam Reinforcement along the length	1/5 of Max. Moment Strength provided at the joint face	1/4 of Max. Moment Strength provided at the joint face

Table 2. Detailing Comparison for columns in accordance with ACI-318-05 Manual

Sr. #	Item description	IMRF	SMRF
1	Class of concrete	Class B (min)	Class B (min)
2	Column Size	8" Min. x-sectional dimension	12" Min. x-sectional dimension with min x-section to perpendicular dimension ratio of 0.4
3	Column Ties Spacing (smaller of three)	8 times the dia. of smaller main bar, one-half of x-sec. dim., 24 times the dia. of ties	6 times the dia. of smaller main bar, one-quarter of x-sec dim., 4"
4	Additional Column Ties at Lap Splices	No	Yes
5	Lap Location	Above beam-column joint	Within middle third of clear column height
6	Lap Length	48 diameter of larger longitudinal bar	55 diameter of larger longitudinal bar
7	Additional Ties at Beam Column Joints	No	Yes
8	Max. Reinforcement Ratio for Columns	4%	3%
9	Additional Column Ties extended into the footings	Nil	Up to 12"

Computer Simulation

Four computer models with different geometry and structural systems are modeled in sophisticated finite element based computer program (STAAD PRO 2005) and are seismically analyzed using Equivalent lateral force analysis procedure laid down in UBC-1997. Using the seismic parameters as laid down in Pakistan Building Code and Geotechnical data as per report for the site in consideration,

Type of structure	RCC FRAME
Roof type	Normal RCC Slab
Code used	ACI 318-02 (For Concrete Design) UBC 1997 (For Lateral Loads)
Soft ware	STAAD.Pro ver 2005

Earth quake zone Z	3 and 2b
Importance Factor I	1.0
Rwx, Rwz	8.5 and 5.5
28-Days Conc. Compressive Strength	3000 psi
Reinforcement strength	Grade 60 Deformed bars
Average Soil Bearing capacity	4.3 KSF
12. Soil type profile	D

Fig 4. Model No.1 (5 Storey regular structure with storey height of 11')

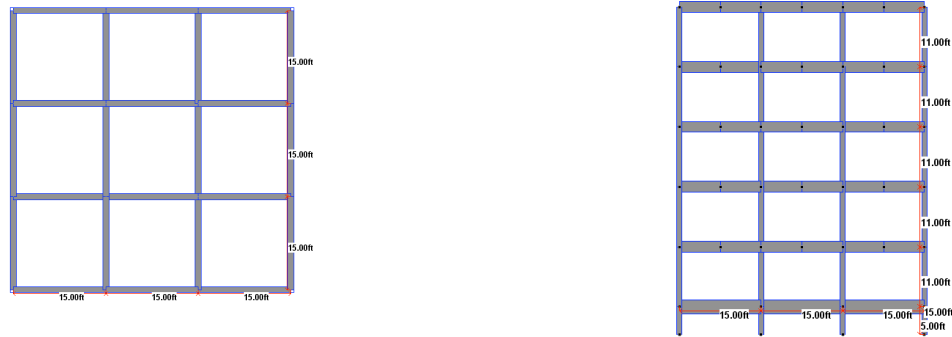
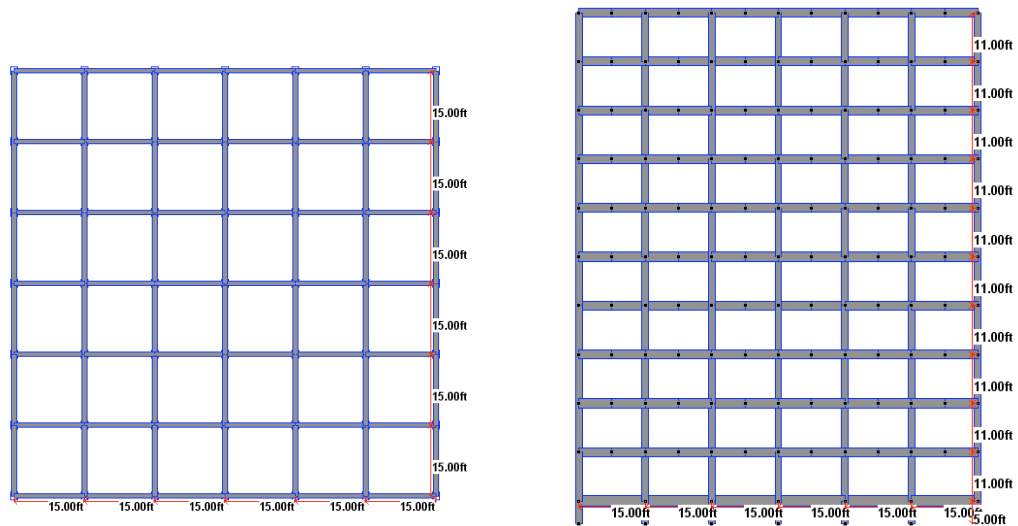


Fig 5. Model No.2 (10 Storey regular structure with storey height of 11')



Analysis Results

Table 3. Comparison of Time periods and Base shear

BUILDING	TIME PERIOD		SMRF		IMRF	
			X-direction	Z-direction	X-direction	Z-direction
5 STOREY 3X3@15' BAY BUILDING	Ta	Time period by using method A	0.647		0.647	
	Tb	Time period by using method B	0.55		0.55	
	T	Time period of building	0.55		0.55	
	V	Base shear	185.71	185.71	287.01	287.01
10 STOREY 6X6@15' BAY BUILDING	Ta	Time period by using method A	1.05		1.05	
	Tb	Time period by using method B	1.38		1.38	
	T	Time period of building	1.38		1.38	
	V	Base shear	750.23	750.23	485.46	485.46

Fig 6. Comparison of storey drifts in 5 storey building

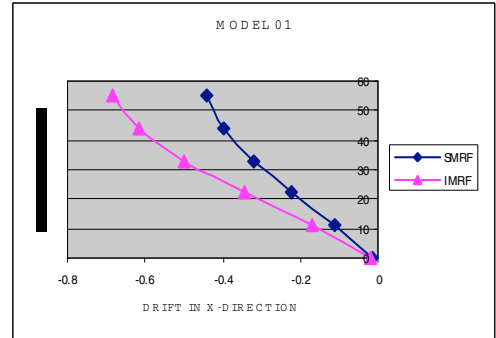
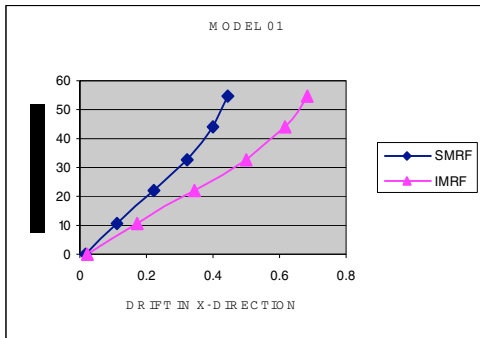


Fig 7. Comparison of storey drifts in 10 storey building

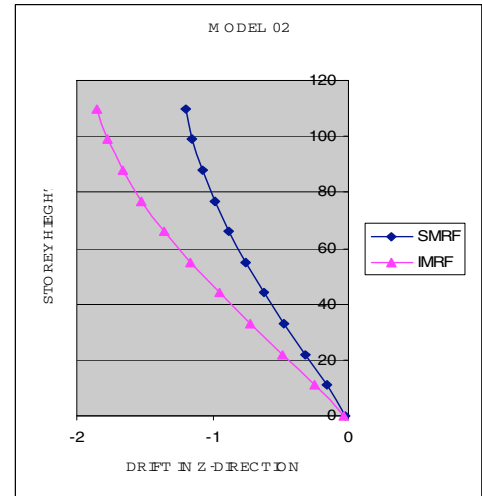
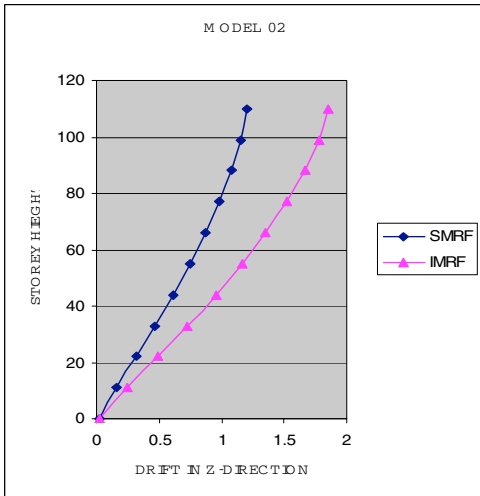


Table 4. Comparison of Footing Results

JOINT #	SMRF				IMRF				RATIO	AVERAGE
	FY	MX	MZ	AREA	FY	MX	MZ	AREA		
1	159.44	3.32	-3.32	49	187.49	3.68	-3.68	64	0.77	0.81
2	249.78	0.04	-3.76	64	272.08	0.1	-4.28	81	0.79	
3	249.78	-0.04	-3.76	64	272.08	-0.1	-4.28	81	0.79	
4	159.44	-3.32	-3.32	64	187.49	-3.68	-3.68	81	0.79	
5	249.78	3.76	-0.04	64	272.08	4.28	-0.1	81	0.79	
6	368.08	0.16	-0.16	90.25	383.19	0.48	-0.48	100	0.90	
7	368.08	-0.16	-0.16	90.25	383.19	-0.48	-0.48	100	0.90	
8	249.78	-3.76	-0.04	64	272.08	-4.28	-0.1	81	0.79	
9	249.78	3.76	0.04	64	272.08	4.28	0.1	81	0.79	
10	368.08	0.16	0.16	90.25	383.19	0.48	0.48	100	0.90	
11	368.08	-0.16	0.16	90.25	383.19	-0.48	0.48	100	0.90	
12	249.78	-3.76	0.04	64	272.08	-4.28	0.1	81	0.79	
13	159.44	3.32	3.32	49	187.49	3.68	3.68	64	0.77	
14	249.78	0.04	3.76	64	272.08	0.1	4.28	81	0.79	
15	249.78	-0.04	3.76	64	272.08	-0.1	4.28	81	0.79	
16	159.44	-3.32	3.32	49	187.49	-3.68	3.68	64	0.77	

Table 5. Comparison of Column Results

COLUMN #	SMRF		IMRF		VARIATION FACTOR	AVERAGE
	AREA	%GE STEEL	AREA	%GE STEEL		
1	144	3.4	144	2.3	1.45	1.49
2	144	4.6	144	3.1	1.49	
3	144	4.6	144	3.1	1.49	
4	144	3.4	144	2.3	1.45	
5	144	4.6	144	3.1	1.49	
6	144	6.3	144	4.1	1.53	
7	144	6.3	144	4.1	1.53	
8	144	4.6	144	3.1	1.49	
9	144	4.6	144	3.1	1.49	
10	144	6.3	144	4.1	1.53	
11	144	6.3	144	4.1	1.53	
12	144	4.6	144	3.1	1.49	
13	144	3.4	144	2.3	1.45	

14	144	4.6	144	3.1	1.49
15	144	4.6	144	3.1	1.49
16	144	3.4	144	2.3	1.45

Discussions & Conclusions

The following conclusions are drawn from the computer simulation program carried out in this investigation:

- Base Shear is reduced by 35% in SMRF as compared to IMRF, it is due to the increased R value which is the measure of ductility and overstrength (by virtue of ductile detailing).
- Storey drifts are reduced by 40% when we switch from Intermediate Moment Resisting Frames to Special Moment Resisting Frames.
- By using Special Moment Resisting frames the design of footings is economized by 20% as compared to Intermediate Moment Resisting Frames. It is due to the fact that base shear is reduced due to increased R value in SMRF.
- By using Special Moment Resisting frames the longitudinal reinforcement required in columns is increased by 40 % as compared to Intermediate Moment Resisting Frames. It is due to the reduced base shear. However secondary reinforcement is increased in columns due to ductile detailing.
- By using Special Moment Resisting frame the longitudinal reinforcement required in beams is decreased by 15 % as compared to Intermediate Moment Resisting Frames. However the secondary reinforcement required in beams is increased due to ductile detailing.
- It is more preferable to use SMRF in zone 2b rather than IMRF. Because it can take more seismic shocks than IMRF due to ductile detailing and increased over strength.

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COMPARISONS OF MEASURED MODAL PROPERTIES AND WALKING ACCELERATIONS WITH ANALYTICAL PREDICTIONS FOR A SLENDER MONUMENTAL STAIR

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Abstract

Over the last few years, slender, clear-span monumental stairs have become increasingly popular in hotels, hospitals, and a variety of other structures. These stairs are vulnerable to annoying vibrations due to walking. Finite element modeling techniques may be used to predict the vibration response of a stair, but experimental verification of these procedures is not available.

An experimental and analytical study was performed toward the goal of better understanding vibration characteristics of monumental stairs and to judge the accuracy of finite element modeling predictions. A monumental stair containing heavy glass treads and guardrails, supported by slender hollow structural section (HSS) stringers was used in the study.

Experimental modal analysis techniques were used to estimate the stair's modal properties. Stair ascents and descents were also performed to determine the acceleration response. Finite element modeling techniques were used to predict the natural frequencies, acceleration FRF peak magnitudes, and acceleration response to walking. The experimental results are compared to the finite element modeling predictions to determine whether or not the finite element modeling techniques provided reasonably accurate predictions.

Introduction

Slender monumental stairs are major architectural features in many hotels, condominiums, educational buildings, hospitals, and other public areas. Architectural requirements for these are usually very aggressive with long clear spans and slender structures being the norm. Slender stairs often have fairly heavy treads and guardrails, which combined with slender stringers, can result in low natural frequencies. Further, humans commonly ascend and descend stairs at frequencies much higher than those observed for walking on a flat surface, so excitation by a low harmonic of the walking force is sometimes possible. Descriptions of footstep force Fourier amplitudes can be found in Murray et al. (1997) Section 1.5 and Smith et al. (2007), Section 2.3.1. Footstep forces while ascending and descending stairs are

much larger than forces applied while walking on a flat surface. Designers of such stairs have little guidance to rely upon because the major design guides (Murray et al. 1997, Smith et al. 2007) do not directly address stairs and no formal, accepted tolerance criterion exists for stairs.

This paper contains the results of an experimental and analytical study of a slender monumental stair. Modal and walking tests were performed on the stair to obtain estimates of the modal properties and response to walking. A practical and relatively simple finite element modeling technique for predicting accelerations due to ascents and descents is described and demonstrated.

Review of Previous Stair Vibration Research

Bishop et al. (1995) identified three major difficulties associated with stair vibration analysis: poorly defined footstep forces, unclear effect of multi-person loading, and lack of tolerance criteria specific to staircases. They stated that ascent and descent footstep harmonic forces can be much larger than those measured for walking on a flat surface. They also noted that it is relatively common to “trot” down a stair at frequencies as high as 4 Hz and that 4.5 Hz is possible. (Walking on a flat surface usually occurs at step frequencies not exceeding 2.2 Hz (Murray et al. 1997, Smith et al. 2007).) Bishop et al. (1995) also stated that groups of walkers might cause much more severe vibrations on very busy stairs.

Bishop et al. (1995) also presented vibration response measurements for four steel stairs. They reported measured natural frequencies of 5 Hz, 27 Hz, 13 Hz, and 9 Hz (and 14 Hz). Corresponding peak accelerations measured for single person rapid descents at approximately 4 Hz were 4.0 %g, 4.5 %g, 1.5 %g, and 1.4 %g, for the four stairs, respectively. All of the stairs were considered acceptable by their users, although there was a comment regarding the liveliness of Stair 1.

The researchers compared measured single walker accelerations to single degree of freedom analytical predictions for the fourth stair. Their calculations predicted increased acceleration response for walking at natural frequency subharmonics, but this was not observed during the tests.

In his doctoral thesis, Kerr (1998) studied two of the research needs identified by Bishop et al. (1995): footstep forces on stairs and group effects. He presented 600 individual footstep force measurements from 25 subjects ascending and descending an instrumented stair. All subjects felt that ascents were comfortable at 2 Hz and 3.3 Hz, and that other frequencies were uncomfortable. Subjects typically reported that any descent step frequency below 4 Hz was comfortable. Results were reported as the first four Fourier amplitudes of the walking force (as a fraction of bodyweight—referred to hereafter as Dynamic Load Factors or DLF) plotted versus step frequency, with tremendous scatter of the data noted. Plots of the first harmonic for ascending and descending indicate DLF that increase with step frequency and sometimes exceed 1.0. He also reported average DLF and ± 2 standard deviation ranges for the second through fourth harmonic of the walking force. Averages and ranges are shown for the entire dataset in Table 1. For comparison, design DLF for walking on a flat surface are also shown (Willford et al. 2006).

Table 1. Average Stair Ascent and Descent DLF Reported by Kerr (1998) and Horizontal Surface Walking DLF Reported by Willford et al. (2006)

Description	DLF	± 2 Standard Deviation Range
Ascending, 2 nd Harmonic	Varies between 0.13 (4 Hz) and 0.07 (7 Hz) depending on step frequency.	0.00 to 0.22
Ascending, 3 rd Harmonic	0.06	0.00 and 0.11
Ascending, 4 th Harmonic	0.03	0.00 and 0.07
Descending, 2 nd Harmonic	0.2	0.03 and 0.33
Descending, 3 rd Harmonic	0.09	0.00 and 0.16
Descending, 4 th Harmonic	0.06	0.00 and 0.13
Horizontal, 2 nd Harmonic	Varies between approx. 0.08 and approx. 0.11	-
Horizontal, 3 rd Harmonic	Approx. 0.06	-
Horizontal, 4 th Harmonic	Approx. 0.05	-

Experimental Program

A slender monumental stair was vibration tested to obtain estimates of the natural frequencies, acceleration response (accelerance FRF magnitude), and response to walking. The stair, large fountain below, large chandelier above, and moving water wall, combine to form a major architectural feature in the lobby and restaurant area of a high-rise hotel and condominium. The stair consists of a main span (left side of Figure 1) from the lobby to an intermediate landing and a shorter span from the intermediate landing to the second floor. The main span is of primary interest in this study.

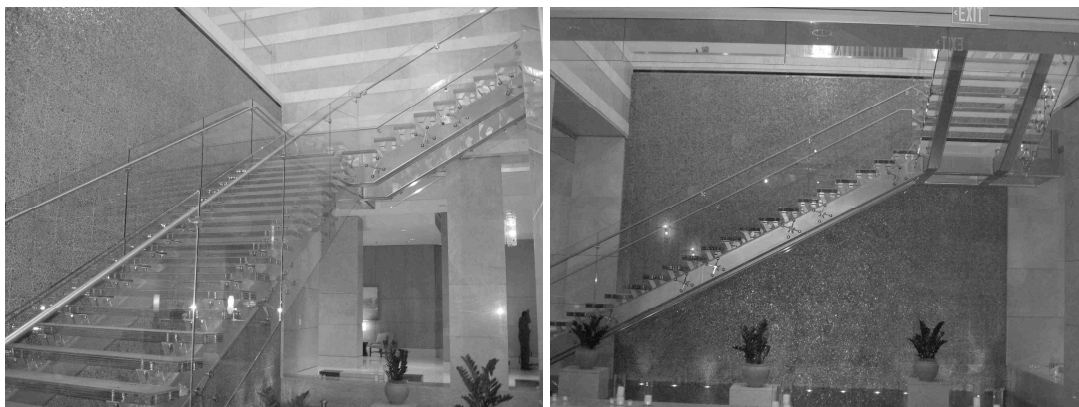


Fig. 1. Stair Photographs

The stair main span is very elegant and slender, with two 305 mm by 203 mm (12 in. by 8 in.) HSS stringers spanning approximately 9.49 m (31 ft) between supports. Each stringer bears on a steel angle seat at the low end and is suspended from a small HSS hanger at the upper end. The translucent, laminated glass treads weigh approximately 1.5 kPa (31 psf) and are supported on thin rubber pads and through-

bolted to bent plates. The clear glass guardrails weigh approximately 0.48 kPa (10 psf) and are connected to the stringers using spider fittings. The total stair weight is approximately 93 kN (21 kips).

Modal tests were conducted to estimate modal properties of the structure. An impulse hammer was used to provide excitation to the underside of a stringer near midspan to avoid standing on the specimen during the tests. Accelerations were measured using accelerometers connected to the structure near the driving point and at other locations of interest.

The estimated driving point accelerance frequency response function magnitude is shown in Figure 2(a). The fundamental frequency is 7.3 Hz and the accelerance peak magnitude is 0.0679 %g/N (0.302 %g/lbf). Two other vibration modes are indicated at 7.6 Hz and 7.8 Hz, but the 7.3 Hz mode provides much higher acceleration response. The damping was estimated to be 1.1% of critical. It is interesting to note the vibration mode at approximately 4.6 Hz. This mode is the first flexural mode for guardrail lateral vibration, indicating that horizontal movement of the guardrail is likely to occur due to vertically applied forces.

Hammer strikes were also applied laterally at midspan near the top of a stringer to determine the frequency of lateral vibration. The estimated accelerance frequency response function is shown in Figure 2(b). The natural frequency, accelerance peak magnitude, and damping ratio are estimated to be 10.0 Hz, 0.0486 %g/N (0.216 %g/lbf), and 1.2% of critical, respectively.

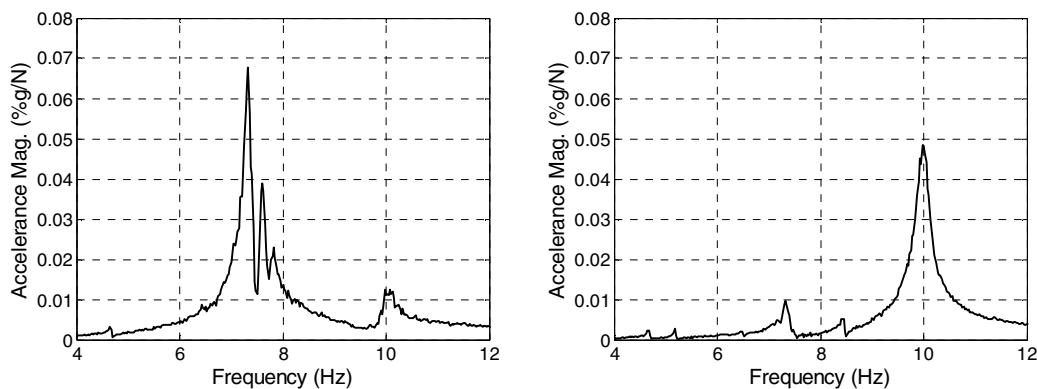


Fig. 2. Estimated Accelerance Magnitude. (a) Vertical; (b) Horizontal

Walking tests were performed to estimate the stair's vertical acceleration response to ascents and descents. Lateral vibration was not significant because the fundamental lateral vibration mode was so high (10 Hz). Top of guardrail lateral vibrations were also measured because guardrail movement was visible during some preliminary tests. Four walkers (healthy males weighing approximately 84 kg (185 lbf), 91 kg (200 lbf), 105 kg (230 lbf), and 105 kg (230 lbf)) ascended and descended the stair individually at various speeds ranging from normal walks to very fast runs. Walkers landed on each step during all tests rather than skipping steps. Three observers also provided subjective evaluations of vibration severity.

The stair’s fundamental frequency is 7.3 Hz. Therefore, walking at the second (3.65 Hz or 219 bpm (beats per minute)), third (2.43 Hz or 146 bpm), and fourth (1.83 Hz or 110 bpm) subharmonic are feasible and would likely result in maximum acceleration responses. Therefore, numerous ascents and descents were performed at each frequency, using a metronome to encourage walking at a resonant frequency (natural frequency subharmonic). The maximum peak acceleration responses are shown in Table 2. The waveform, filtered between 1 Hz and 15 Hz, and frequency spectrum for the test with maximum response (descending, 2nd subharmonic) are shown in Figure 3. Resonant build-ups were observed in most tests, and were most obvious in the tests with walking at the natural frequency second subharmonic. The spectra clearly indicate the step frequencies and that the majority of the response at or near the natural frequencies.

Table 2. Maximum Measured Peak Acceleration Responses to Walking

Description	Step Frequency (Hz)	Peak Acceleration at Midspan (%g)	Peak Acceleration at Intermediate Landing (%g)
Ascending, 2nd Subharmonic	3.45	4.1	2.8
Ascending, 3rd Subharmonic	2.45	1.1	0.74
Ascending, 4th Subharmonic	1.83	1.2	0.86
Descending, 2nd Subharmonic	3.63	4.7	3.4
Descending, 3rd Subharmonic	2.48	1.7	1.3
Descending, 4th Subharmonic	1.85	1.6	1.2

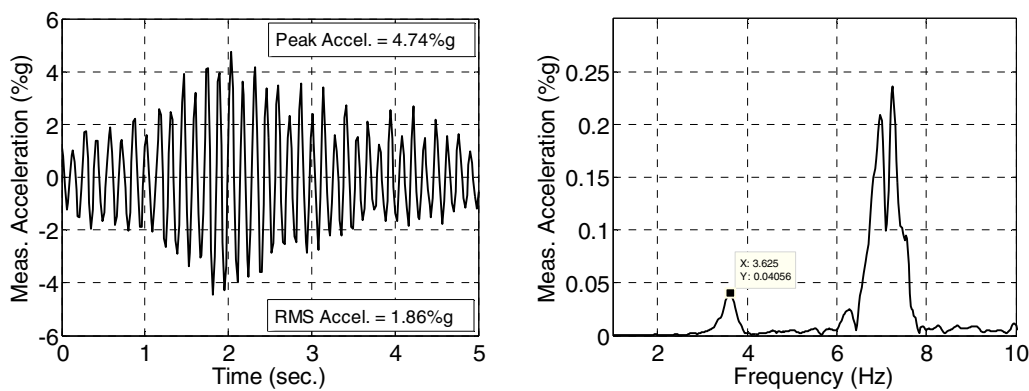


Fig. 3. Acceleration Due to Walking. (a) Waveform; (b) Spectrum

Walkers reported that rapid trips up and down the stairs were comfortable at or below approximately 240 bpm (4 Hz). Very rapid, but controlled descents were found to be possible up to approximately 270 bpm (4.5 Hz), which is consistent with previous results (Kerr 1998, Kerr and Bishop 2001). As expected, ascents at 147 bpm (2.45 Hz)—in the “mixture” zone described by Kerr (1998)—were uncomfortable.

A group of three individuals stood on the intermediate landing and offered their opinions of the vibration level and likelihood that complaints will occur. They stated that walking at normal speeds resulted in only mildly perceptible vibrations and they

considered it unlikely that occupants would complain. Peak accelerations during these tests were between 1.0 and 1.5 %g. They also experienced 3.5-4.0 %g peak accelerations due to fast trots and running on the stair. They described these as strongly perceptible and offered the opinion that very sensitive occupants would likely be disturbed.

One observer commented that one of the guardrails moved noticeably during fast trots and running on the stair, perhaps indicating that this increased his perception of the vibration level.

Analytical Predictions and Comparison to Measurements

In this section, the methods and results from finite element modeling predictions are presented. The modeling techniques were: construct a design model of the stair, use steady-state analysis to predict the acceleration peak magnitude, apply the footstep forcing functions reported by Kerr (1998) to predict peak steady-state acceleration response, and reduce the prediction to account for imperfect resonant build-up. A commercial structural analysis program, SAP2000, Version 10 (CSI 2006) was used for the analyses.

The finite element model was constructed using only information that would be available to a designer, but is comprehensive and detailed. Shell elements were used to model the stair treads and guardrails. Guardrails were included to accurately model their stiffness contributions and to allow prediction of their movement during ascents and descents. Treads and guardrails were connected to the stringers using frame elements with section properties corresponding to the bent plate connections and spider fittings, respectively. Grab-rails and stringers were modeled using frame elements.

Standard eigenvalue analysis was used to predict the stair's natural frequencies and mode shapes. The first mode shape (Figure 4(a)) was predicted to have a frequency of 3.9 Hz and represents the first bending mode for the guardrails along the main span, with little displacement elsewhere. The predicted frequency is 15% lower than the measured frequency, 4.6 Hz. The second mode shape, Figure 4(b), is the first stringer bending mode with some guardrail displacement and is predicted to have a natural frequency of 6.1 Hz which is 16% lower than the measured frequency, 7.3 Hz. Mode 8 (not shown) is the main lateral vibration mode and is predicted to have a natural frequency of 9.5 Hz which is 5% lower than the measured frequency, 10 Hz.

As is typical of finite element analysis of floors or stairs, numerous modes were predicted and it was difficult or impossible to determine which would result in the most severe predicted acceleration responses simply by viewing the mode shapes. In this model, the inclusion of the guardrails further obscured the relevant overall bending modes. Therefore, steady-state analysis was used to predict the vertical acceleration response at midspan due to sinusoidal load applied at midspan, as a function of frequency. Peaks in the resulting frequency response function indicated natural frequencies. Viscous modal damping was assumed to be 0.01 due to the lack of non-structural attachments, but hysteretic damping was specified for the steady-state analyses. Constant hysteretic damping with a stiffness proportional coefficient

of 0.02 (double the assumed viscous damping) was used (Chopra 1995). The predicted accelerance magnitude (response and force both vertical at the center of the tread nearest to the stringer midspan) is shown in Figure 5(a). The plot indicates that the second mode provides the largest acceleration response by a very wide margin. The accelerance peak magnitude is predicted to be 0.0751 %g/N (0.334 %g/lbf).

The accelerance magnitude was also predicted for force applied to a stringer to allow comparison with the measured accelerance magnitude, as shown in Figure 5(b). Vertical acceleration was measured and predicted at the center of the tread closest to the stringer midspan. The predicted accelerance peak magnitude is 0.0701 %g/N (0.312 %g/lbf) which is 3% higher than the measured value, 0.0679 %g/N (0.302 %g/lbf). From observing the predicted and measured accelerance magnitude peaks (and from the half-power damping calculation discussed previously), the assumption of 1% viscous damping appears to be reasonable.

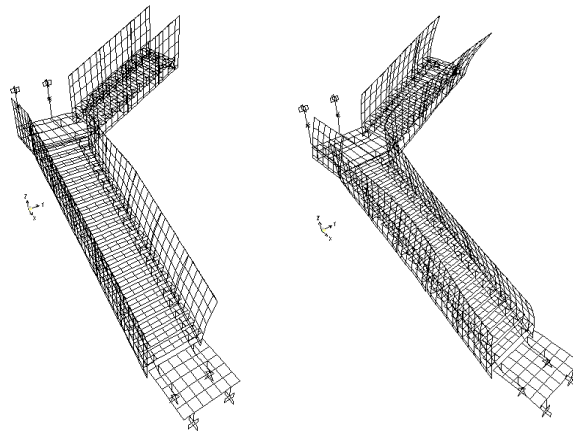


Fig. 4. Predicted Mode Shapes. (a) Mode 1, 3.9 Hz; (b) Mode 2, 6.1 Hz

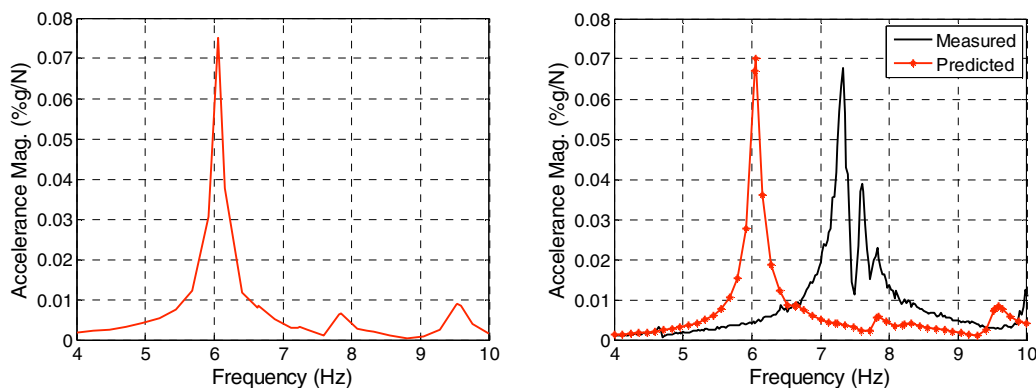


Fig. 5. Accelerance Magnitudes. (a) Predicted, Vertical Acceleration and Load; (b) Measured and Predicted, Vertical Acceleration, Load Perpendicular to Stringer

The predicted accelerance peak magnitude, 0.0751 %g/N (0.334 %g/lbf), was then used to predict the steady-state acceleration response due to walking using harmonic forces reported by Kerr (1998). The method is similar to the one proposed by Barrett

(2006) for floor vibration evaluation. Because the second mode (6.1 Hz) is highly dominant, its response was used exclusively. No other modes contribute significantly, so there was no need to combine modal results. Because stair descents may occur at step frequencies between approximately 120 bpm (2 Hz) and 270 bpm (4.5 Hz), walking at the second or third subharmonic (3.05 Hz and 2.03 Hz) should provide the maximum response. From Table 1, the average second and third harmonic DLF of ascents are 0.09 (interpolated) and 0.06. The second and third harmonics DLF of descents are 0.2 and 0.09. The walker’s bodyweight was 84 kg (185 lbf), so this weight was multiplied by the DLF to determine harmonic loading shown in Table 3. (Note that the actual bodyweight and average DLF are used because the goal of this calculation is comparison with measurements. General design loads are proposed in a later section.) The predicted acceleration peak magnitude for vertical force and acceleration at the center of the stair, 0.0751 %g/N (0.334 %g/lbf), is multiplied by the harmonic forces to predict the steady-state acceleration to walking. These are also shown in Table 3.

Table 3. Harmonic Loads and Predictions of Steady-State Accelerations

Description	DLF	Force Amplitude N (lbf)	Steady-State Peak Acceleration (%g)
Ascending, 2nd Harmonic	0.09	74 (17)	5.6
Ascending, 3rd Harmonic	0.06	49 (11)	3.7
Descending, 2nd Harmonic	0.2	165 (37)	12
Descending, 3rd Harmonic	0.09	74 (17)	5.6

The predicted steady state accelerations must be scaled down to provide realistic predictions of accelerations due to walking. Walkers do not ascend or descend stairs at precisely the natural frequency subharmonic and they do not walk for infinite amounts of time, so the resonant build-up will not be complete. Two separate resonant build-up factors are given in British design guides (The Concrete Centre 2006, Smith et al. 2007), but these apply only to walking on floors. The authors are not aware of a resonant build-up factor that has been developed for use on a stair.

A simple adjustment factor is tentatively proposed to account for imperfect resonant build-up and the fact that not all steps will occur at midspan by averaging the ratios of measured peak accelerations and predicted steady-state peak accelerations with results shown in Table 4. The average ratio is 0.43; the average ratio excluding the outlying measurement (ascending, 2nd harmonic) is 0.33. Considering very large scatter in footstep force data (Kerr 1998), an adjustment factor of 0.35 seems sensible. Descending will always be the critical design load case, so more consideration is given to those ratios. While every effort was made in this study to walk precisely at a natural frequency subharmonic, and numerous tests were performed, additional research is required to determine the reliability of the proposed adjustment factor. Longer stairs might undergo more complete resonant build-ups, for example: however, there is some doubt about a walker’s ability to match the resonant frequency for the dozens of steps required to traverse a very long stair. Very long stairs also usually have intermediate landings which will break up any rhythm that a walker may have developed.

Table 5 shows the measured and predicted peak accelerations, using the proposed 0.35 adjustment factor, due to walking. The predictions are reasonably accurate for all tests except for the case of ascending, second harmonic.

Table 4. Measured Accelerations and Predicted Steady-State Accelerations

Description	Measured Acceleration (%g)	Predicted Steady-State Peak Acceleration (%g)	Ratio
Ascending, 2nd Harmonic	4.1	5.6	0.73
Ascending, 3rd Harmonic	1.1	3.7	0.30
Descending, 2nd Harmonic	4.7	12	0.39
Descending, 3rd Harmonic	1.7	5.6	0.30
		Average =	0.43
		Average, excluding outlier =	0.33

Table 5. Measured and Predicted Accelerations Due to Walking

Description	Measured Peak Acceleration (%g)	Predicted Peak Acceleration (%g)	% Difference
Ascending, 2nd Harmonic	4.1	1.9	-54
Ascending, 3rd Harmonic	1.1	1.3	+18
Descending, 2nd Harmonic	4.7	4.2	-11
Descending, 3rd Harmonic	1.7	1.9	+12

Summary

A slender monumental stair was vibration tested to estimate its natural frequencies, steady-state acceleration response, damping ratio, and acceleration due to ascents and descents. Vibrations due to normal ascents and descents were described as mildly perceptible and the accelerations were less severe than the limit proposed by Bishop et al. (1995). Rapid ascents and descents resulted in strongly perceptible vibrations that exceeded limits proposed here.

Analytical predictions of natural frequencies, steady-state acceleration response, and acceleration due to ascents and descents were made using a commercially available finite element analysis program. The model was created using only information that would be available to a designer (including the use of an assumed damping ratio), but was detailed, and not adjusted to force its modal properties to match measured values. Natural frequency was reasonably predicted and steady-state acceleration response (accelerance peak magnitude) was very closely predicted. Footstep loading from Kerr (1998) was used to predict the steady-state acceleration due to walking. An adjustment factor was derived because no established method was found for considering imperfect resonant build-up. The resulting walking acceleration predictions reasonably matched the measured values.

Acknowledgements

The authors would like to thank Bernard Shumate and James McRoberts of Armada Hoffer Construction Company for approving vibration tests and Jose Rodriguez of Armada Hoffer Construction Company for facilitating and coordinating our access to the monumental stair. We also thank Val Courtney of Valstone Engineering and Tate Ornamental for providing drawings and other information necessary for completion of this study.

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SIMULATION OF REINFORCED CONCRETE SLAB BEHAVIOR UNDER IMPACT LOADING

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ABSTRACT

Reinforced concrete slabs are among the most common structural elements. In spite of the large number of slabs designed and built, the effect of their details on their behavior under impact loads are not always appreciated or properly taken into account. Reinforced concrete slabs exhibit interesting behavior under severe dynamic loads associated with impact and explosions. The assessment of the behavior of structural concrete slabs subjected to impact loading requires tools that can be used to analyze and design structural element that will improve the state of the art of protective design. The behavior of concrete slabs under impact loading was studied numerically. This numerical study was aimed at understanding the dynamic behavior of structural concrete slabs under impact loading before conducting full scale experimental tests. This study investigated the effects of different types of slab reinforcements and the applied impact loads on the dynamic response and behavior of reinforced concrete slabs. Three-dimensional numerical simulations were performed with the computer code ABAQUS/Explicit, addressing the parameters associated with the impact load and concrete slab. The effects of different types of slab reinforcement and the applied impact load on the behavior and dynamic response of concrete slabs were numerically investigated.

Three different types of 90 x 1,524 x 3,353 mm (3-1/2" x 5' x 11') slabs were simulated, as follows: First; using two 152 x 152 mm (6"x 6") meshes of welded steel wires located under 25 mm (1") of concrete cover on both faces of the slab; second, with one 152 x 152 mm (6"x 6") mesh of No. 3 steel bars located at the middle of the slab thickness; and third, with two 152 x 152 mm (6"x 6") meshes of No. 3 steel bars located under 25 mm (1") of concrete cover on both faces of the slab. The drop heights of the impact weight were 152 mm, 305 mm, and 610 mm (6", 12", and 24"), respectively.

1. Introduction

The reinforced concrete slabs chosen for this study were part of an experimental test program, where nine slabs were subjected to impact loads and their performance under these loads was examined, Table 1. The specific physical test setup and experimental results for this study were presented in the International Journal of Impact Engineering. The purpose of the initial numerical simulation was to predict the dynamic response used to prescribe the parameters of a

precision impact test that produces an equivalent dynamic response. Once the drop height has been determined, and the structural component of the impact system considered, the preliminary simulations of the slab system behavior can be performed to determine the levels of strains, accelerations, stresses, deflections, and peak load the slab will experience during the impact event [6].

Table 1. Summary of Slab Details

Slab	Description of Cases
2W-1	12" (305 mm) drop on slab with 2 welded-wire fabric meshes
2W-2	6" (152 mm) drop on slab with 2 welded-wire fabric meshes
2W-3	24" (610 mm) drop on slab with 2 welded-wire fabric meshes
1#3-1	12" (305 mm) drop on slab with 1 mesh of # 3 rebar
1#3-2	6" (152 mm) drop on slab with 1 mesh of # 3 rebar
1#3-3	24" (610 mm) drop on slab with 1 mesh of # 3 rebar
2#3-1	12" (305 mm) drop on slab with 2 meshes of # 3 rebar
2#3-2	6" (152 mm) drop on slab with 2 meshes of # 3 rebar
2#3-3	24" (610 mm) drop on slab with 2 meshes of # 3 rebar

The main objective of generating and analyzing the nine different three dimensional case models was to determine the overall effects of each individual reinforcement variation and impact velocity as they were assigned to the concrete slab model. By determining the effects that each case had on the structural response of the concrete slabs, a better understanding of the entire system's behavior could be gained, in addition to discovering the effects of the localized impact on the structural behavior of reinforced concrete slabs. Nodal displacement and element strain-time history data for the concrete, steel reinforcement, and impact mass of each case were recorded to compare and contrast each case for the concrete and steel meshes. In addition to the time-history plots, figures of the deformed structure for each case are shown to illustrate the composite structural behavior for each case. Comparing the numerical results of the concrete and steel meshes will enhance the effort to advance the state of knowledge in precision impact testing and its potential for future code validation [5].

2. Numerical Procedures

The numerical simulations were performed using the explicit finite element code ABAQUS/Explicit implementing the dynamic constitutive laws of concrete and steel. The selection of the material or constitutive models is essential to accurately simulate the behavior of the structure. ABAQUS/Explicit contains a large library of material models, and the ones used were those that best represented the material's behavior within the constraints of the available material property data. The concrete in the slabs was generated using 1" three-dimensional eight-node "solid" brick elements with one-point integration. The steel reinforcement in the slabs was modeled by discrete two-node beam elements connected to the nodes of

adjacent concrete elements. The modeling of the reinforcement using discrete beam elements is a more accurate method of simulating the function of the steel bars interacting with the concrete, and enables direct calculation of the forces within the beam (i.e. the stresses in the steel).

In this model, a perfect bond between the concrete and steel was assumed. Although this is not the case in reality, especially during large inelastic deformations, the constitutive model of the steel was selected to compensate for bond slip between steel bars and the surrounding concrete.

Elements used in ABAQUS/Explicit are based on a Lagrangian formulation where the element deforms with the material. Therefore, it is important to select an appropriate material model that can closely represent the actual behavior of the physical entity under consideration. An elasto-plastic material property with isotropic strain hardening was selected to simulate the material behavior of all the components of the finite element model except the concrete slabs, which were assigned the modified Drucker-Prager/Cap plasticity model. This model is defined by using the *CAP PLASTICITY option to define the yield surface parameters. The addition of the cap yield surface to the Drucker-Prager model serves two main purposes: it binds the yield surface in hydrostatic compression, thus providing an inelastic hardening mechanism to represent plastic compaction; and it helps to control volume dilatation when the material yields in shear providing softening as a function of the inelastic volume increase created as the material yields on the Drucker-Prager shear failure surface [1].

The yield surface has two principle segments: a pressure-dependent Drucker-Prager shear failure segment and a compression cap segment. The Drucker-Prager failure segment is a perfectly plastic yield surface (no hardening). Plastic flow on this segment produces inelastic volume increase (dilatation) that causes the cap to soften. On the cap surface plastic flow causes the material to compact. The concrete material model is shown in Figure 1 [1].

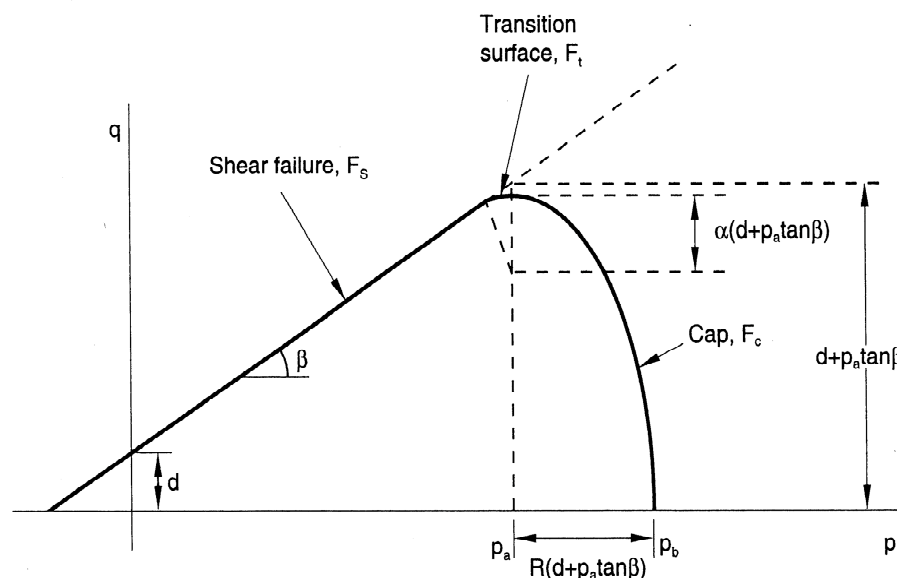


Figure 1. Modified Drucker-Prager/Cap Model

The bilinear stress-strain relation shown in Figure 2 was adopted as the steel properties used as reinforcement for concrete and steel frame. Two stages of behavior in the stress-strain curve have been generally observed. The curve in the first stage is linear up to yield point at a yield stress, f_y , of 61 ksi and corresponding to a slope (or modulus of elasticity, E) of 2.9×10^7 psi. The second stage of curve which depicts the plastic plateau and the strain hardening branch from uniaxial tensile tests is 77 ksi. The steel material model is shown in Figure 2 [2].

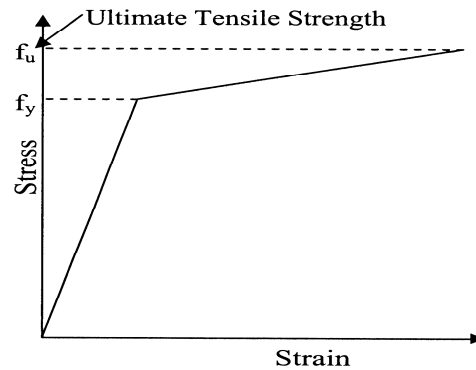


Figure 2. *Elasto-Plastic with Isotropic Strain-Hardening*

The geometrical approach used to simulate the reinforced concrete slabs and the impacting mass behaviors, considers one quadrant of a full size rectangular slab $90 \times 1,524 \times 3,353$ mm ($3\text{-}1/2 \times 5 \times 11$ ') subjected to impact load at the center. All edges of the slab were constrained against displacement and rotation. For the purpose of reducing the number of elements and saving computational time, the model consisted of one quarter (defined by the two symmetry lines in the plane of the slab) of the original slab subjected to the impact load at the free corner region, without affecting the response of the system (Figure 3).

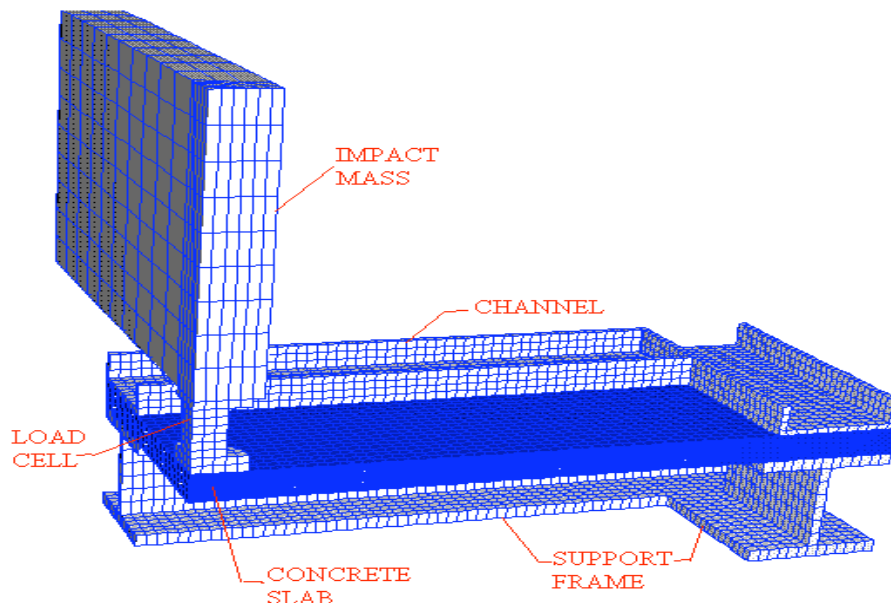


Figure 3. *Eight-node Solid Elements Mesh of the Entire System*

3. Comparison of Displacements

A visual comparison between the experimentally measured and numerically predicted deformations for the slab subjected to impact loading is shown in Figure 4. A major difference between the behavior of the slab under impact and the numerical simulation is illustrated here. These particular simulations had the slab numerically fixed to the support frame and the impact mass velocity continued until the end of the run (30 ms). The large deflection in the numerical simulation was caused by the continuous penetration of the impact mass into the slab.

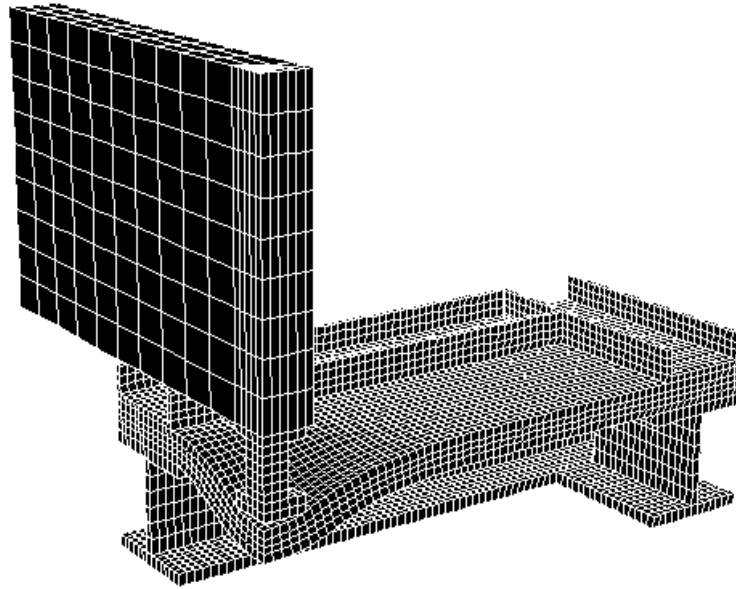


Figure 4. Deformation of Simulated Slab

The maximum displacements of elements and associated times of maximum displacements are listed for all slabs in Table 2. All numerical simulations for all cases were run to a maximum of 30 ms, which was determined from the experimental test. A comparison of the deflection at two mid-points, where the deflection gages were attached to the slab, along the longitudinal length of the slab with the numerical results is also presented in Table II.

The overall deflection of the tests and the numerical simulations are similar in the behavior of the deflected shapes. The deflections at the center of the slab are linear throughout the tests and the numerical simulations. Deflections at DG2 peaked in the positive direction for all tests and simulations with a variation in time based on the speed of the impact mass. DG2 data is off from the simulation due to problems with the ball joint attachment. The actual column in Table II represents the experimental test results and the simulation column represents the numerical results. From the above data we can state that despite the challenge of mimicking the physical behavior of the impact test, the simulation provided us with a reasonable estimation of the actual test deflections. Variations in the recorded time between the actual tests and simulations are caused by the initial trigger time of the drop mass during the actual impact tests [7].

Table 2. Maximum Displacements

Slab ID.	Act. Δ_{max} , mm		Sim. Δ_{max} , mm		Act. Δ_{max} , mm		Sim. Δ_{max} , mm	
	DG 1	T(ms)	DG 1	T(ms)	DG 2	T(ms)	DG 2	T(ms)
1#3-1	102	62	86	30	28	19.9	2.5	30
2W-1	102	40.4	89	30	28	21.1	2.5	22
2#3-1	71	56.1	86	30	36	38.9	2.5	17
1#3-2	66	96.0	61	30	23	28.7	2.5	30
2W-2	102	87.3	61	30	25	26.5	2.5	23
2#3-2	43	48.5	61	30	23	21.5	2.5	27
1#3-3	102	35.7	127	30	58	62.3	2.5	24
2W-3	102	28.6	127	30	36	15.8	2.5	21
2#3-3	102	22.7	122	30	31	15.1	2.5	27

DG – Deflection Gage

As mentioned previously deflection gage 1 (DG1) was located at the center of the slab. Deflection gage 2 (DG2) was located at the middle point between DG1 and the support steel frame along the major axis of the slab.

4. Comparison of Loads

In order to simulate the response of the slabs to impact loading, it was considered essential to replicate, as closely as possible, the impact pulse of the impact tests. A stronger but stiffer numerical model will incur a higher dynamic load from the actual test. Table 3 presents a summary of peak loads from the impact tests and the numerical simulations. It must be noted that the peak of the applied load and the deflection are interrelated. The more "rigid" the slab was, the higher the peak load and the lower the displacement.

Table 3. Summary of Loads

Slab ID.	Drop Height in (mm)	Experimental Load		Simulated Load	
		Load, kips (kN)	T (ms)	Load, kips (kN)	T (ms)
1#3-1	12 (305)	47.75 (212)	0.684	80 (356)	1.0
2W-1	12 (305)	78.44 (349)	2.47	140 (623)	1.0
2#3-1	12 (305)	84.15 (374)	0.869	150 (667)	1.0
1#3-2	6 (152)	51.34 (228)	0.597	60 (267)	1.0
2W-2	6 (152)	63.65 (283)	0.960	100 (445)	1.0
2#3-2	6 (152)	68.26 (304)	0.895	100 (445)	1.0
1#3-3	24 (610)	124.2 (552)	0.818	120 (534)	1.0
2W-3	24 (610)	98.22 (437)	0.817	170 (756)	1.0
2#3-3	24 (610)	121.1 (539)	0.541	200 (890)	1.0

The numerical simulations over-estimated the test loads by 40% in most cases. This was anticipated because the numerical models were more "rigid" and the actual concrete materials in the experimental tests were weaker. The numerical simulations with two steel meshes gave better load-time history definitions than those with one mesh. This can also be related to the location of the reinforcement (thicker layer of concrete cover). On the other hand, 1#3-2 and 1#3-3 simulations are very close to the actual test values because of the lesser amount of elements bond fixity between the steel and concrete (models were less rigid).

5. Comparison of Concrete Stresses

The average results of testing concrete cylinder to determine the concrete compressive strength were 4,000 psi. Under this circumstance an estimation of 4,000 psi concrete was used in the numerical simulations. No strain measurement of concrete was acquired during testing and the only physical property left to be compared is the simulation stresses against the average concrete compressive strength of 4,000 psi. Figure 5 shows maximum concrete stresses distribution throughout the finite element mesh and Table 4 presents a summary of the simulation stresses.

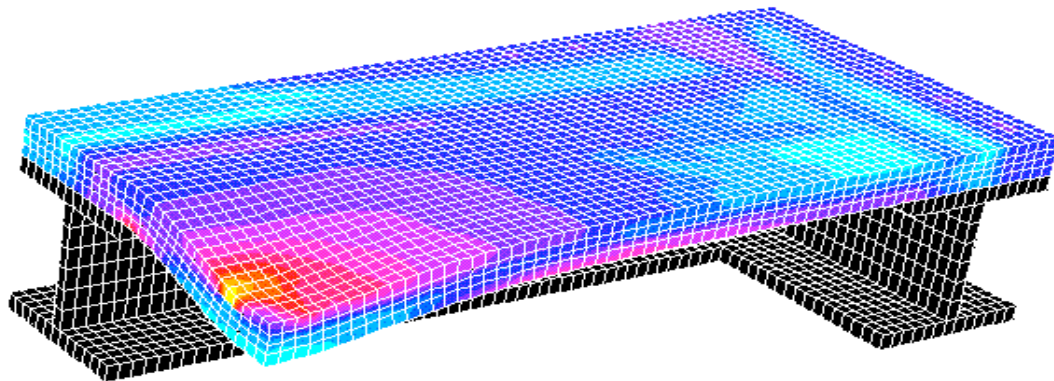


Figure 5. Concrete Stresses Distribution

Table 4. Summary of Simulated Stresses for Concrete

Slab ID.	Drop Height, in (mm)	Simulated Stress, psi (kPa)	Concrete Compressive Average Strength, psi (kPa)
1#3-1	12 (305)	2,768 (19,085)	4,000 (27,579)
2W-1	12 (305)	3,557 (24,525)	4,000 (27,579)
2#3-1	12 (305)	3,979 (27,434)	4,000 (27,579)
1#3-2	6 (152)	2,496 (17,209)	4,000 (27,579)
2W-2	6 (152)	3,994 (27,538)	4,000 (27,579)
2#3-2	6 (152)	3,472 (23,939)	4,000 (27,579)
1#3-3	24 (610)	2,974 (20,505)	4,000 (27,579)
2W-3	24 (610)	3,047 (21,008)	4,000 (27,579)
2#3-3	24 (610)	3,472 (23,939)	4,000 (27,579)

6. Conclusions

A better correlation between predicted and measured responses might be achievable if a more representative material model for the concrete under impact could be implemented numerically. Currently, the material models used for concrete do not account for concrete failure by punching shear. In addition, a more accurate representation of the physical condition of the impact mass, i.e. an explicit representation of the drop mass catching mechanism, should be included in the analyses. This option is not currently permitted in the finite element model due to the physical and mathematical complexities of the drop weight movement. These comparisons also revealed a need for a better model of the concrete-to-steel bond.

The modifications made to the numerical simulations during its comparison with the precision impact test did not improve the correlation with the test data, due to code limitation of the material models input parameters. The parameters of the numerical simulation needed to be changed after correlating the impact model to obtain a better representation of the precision impact test. The adjustments made to the impact model consisted of only altering the material models used for concrete and steel within the slab. No changes were made to the geometry or the boundary conditions of the impact meshes. As stated earlier the best correlation was obtained with the deflections and the reinforcement stresses within the concrete slab.

The precision impact test and numerical simulation were able with some degree of accuracy to identify the dynamic failure modes of reinforced concrete slab under localized impact.

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COMPARISONS OF MEASURED MODAL PROPERTIES WITH FINITE ELEMENT ANALYSIS PREDICTIONS FOR A COMPOSITE SLAB FLOOR

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Abstract

Floor vibration serviceability is a primary design consideration for steel framed composite floors. The accuracy of any floor vibration evaluation procedure depends on a reasonably accurate prediction of modal properties. To consider multiple modes, a procedure must also accurately predict the spacing of modal frequencies. The goal of this paper is to investigate the ability of current finite element modeling techniques to accurately predict these modal properties. A composite floor slab was vibration tested to determine its natural frequencies, mode shapes, and acceleration frequency response functions. Analytical predictions were made using a commercially available finite element modeling program. The measured and predicted properties are compared to assess the accuracy of the modeling techniques.

Introduction

In recent years, floor vibration due to human activity has become widely recognized by structural design engineers as an important limit state for steel-framed floor systems. While in decades past, design engineers focused their attention exclusively on strength and deflection limit states, it is now common to evaluate floor vibrations for most, if not all, projects constructed using steel-framed systems. The AISC *Design Guide Series 11, Floor Vibrations Due to Human Activity* (DG11) (Murray et al. 1997) and SCI *Design of Floors for Vibration: A New Approach* (SCI DG) (Smith et al. 2007) provide engineers with evaluation criteria for most common situations and also have served to enhance awareness of the importance of routinely considering this limit state.

The methods found in DG11 are based on single degree of freedom (SDOF) approximations of the floor bay under consideration. The SCI DG includes both a SDOF simplified method and a general method based on finite element analysis of the floor system. The SDOF methods used in DG11 and SCI DG were developed for cases with regular framing, but can be extended to other cases using engineering judgment. Finite element based methods are desirable because it is assumed that they better predict modal properties for irregular situations. It is also well known that floor systems usually have numerous vibration modes in the frequency bandwidth

that can be excited by human walking—this can be considered using finite element modeling, but not by SDOF approximate methods.

The accuracy of any method based on finite element modeling depends on the model's ability to predict the natural frequencies, natural frequency spacing, mode shapes and sequence, and acceleration frequency response function. If these properties can be reasonably predicted, then forcing functions representing human walking can be applied to predict the acceleration response.

Previously, Barrett (2006) and Pavic et al. (2007) presented comparisons of measured and predicted modal properties for composite floor systems. Barrett (2006) reported that his models of two composite floors accurately predicted natural frequencies, within approximately 5%, but did not consistently predict the acceleration FRF (Frequency Response Function – complex number ratio of output acceleration to input force) shapes or peak magnitudes. Pavic et al. (2007) indicated that their unadjusted model developed using the best current engineering judgment and modeling techniques only predict natural frequencies within approximately 10-15%. Modes were also predicted in the incorrect sequence. The researchers did not present predicted versus measured FRF magnitudes, so the success of those comparisons is unknown.

This paper presents comparisons of measured modal properties of a composite slab floor with those predicted by finite element modeling. The objective is to further investigate the accuracy of the modeling techniques.

Description of Building Floor

The building was under construction when tested. It is four stories with 16800 m² of occupied space. The Second Floor (second elevated floor—ground floor is a parking garage) was chosen because its topside supported less construction material than the other floors at the time of testing. The Second Floor Framing plan is shown in Figure 1.

Minimal piping and ductwork had been installed below the test floor. The floor system is a conventional steel framed composite system. The normal weight concrete slab depth is 140 mm and the deck height is 51 mm. The tested bays are 9.14 m x 9.14 m; the interior of the building contains an elevator core surrounded by masonry shear walls and a large mechanical opening. The mass of each tested bay is approximately 22000 kg.

Experimental Program

Forced vibration tests were performed to obtain estimates of natural frequencies, mode shapes, and steady-state acceleration response due to sinusoidal load (accelerance). These tests were performed by applying measured burst chirp excitation at the bay centers using an electrodynamic shaker and measuring floor accelerations using accelerometers placed on the slab. The applied force was directly measured using a custom built force plate placed between the slab and the shaker.

The force level was chosen to result in driving point peak accelerations approximately equal to the anticipated peak acceleration due to walking.

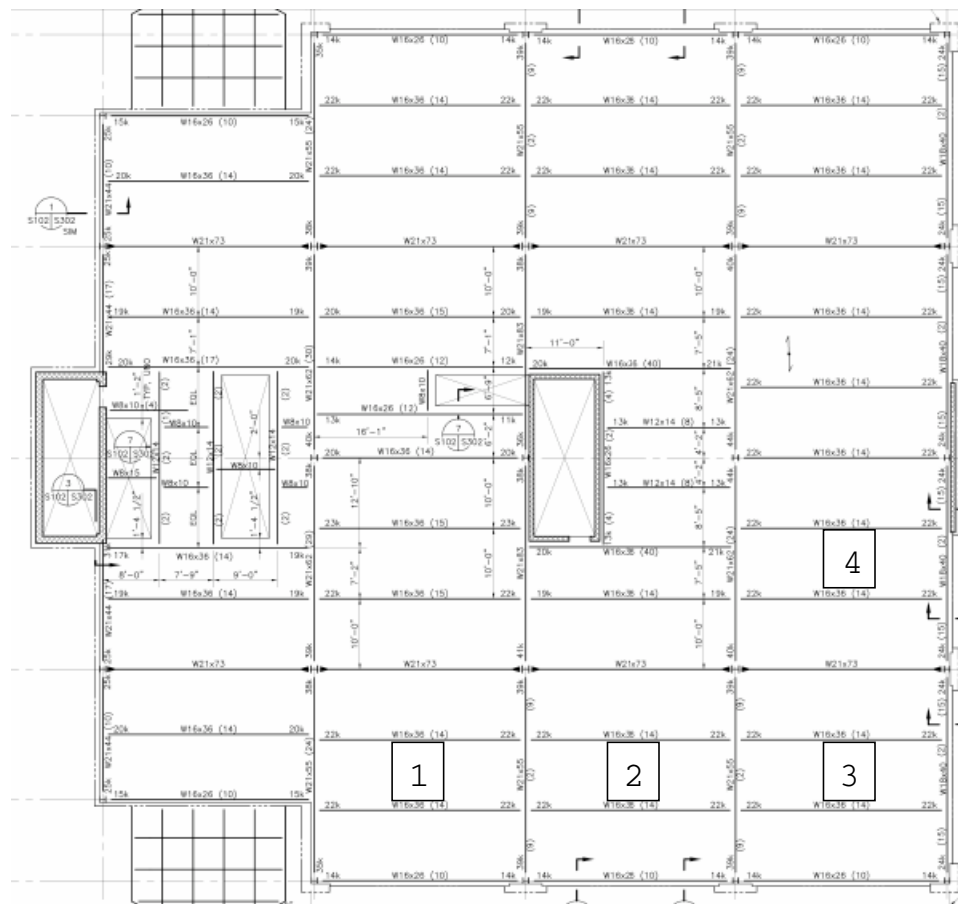


Figure 1: 2nd Floor Framing Plan, Including Tested Bay Numbers

Four bays were subjected to forced vibration tests, designated Bays 1 through 4 on the framing plan. These tests resulted in the driving point acceleration FRFs shown in Figure 2. Mode shapes are shown in Figure 3. The acceleration plots indicate several natural modes with frequencies ranging from 6.4 Hz to 8.1 Hz, with the dominant mode varying from bay to bay. Natural frequencies and acceleration peaks for significant modes are listed in Table 1. ME’scopeVES (Vibrant Technologies 2003) was used to curve fit the measured FRFs to determine the viscous modal damping ratios: 0.51%, 0.59%, 0.56%, 0.61%, 0.54%, and 0.51% respectively, for the modes shown in Figure 3.

Table 1: Accelerance FRF Peak Magnitudes Summary

Shaker in Bay 1		Shaker in Bay 2		Shaker in Bay 3		Shaker in Bay 4	
f_n (Hz)	A (%g / N)	f_n (Hz)	A (%g / N)	f_n (Hz)	A (%g / N)	f_n (Hz)	A (%g / N)
6.4	0.00643	6.4	0.0160	6.4	0.00700	6.6	0.00596
7.0	0.0146	8.1	0.0268	7.0	0.0247	7.1	0.00635
8.1	0.0316			8.1	0.0112	7.4	0.0161

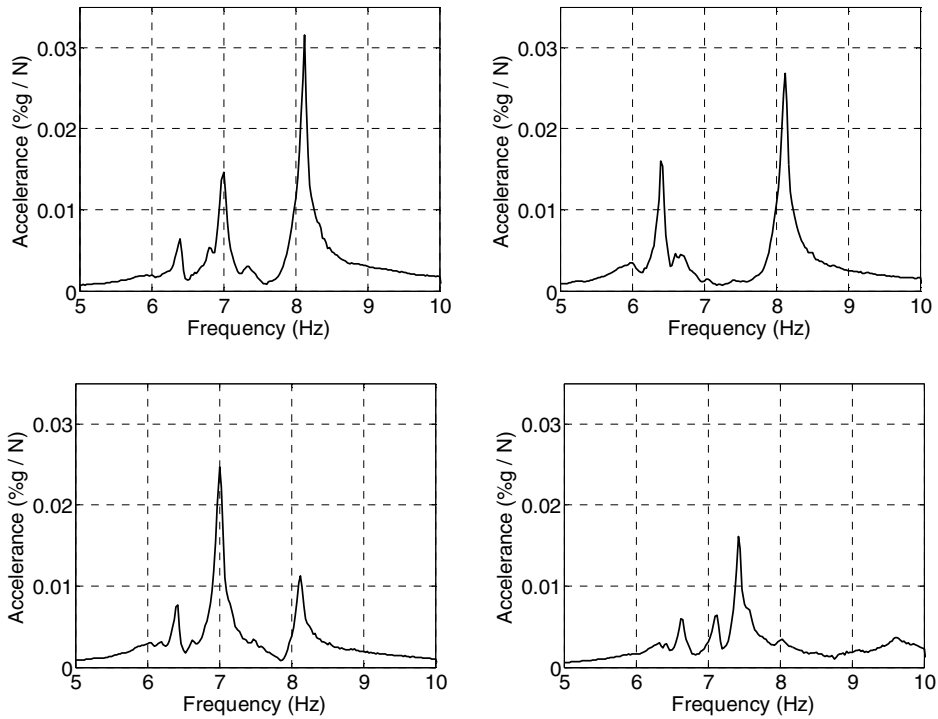


Figure 2: Accelerance FRF Magnitudes (a) Shaker in Bay 1; (b) Shaker in Bay 2; (c) Shaker in Bay 3; (d) Shaker in Bay 4

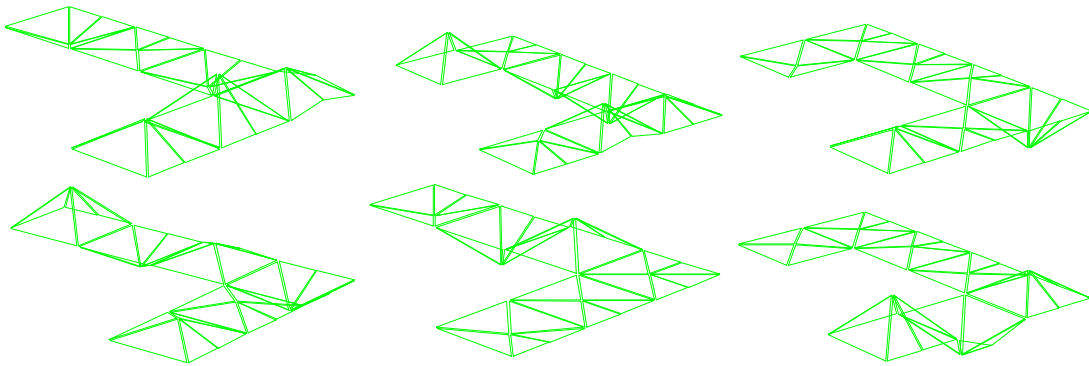


Figure 3: Measured Mode Shapes (a) 6.4 Hz; (b) 6.6 Hz; (c) 7.0 Hz; (d) 7.1 Hz; (e) 7.4 Hz; (f) 8.1 Hz

It is interesting to note that the modes at 6.4 Hz, 7.0 Hz, and 8.1 Hz resemble the first three modes of a three span continuous beam, except in the reverse order. The dominant mode in each bay is difficult to determine from viewing only the mode shapes, although the expected pattern appears to exist: the dominant mode has a shape with significant amplitude in the bay under consideration and little amplitude in most other bays.

Analytical Predictions and Comparison to Measurements

Analytical predictions of modal properties were made using a commercially available finite element analysis program, SAP2000 Version 10 (CSI 2005). A model of the floor slab was created and used to predict natural frequencies, mode shapes, and accelerance FRF magnitude. It was constructed using only the information that would be available to a designer (other than damping), rather than being updated or otherwise modified to force the analytical predictions to match the measurements. Orthotropic shell elements were used to model the floor slab and frames with transformed section properties, computed per DG11, were used to model the beams and girders. Beams and girders were placed in the same plane as the shell elements rather than using members placed at the steel beam mid-depth and connected to the slab using rigid links. Columns were also included in the model, extending halfway below and above the floor under consideration. All beam-to-girder and girder-to-column connections were modeled as moment connections because insufficient force and displacement occurs during floor vibration to cause connection slip. Spandrel elements were given an increased moment of inertia (by 2.5 times, per Barrett (2006)) to account for cladding restraint.

Steady state analysis was used to predict the driving point accelerance FRF at midspan of the four tested bays. An estimate of damping was required for use in this type of analysis. To create a more valid comparison, measured damping values were used rather than assumed values. (From previous experience, approximately 0.5% of critical damping is a reasonable assumption for bare slab floors such as this one.) The measured and predicted FRF magnitudes are shown in Figure 4. The model reasonably predicted the FRF in Bay 3, but did not reasonably predict the FRF in other bays. The ratio of predicted-to-measured peak magnitude was 2.0, 2.2, 1.1, and 2.1 for the four tested bays, respectively. Because measured damping was used, the main variable in question is modal mass.

Standard eigenvalue analysis was used to predict the floor's natural frequencies and mode shapes, which are shown in Figure 5. By comparing Figure 3 and Figure 5, it can be seen that the predicted mode shapes are not in the same sequence as the measured ones. Also, the model predicted several mode shapes that were not measured. The opposite is also true: some mode shapes were measured, but not predicted. Only three of the first six measured and predicted mode shapes can be directly compared. These are shown in Figure 6. Of these, the measured and predicted natural frequencies are within 11% of each other. In general, the model predicted natural frequencies that in the same bandwidth as the measured ones: 6.5 Hz to 8.1 Hz, approximately.

Consequences of Inaccurate Prediction of Modal Properties

Natural frequencies, FRF magnitudes, and mode shapes collectively form the definition of the physical system for prediction of acceleration response to walking. Regardless of the complexity of the subsequent walking acceleration prediction, the results will not be reliable if the physical system has not been reasonably approximated by the model.

Natural frequencies must be predicted with reasonable accuracy to allow selection of the walking force harmonic that can cause resonance. For example, if the predicted natural frequency is 7.0 Hz, then it is assumed that the 4th harmonic of the walking force is the only one that can cause resonance because the 3rd harmonic can only reach approximately 3 times 2.2 Hz (the maximum considered step frequency). If the actual natural frequency is 6.5 Hz, then the 3rd harmonic can indeed cause resonance and the applied force will be underestimated in the model. Fortunately, the 2nd, 3rd, and 4th harmonic amplitudes do not vary greatly relative to the overall variability in footstep forces. Also, the applied force varies with step frequency for a given harmonic of the walking force, although not by a great deal (Kerr 1998, Willford et al. 2006).

Natural frequency spacing must also be accurately predicted if multiple modes are to be taken into account in the analysis. For example, Willford et al. (2006) laid the groundwork for the SCI DG finite element based procedure and state that the procedures “deal exactly with closely spaced modes.” Of course, the success of the prediction will depend on whether the model’s closely spaced modes reasonably match those of the actual floor. This will be true of any response prediction procedure based on finite element modeling of the floor.

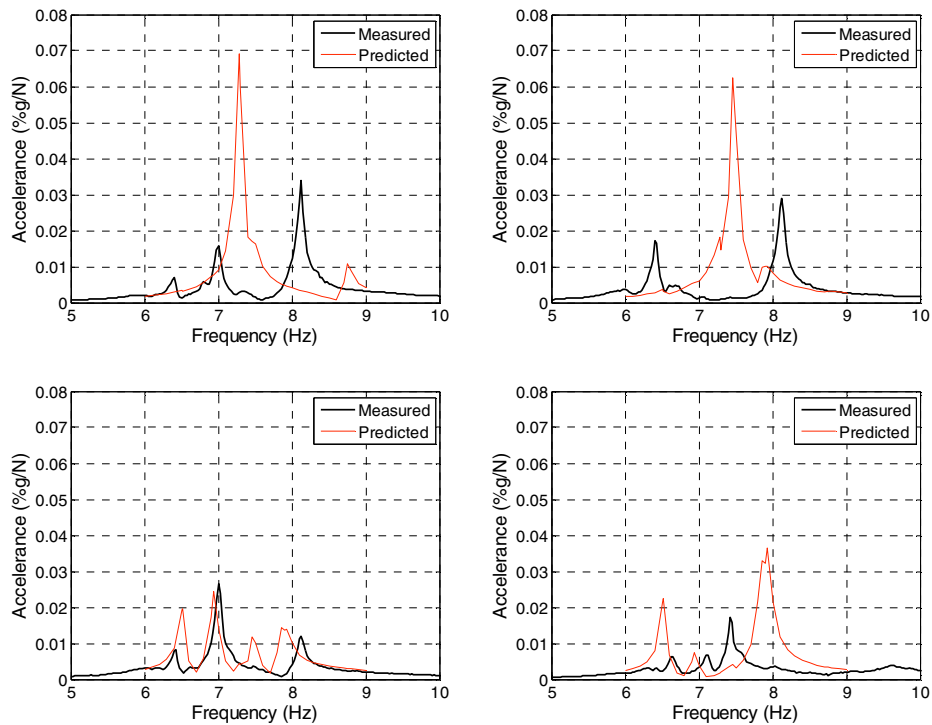


Figure 4: Measured and Predicted Accelerance FRF Magnitudes (a) Shaker in Bay 1; (b) Shaker in Bay 2; (c) Shaker in Bay 3; (d) Shaker in Bay 4

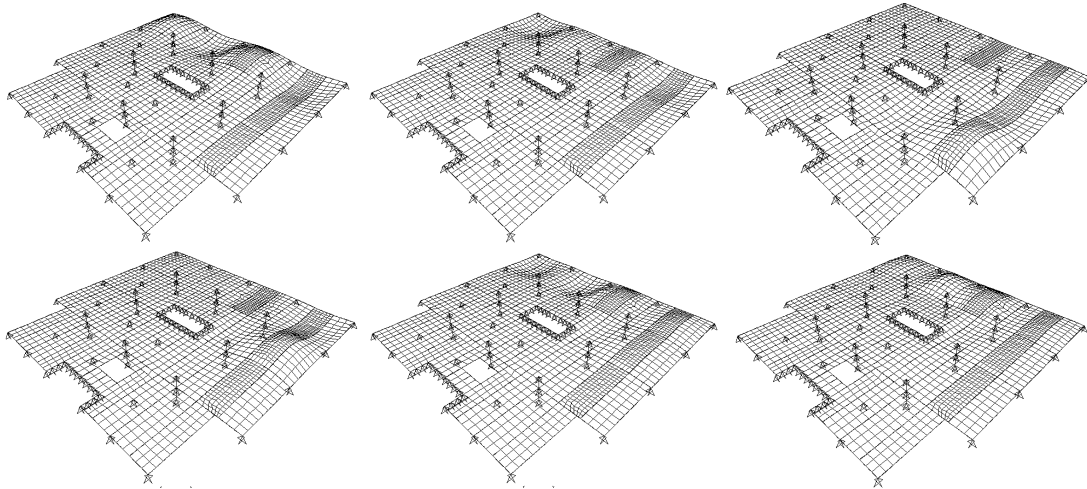
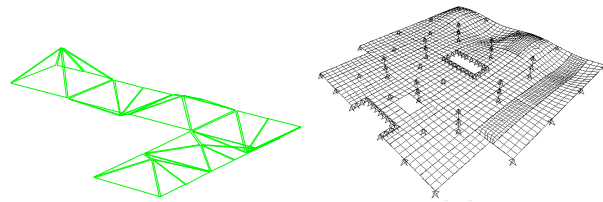
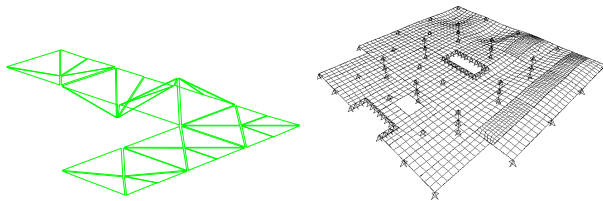


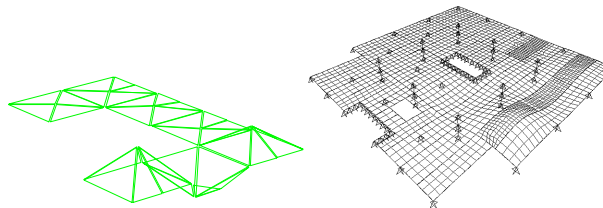
Figure 5: Predicted Mode Shapes (a) 6.5 Hz; (b) 6.9 Hz; (c) 7.3 Hz; (d) 7.5 Hz; (e) 7.9 Hz; (f) 7.9 Hz



(a) 7.1 Hz Measured, 6.5 Hz Predicted



(b) 7.4 Hz Measured, 7.9 Hz Predicted



(c) 8.1 Hz Measured, 7.3 Hz Predicted

Figure 6: Comparable Measured and Predicted Mode Shapes.

FRF magnitudes directly relate to the subsequent prediction of acceleration due to walking. As a simplistic example, an increase in FRF magnitude of two results in approximately a twofold increase in the prediction of acceleration due to walking. FRF peak magnitude is, in the authors' opinion, the single most important modal property. Unfortunately, it is the most difficult to accurately predict because it relates to damping and modal mass (which depends on mode shape), both of which are difficult, if not impossible to accurately predict using current means.

Summary and Conclusions

Several floor vibration evaluation procedures exist, some of which use SDOF approximations of the floor bay under consideration and some depend on the use of a finite element model of the floor system. It is likely that most or all future procedures will be based on finite element modeling. The goal of this paper is to investigate the accuracy of finite element modeling techniques for defining the physical system in terms of modal properties: natural frequencies, mode shapes, and frequency response.

A composite floor system was modal tested to determine its modal properties and experimental modal analysis techniques were used to estimate the FRF magnitudes, mode shapes, and damping. A finite element model was then created using a commercially available analysis program, SAP2000, and used to predict the modal properties for comparison with those measured. The model was not adjusted to force the results into agreement. Measured damping values were used, but the damping values were approximately equal to those that should be assumed for a bare slab.

The predicted results are in partial agreement with the measurements. Several natural frequencies were predicted in the same general frequency range as those measured, but only half of the predicted mode shapes correspond to measured mode shapes. Of these directly comparable modes, there is only partial agreement because the predicted shapes are in a different sequence than the measured shapes. The predicted frequencies are also not at the same spacing as the measured ones. Finally, the predicted FRF peak magnitudes are very significantly different from the measured ones, even though measured damping was used in the prediction.

Consequences of incorrect predictions of modal results were also discussed. Natural frequencies are used to determine the walking force harmonic and the amplitude of the harmonic force. The FRF peak magnitude directly relates to the acceleration due to walking, so its accurate prediction is essential. Reasonably accurate predictions of these properties are essential for the success for any acceleration prediction procedure, regardless of its complexity.

In conclusion, from the previous research and this study, it seems clear that finite element modeling techniques for prediction of natural frequencies and especially FRF peak magnitudes must be improved to allow accurate predictions of acceleration due to walking.

Acknowledgments

The authors thank Sarah Glenn, Barry Band, and Bob Foard of HSMM and Stephen Doyle of J.M. Turner for facilitating access to the facility. They also thank Dr. Mehdi Setareh and Dr. Anthony Barrett for their assistance during the experimental program.

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Effects of Bottom Chord Extensions on the Static and Dynamic Performance of Steel Joist Supported Floors

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INTRODUCTION

The use of lightweight and high strength materials in building construction have resulted in larger bays and longer spans in floor systems. Lighter materials in addition to the trend towards a “paperless” office decreases damping and the amount of live load on the floors. As a result, floors have become more vulnerable to annoying vibrations induced by human occupancy. The number of complaints by occupants has increased in recent years and floor vibrations have become an area of serviceability concern.

Activities like walking, dancing, running, jumping, aerobics, etc. generate floor vibrations and humans have different tolerance levels for these vibrations. Because humans are both the source and the sensor, human-induced vibration can not be isolated from the structure; vibration must be controlled by the floor system. Extensive research has been conducted on human perceptibility of floor vibrations and dynamic behavior of floor systems. The procedures in the AISC/CISC Design Guide 11- Floor Vibrations Due to Human Activity (Murray et al. 1997) are available for designers to determine acceptable acceleration levels and minimize excessive floor vibrations.

The best approach is to design floors that do not allow annoying vibrations, as remedies for floors susceptible to excessive floor vibrations are very expensive. Possible modifications of existing problem floors include adding mass, increasing damping (using partitions, damping posts or tuned mass dampers) and increasing structural stiffness of the floor systems.

Open web steel joists are very popular in steel building construction. The open web allows the duct work to be run through the web and maintenance can be done easily after the construction is completed. This makes open web steel joists very feasible and economical; however, vibrations of joist supported floor systems, and in particular, the effect of extended bottom chords, are still not very well understood. Extending joist bottom chords in an attempt to improve problem floors has been reported but without success. However, these retrofits were done after fit-out of the buildings and without introducing a preload into the extensions.

The purpose of this study is to examine the effect of bottom chord extensions on deflections and vibration characteristics of joist supported floor systems when joist bottom chord extensions are installed (a) before the concrete was placed, (b) after the fit-out with jacking for a single span footbridge, and (c) after the fit-out without jacking for a three span footbridge. Extensive analytical and experimental studies are performed on two laboratory footbridges with different bottom chord extension configurations to understand the effect of extended bottom chords on deflection, natural frequency, damping, mode shape and effective mass. Finite element computer models are created to simulate and compare the results of stiffness and vibration tests.

EXPERIMENTAL SETUP

A single span footbridge (Figure 1) and a three span footbridge (Figure 2) were constructed at the Virginia Tech Structures and Materials Research Laboratory, Blacksburg, Virginia. The single span footbridge (7 ft by 30 ft) was built inside the laboratory, while the three span footbridge (7 ft by 90 ft) was built outside. Both of the footbridges were constructed using 1.5VL deck (depth=1.5 in.) and a 4.5 in. normal weight concrete slab (total slab depth= 6.0 in.) supported on two parallel lines of 30K7x30 ft span, non-composite joists at 4 ft on center. Stand-off screws were used to connect the cold-formed steel decks to joist top chords before the concrete was placed.



Figure 1 Single Span Floor (7 ft x 30 ft)



Figure 2 Three Span Floor (7 ft x 90 ft)

Two concrete walls were used as supports for the single span bridge. The joists were welded to the bearing plates located on the concrete walls. The same bearing walls were used as exterior supports for the three span footbridge; built-up cross sections were used as interior supports. At the interior supports, the two joist top chords were welded to each other using a steel bar to provide top chord continuity. Bottom chord continuity was provided by HSS1.5x1.5x3/16 members (Figure 3).



a) At Exterior Support



b) At Interior Support

Figure 3 Bottom Chord Extensions

The bottom chord extensions were instrumented with load cells (Figure 4) to monitor the axial load levels which were recorded using a data acquisition system in static loading tests. To determine the effect of bottom chord extensions on stiffness and vibration characteristics, both the single span and three span footbridges were tested with different bottom chord extension configurations (Figures 5 and 6). Stiffness and vibration tests were repeated for each stage after modifications on joist bottom chords. The three stages of the single span footbridge and five stages of the three span footbridge include all the bottom chord extension configurations needed to compare the two cases: removing the bottom chord extension elements from the structure and re-installing them to an existing structure when the concrete slab is already in place and cured.

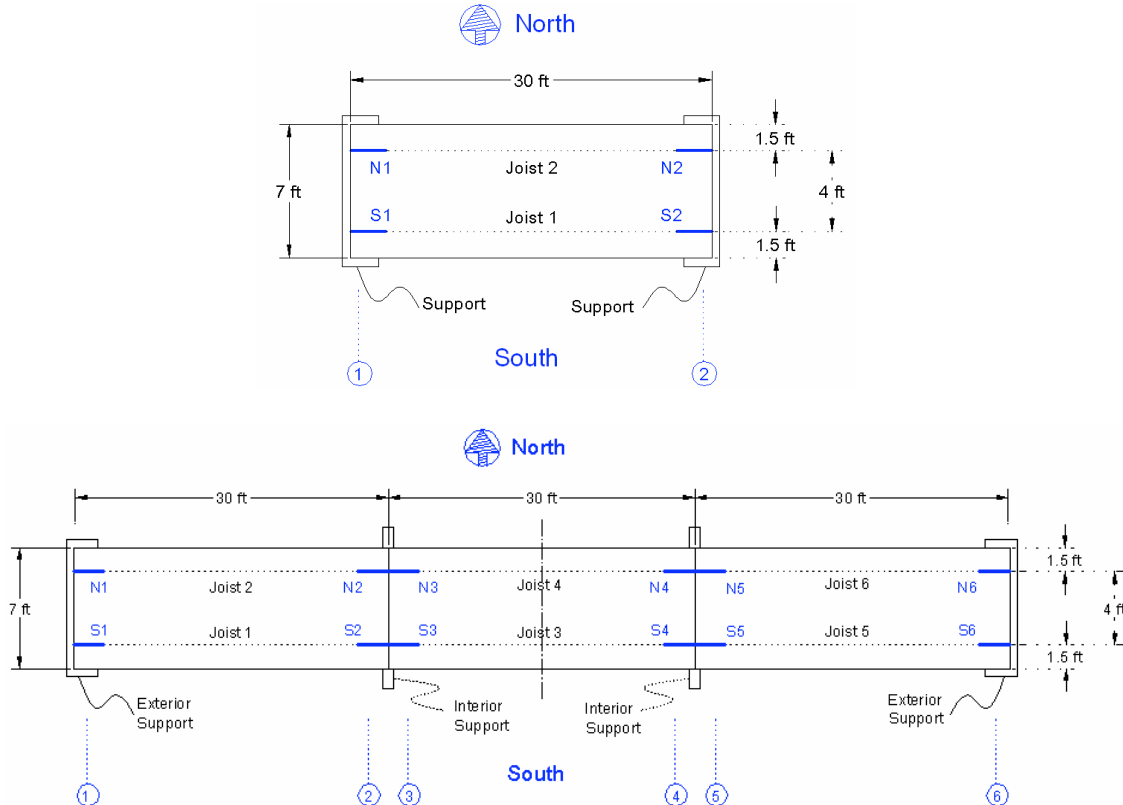


Figure 4 Load Cell Configurations- Single Span and Three Span Footbridges

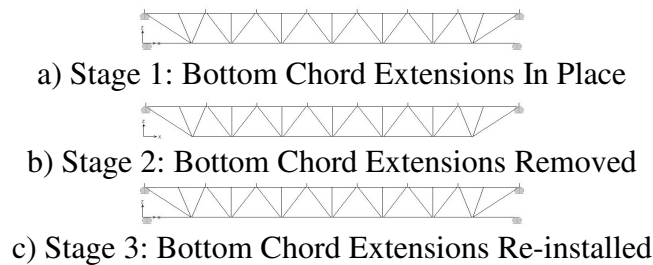


Figure 5 Bottom Chord Extension Configurations for Single Span Footbridge

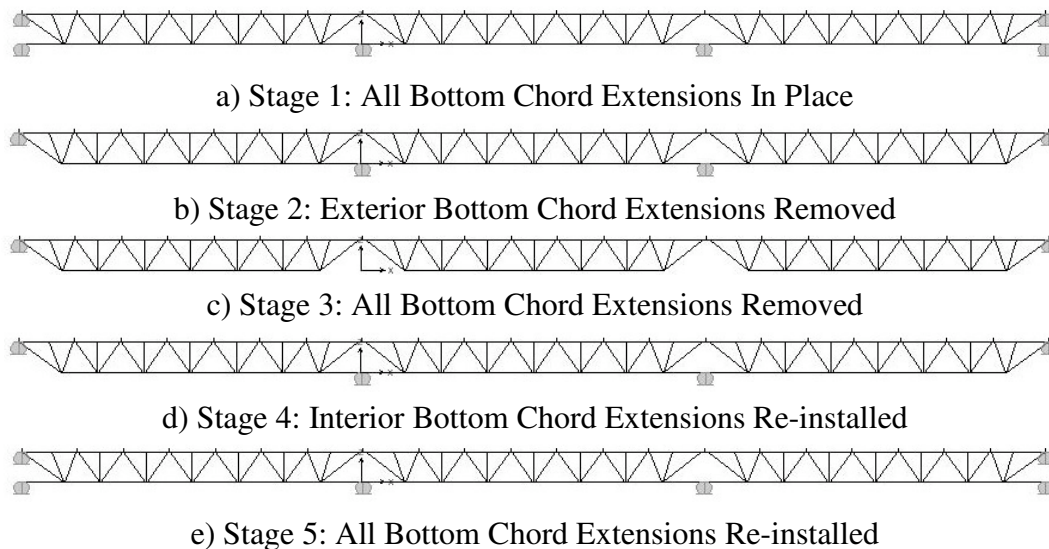


Figure 6 Bottom Chord Extension Configurations for Three Span Footbridge

FINITE ELEMENT MODELING

Three dimensional finite element models of the footbridges were created using SAP2000 program to match the stiffness test results of the bare joists and joists with cured concrete, for all bottom chord extension configurations. Then, the same models were used to analyze dynamic loading tests. Frame elements are used to model the chords and the web elements of the joists, in the program. Joint eccentricities are taken into consideration for the web member-chord nodes. The concrete slab is modeled with shell elements and connected to top joist chords with rigid links as described by Beavers (1998) and Sladki (1999). However, the use of fixed-fixed rigid links on every node of the joist top chords resulted in discrepancies between the measured deflections from stiffness tests and natural frequency values from vibration tests. Thus, the moment is released on the slab end of the links while the top chord ends of the links remain fixed as shown by Avci (2005). The rigid links are introduced on every other node of the joist top chord. All the chords and the web members of the trusses are pin-connected to each other. Using common support conditions like rollers and pins in the finite element model does not yield satisfactory results for these requirements. For that purpose, linear and rotational springs are

introduced to models to match the measured deflections, bottom chord extension forces and natural frequencies. Linear and rotational springs were also used by Hanagan (1994) and Rottmann (1996) in finite element models to match the experimental data.

STATIC TESTING: STIFFNESS TEST RESULTS

To experimentally determine the properties of the bare joists, the joists were loaded at midspan. Rectangular steel plates were placed in stacks to simulate “point” loading and vertical deflections were measured by dial gages placed under the bottom chords at the joist midspans. The deflections and bottom chord extension forces were monitored and recorded at 100 lb increments up to 600 lb on each joist. For both of the footbridges, the concrete was poured with all the bottom chord extensions in place. When the concrete was cured, the midspan point loading tests were conducted with all bottom chord extensions in place (Figure 7a). Then, a 20 psf uniformly distributed load was applied to the footbridges (Figure 7b). Steel bars were used for loading. Deflections and bottom chord extension forces were recorded.



a) Midspan Point Loading



b) 20psf Uniformly Distributed Loading

Figure 7 Static Testing of Footbridges

To determine the effect of the bottom chord extensions on joist stiffness, the stiffness test results are compared to the finite element model results and predictions from basic mechanics. When the joist bottom chord extensions are not in place, the joist member is assumed to behave like a simply supported beam. When the bottom chord extensions are in place, the member is expected to have more stiffness and approach the fixed-fixed condition.

For the midspan point loading of the single span bridge; the deflections at maximum loading (600 lbs at each joist midspan) increased from 0.025 in. to 0.037 in. (48% increase) when the bottom chord extensions were removed and decreased to 0.029 in. when they were re-installed. For reference, the corresponding predicted deflection based on simply supported condition assumption is 0.033 in. while the deflection for the fixed-fixed condition deflection is 0.008 in. For the 20 psf uniform distributed loading tests, the deflections at maximum loading increased from 0.0545 in. to 0.0675 in. (24% increase) when the bottom chord extensions were removed and decreased to 0.057 in. when they were re-installed. For reference, the corresponding predicted

deflection based on the simply supported condition is 0.071 in. while the deflection based on the fixed-fixed condition is 0.014 in.

From both point load and uniformly distributed load stiffness tests it is found that re-installing the bottom chord extensions (system jacked-up) to the joists of the single span footbridge with cured concrete helps to reduce the uniform load deflections to some extent, but not as much as placing the bottom chord extensions before the concrete pour. However, even the improved deflections are nowhere near the ideal fixed end case.

For the three span footbridge, it is realized that the deflections due the midspan point loading are reduced around 20% when both exterior and interior bottom chord extensions are in place. The deflections due to 20 psf UDL loading are reduced around 30% on the outside bays and about 10% on the interior bay when both exterior and interior bottom chord extensions are in place. It is found that re-installing the bottom chord extensions to the joists of the three span footbridge helps reduce the deflections, however, placing the bottom chord extensions before the concrete pour is observed to be a better solution. This is the same type of behavior observed in single span footbridge.

DYNAMIC TESTING: VIBRATION TEST RESULTS

The objective of vibration testing conducted in this study was to experimentally determine the natural frequency, damping, mode shape and effective mass properties of the laboratory footbridges and to determine the effect of bottom chord extensions for each configuration. The variation of the ratio of the acceleration response of the footbridge to the excitation force, also called Frequency Response Function (FRF), is the key to identify the effect of bottom chord extensions on the acceleration response.

Four types of excitations, chirp, sinusoidal, heel drop and walking, were used to excite the footbridges, as the acceleration response of the footbridges was measured through arrays of accelerometers placed on the surface of the footbridges. Chirp and sinusoidal excitations were generated using an electromagnetic shaker located on a force plate. Heel drops were also conducted on the force plate; however it was not possible to conduct walking on the force plate. For chirp, sine and heel drop excitations, frequency response functions were measured and recorded. For walking excitations, acceleration time histories of walking excitations were used for comparison of bottom chord extension effects in the different configurations.

For vibration tests, the shaker and force plate assembly were placed at different locations on the footbridge and chirp signals were used to sweep a frequency range of 4-20 Hz for each bottom chord extension configuration. Resonance frequencies for different modes were determined from the frequency response functions (FRF). Then sinusoidal excitations at system resonance frequencies were used to cause resonance, and response measurements were conducted.

MEScopeVES software was used for further modal analysis (curve fitting of all collected data and determination of the natural frequencies, modal damping and mode shapes for each bottom chord extension configuration). Once the mode shapes were known, then the effective masses for different modes were calculated for each stage.

For single span footbridge, accelerometer locations, first bending mode shapes from SAP2000 finite element model and MEScopeVES curve fits are shown in Figure 8. Natural frequencies and curve fitted modal damping ratios are shown in Table 1.

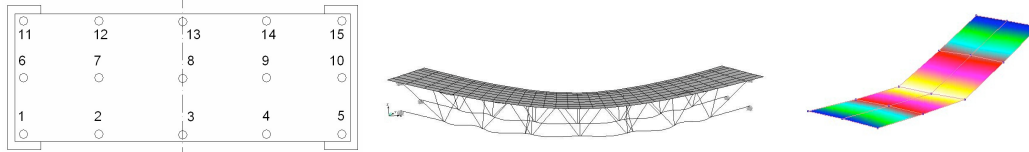


Figure 8 Single Span Footbridge- First Bending Mode

The natural frequency for the first bending mode is the highest for Stage 1, as expected. Removing bottom chord extensions from the system (Stage 2) caused a drop (8.08 Hz to 6.95 Hz) in the natural frequency. Re-installing the bottom chord extensions by jacking up the footbridge (Stage 3) increased the natural frequency of Stage 2 (6.95 Hz to 7.65 Hz) but it did not bring the natural frequency back to the original value of Stage 1. It is noted that the static test deflection results are consistent with the natural frequency results. However, having, removing and re-installing the bottom chord extensions do not have any significant effect on the modal damping ratio of the first bending mode.

Table 1 Single Span Footbridge- First Bending Mode Modal Parameters

	MEScope Results		FE Model
	f_n (Hz)	Modal Damping Ratio (%)	f_n (Hz)
Stage 1	8.08	0.451	7.99
Stage 2	6.95	0.448	7.03
Stage 3	7.65	0.409	7.99

For the three span footbridge, accelerometer locations are shown in Figure 9. The first three bending mode shapes from SAP2000 finite element model and MEScope curve fits are shown in Figure 10. Natural frequencies and curve fitted modal damping ratios are shown in Table 2.

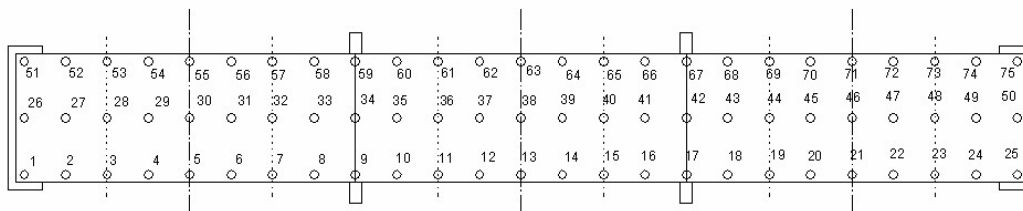
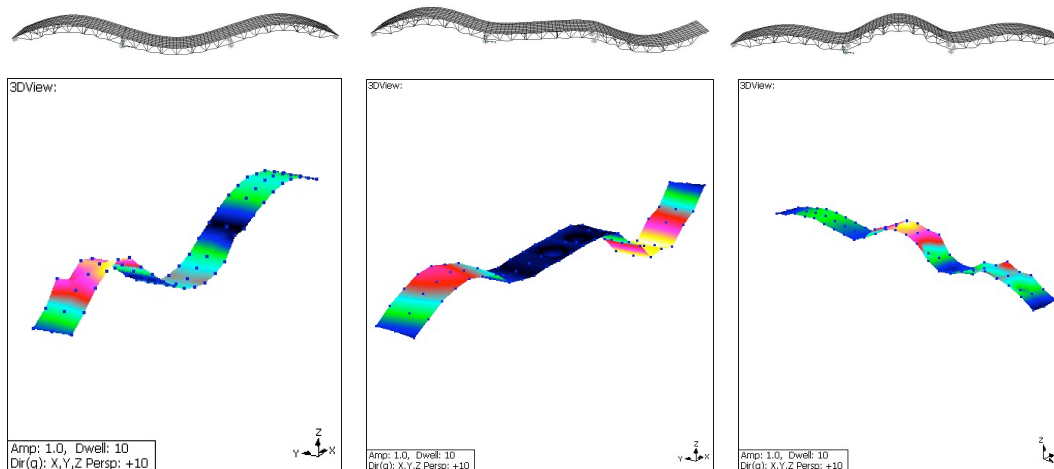


Figure 9 Accelerometer Locations- Three Span Footbridge



a) Second Bending Mode b) Second Bending Mode c) Third Bending Mode

Figure 10 Three Span Footbridge- First Three Bending Modes

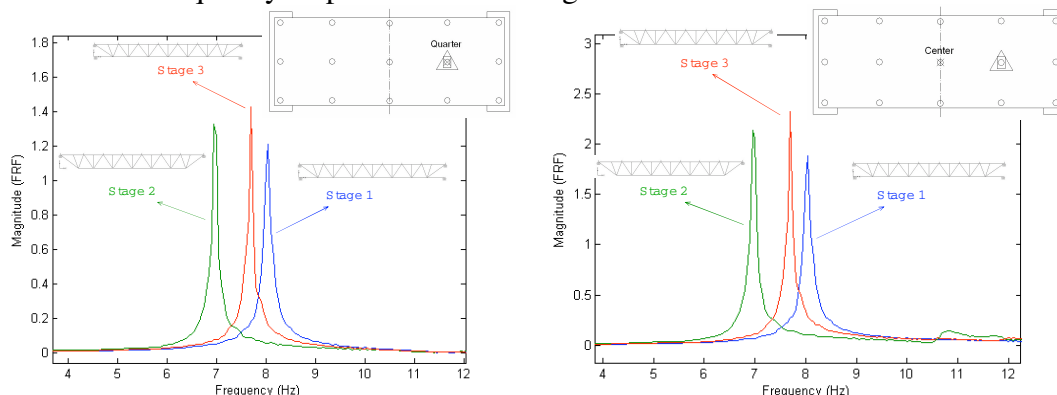
The natural frequency for the first bending mode is highest for Stages 1 and 5 for all three bending modes, which was expected. Removing the exterior bottom chord extensions for Stage 2 decreased the natural frequency values for all three bending modes and removing all of the bottom chord extensions for Stage 3 decreased the natural frequency values for all three bending modes, both as expected. Re-installing the interior bottom chord extensions for Stage 4 increased the natural frequency values for all three bending modes, as expected. Re-installing the exterior bottom chord extensions for Stage 5 further increased the natural frequency values for all three bending modes, as expected. It is noted that the static test deflection results are consistent with the natural frequency results. Like the single span footbridge, it is realized that removing and re-installing the bottom chord extensions did not have any significant effect on the modal damping ratios of the three span footbridge.

Table 2 Single Span Footbridge- First Bending Mode Modal Parameters

	First Bending Mode			Second Bending Mode			Third Bending Mode		
	MEScope Results		FE Model	MEScope Results		FE Model	MEScope Results		FE Model
	f_n (Hz)	Modal Damping Ratio (%)	f_n (Hz)	f_n (Hz)	Modal Damping Ratio (%)	f_n (Hz)	f_n (Hz)	Modal Damping Ratio (%)	f_n (Hz)
Stage 1	7.76	0.265	7.76	8.11	0.255	7.88	8.93	0.332	9.76
Stage 2	7.51	0.272	7.49	7.90	0.361	8.10	8.81	0.539	9.63
Stage 3	7.49	0.281	7.49	7.81	0.402	7.78	8.37	0.213	8.70
Stage 4	7.60	0.273	7.49	8.14	1.440	8.10	8.78	0.203	9.63
Stage 5	7.80	0.178	7.76	8.13	0.235	7.88	8.89	0.571	9.76

The chirp FRF comparisons between Stages 1, 2 and 3 were made for the outside quarter point excitation location as shown in Figure 11. The results of the accelerometer placed next to the shaker are shown in Figure 11a while the results of the accelerometer placed at the footbridge center are displayed in Figure 11b. It is shown in Figure 11 that removing bottom chord extensions from the system (Stage 2) caused about a 10% increase in the frequency response function magnitude. Re-

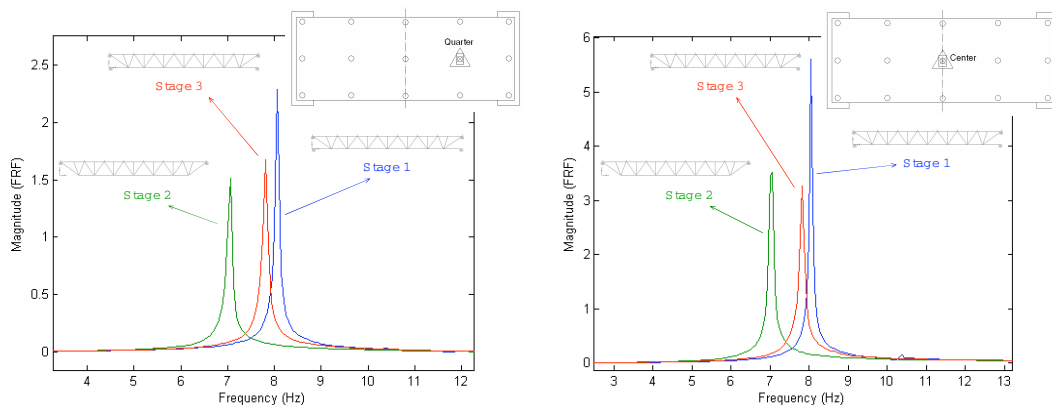
installing the bottom chord extensions by jacking up the footbridge (Stage 3) increased the frequency response function magnitudes about 8%.



a) Accelerometer at Outside Quarter Point b) Accelerometer at Center Point
Figure 11 Chirp FRF comparisons for Quarter Point Excitations- Single Span Footbridge

To further investigate the footbridges, heel drop excitations were performed on the force plate. For the single span footbridge the shaker was placed at the center and quarter point locations of the footbridge. Heel drop data was recorded for all three bottom chord extension configurations. The FRF comparisons between Stages 1, 2 and 3 for the center and quarter point locations are shown in Figure 12.

As shown in the quarter point data in Figure 12a, removing the bottom chord extensions from the system (Stage 2) caused a 34% drop in the frequency response function magnitude. Re-installing the bottom chord extensions (Stage 3) increased the frequency response function magnitudes about 11%. From the center point data in Figure 12b, removing bottom chord extensions from the system (Stage 2) caused a 37% drop in the frequency response function magnitude. Re-installing the bottom chord extensions (Stage 3) decreased the frequency response function magnitude about 7%.



a) Accelerometer at Quarter Point b) Accelerometer at Center Point
Figure 12 Heel-drop FRF Comparisons for Quarter and Central Point Excitations- Single Span Footbridge

The stiffness tests showed that Stage 1 was the stiffest configuration and the question was whether the bottom chord extensions will effect the frequency response function of the footbridges or not. It would be expected that as the structure gets stiffer the FRF peak magnitudes get higher, in other words the Stage 1 peaks are expected to be higher than Stage 2 FRF peaks. It is observed that this is indeed the case for the heel drop data of Figure 12 but not for the chirp data of Figure 11. As seen in Figures 11 and 12, the effect of bottom chord extensions on acceleration response of Stages 1 and 2 does not follow a specific pattern in the levels of frequency response function magnitudes.

Also, for sinusoidal excitations (data not shown in this paper), when Stage 1 and Stage 2 are compared, it is realized that the frequency response function magnitudes are very close to each other for relatively higher excitation amplitudes (27 lbs and 44 lbs). However, for lower excitation amplitudes (12 lbs), the frequency response function value is 20% higher in Stage 2 than in Stage 1.

The inconsistency in the FRF peak magnitudes is also observed in the vibration response of the three span footbridge. The acceleration response of the three span footbridge is governed by different contributions of the first three bending modes; as the stiffness of the structure increases or decreases, the response of the system to an excitation is not necessarily affected at the same level for the three modes.

CONCLUSIONS

From deflection control point of view, it can be concluded that re-installing the bottom chord extensions to the joists of the footbridges helps reduce the deflections; however placing the bottom chord extensions before the concrete placement gives better results.

From vibration serviceability point of view, it can be concluded that the addition of the bottom chord extensions do increase the natural frequencies for all three governing bending modes. However, although the footbridges were stiffened by the bottom chord extensions, that does not necessarily mean that the acceleration levels, and hence the frequency response function peaks, decrease.

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Fiber Reinforced Mortar Mixtures for the Reconstruction and Rehabilitation of Existing Masonry Structures

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Abstract

The majority of the world's architectural heritage encompasses unreinforced masonry structures, which are generally strong enough for the gravity loads, but are vulnerable against unexpected magnitudes of external out-of-plane loads and environmental effects. The recent trend of rehabilitating and strengthening unreinforced masonry with fiber reinforced polymer laminates, while effective, is not desirable in scenarios where aesthetics are important. Thus, there is a need for an equally effective yet aesthetically pleasing methodology. In this paper, the use of fiber-reinforced-mortars (FRMs) is proposed for masonry rehabilitation and reconstruction applications. Two types of fibers are considered; polyvinyl alcohol (PVA) fibers and novel organic fibers, such as corn silks. Several specimens with different type, size and volume fraction of fibers are tested in compression and flexure. Mixture proportions and the tested mechanical properties of each mixture are gathered in the form of a database. While the study is ongoing and the results are preliminary, it is found that certain volume fractions of PVA fibers are very effective in improving the toughness, ductility and durability of mortars. It is also found that corn silk FRMs show great potential for inexpensive and sustainable means to strengthen masonry structures.

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Introduction

Over the past decades, fiber reinforced concrete (FRC) has been studied extensively in its mechanical properties, application, and its behavior under various circumstances. It has been discovered that fiber reinforcement can in many ways enhance the performance of the concrete, and numerous equations and algorithms are proposed, which predict how the type, volume fraction and size of the fibers in the concrete relate to the compressive strength, bond strength, toughness, and modulus of elasticity. There has been a scarce amount of research, however, that is geared towards studying the application of fiber-reinforced cementitious mixtures in masonry structures.

Motivated with this gap in the field, in this study, both synthetic (PVA) and organic (corn silk) fibers are studied as a means to improve the mechanical characteristics of mortars that would be used in masonry structures. The scope is set such that the structures of concern are existing masonry buildings, but are damaged or fully collapsed, and a rehabilitation or reconstruction is pending. Therefore, the FRMs are evaluated for compatibility with older structures, and feasibility for joint repointing is among the concerns. To sustain compatibility, the FRMs are desired to be relatively low strength, especially in compression, to maintain the strong unit- weak mortar balance. Also in the scope of this preliminary work, a common higher-lime content and weaker-strength mortar, *i.e.* type N PCL mortar, is utilized. The mortar type is kept constant for all specimens while the fiber type, size and volume fractions are varied and the effects of these parameters on the FRM characteristics are studied.

Experimental Program

Two types of fibers are considered in the experimental program; polyvinyl alcohol (PVA) fibers and novel organic fibers, such as corn silks. These two fiber types are selected, because literature reports PVA fibers to be one of the most common and suitable reinforcements to use in engineered cementitious composites (Wang *et al.* 2005) and corn silk fibers are abundant by-products in Nebraska, the home of the authors. Three different PVA fibers are used (Figure 1). The corn silk fibers were not professionally manufactured and their dimensions are not exact, however, the aimed average length was one inch.

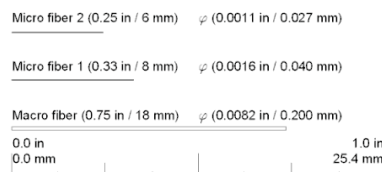


Figure 1: PVA Fibers Used in the FRMs

Seven mixtures are prepared using the ASTM C270 Portland cement/lime (PCL) type N mortar as the common binding matrix (Table 1). The first one is a control mixture without fibers, mixtures 2-6 are PVA-FRMs, and mixture 7 has includes corn-silk fibers.

Table 1: Experimental Matrix with PVA and Corn Fibers

Spec-mix lbs (kg)	Macro fiber lbs (kg)	Micro fiber 1 lbs (kg)	Micro fiber 2 lbs (kg)	Corn Silk lbs (kg)	Water lbs (kg)
160 (72.6)	-	-	-	-	33 (15)
160 (72.6)	2 (0.9)	-	-	-	35 (16)
160 (72.6)	-	1 (0.5)	-	-	35 (16)
160 (72.6)	-	-	1 (0.5)	-	35 (16)
160 (72.6)	1 (0.5)	1 (0.5)	-	-	35 (16)
160 (72.6)	1 (0.5)	-	1 (0.5)	-	35 (16)
12 (5.4)	-	-	-	0.2 (0.1)	2.6(1.2)

A total of 45 specimens are tested in this preliminary study, using the following three testing methods.

1. *Mortar compressive strength (ASTM C39)*: Three 6 in. by 12in. cylinders for each of the plain control mixture and five PVA-FRM mixtures (2-6) are tested for compressive strength according to ASTM C39. The mortar compressive strength (f'_c) and cylinder failure mode are reported for these 18 specimens.
2. *Mortar Flexural Strength (ASTM C1018, C78)*: Two 4in. by 4in. by 24in. beams for the plain control mixture and five PVA-FRM mixtures (2-6) are tested for flexural performance according to ASTM C1018. Toughness is measured in terms of areas under the load-deflection curve, and indicates the energy absorption capability of the test specimens. Modulus of rupture values are derived from the test results based on beam geometry and loading data (ASTM C78).
3. *Flexural bond strength between FRM and Units (ASTM E518)*: Two masonry prisms from the first 6 mixtures and three prisms from the corn silk FRM are prepared and tested according to ASTM E518. The flexural strength of unit-mortar specimens was obtained for each of the 15 specimens. The test specimens were simply supported in a horizontal orientation and vertical third-point loading was applied. The maximum load and mode of prism failure are recorded, and an experimental value for modulus of rupture is derived.

Experimental Results

The results of the above listed experiments are presented in this section along with the pertinent discussions.

1. Mortar Compressive Strength

The average compressive strength as well as the type of failure for each PVA-FRM mixture is provided in Table 2. When compared to the plain mortar, the FRMs both increase and decrease in compressive strength, a result that has been reported elsewhere (ACI 1988). The inclusion of fibers does not prevent the mortar from reaching the specified minimum strength of 750 psi for the ASTM C270 Type N mortar. Mixture 2 is an exception; however, the authors deduce that the low value may be due to the shear failure, characterized by a diagonal crack through the specimen as opposed to the shear/cone failure. Generally, shear failure occurs at a lower load than the actual compressive strength of the cementitious matrix (Kosmatka *et al.* 2002). Therefore, the pure compressive strength of the FRM mixtures meets the minimum specified compressive strength for all tested cases.

Generally, the addition of fibers to the mortar did not significantly affect compressive strength. This is the desired result because historic masonry is characterized by low compressive strength and a repair mortar should not introduce an uncharacteristically high strength joint. Mixture 5, a hybrid FRM of the two longer size fibers, shows the largest increase in compressive strength. For the same volume fraction of fibers, longer fibers decrease workability more than shorter fibers. Additionally, the longer fibers projected from the mortar joints between units in the prism specimens. Considering the excessive increase in compressive strength along with the workability and aesthetics problems presented by these longer fibers, the authors conclude that Mixture 5 is not a feasible FRM for masonry applications.

Table 2: ASTM C39 Report (Average values for each mixture)

Mixture	f'_c	
	psi (Mpa)	Fracture Type
1	900 (6.2)	shear/cone
2	630 (4.4)	shear
3	780 (5.4)	shear
4	970 (6.7)	shear
5	1370 (9.4)	shear/cone
6	800 (5.5)	shear/cone

2. Mortar Flexural Strength

The ASTM C1018 testing report is presented in Table 3. The data presented include the first crack strength ($P_{\text{first crack}}$), maximum strength (P_{max}), first crack deflection ($\Delta_{\text{first crack}}$), and the first crack modulus of rupture ($f_{r(\text{first crack})}$).

Table 3: ASTM C1018 Report (Average values for each mixture)

	$P_{\text{first crack}}$	P_{max}	$\Delta_{\text{first crack}}$	$f_{r(\text{first crack})}$
Mix	lb (N)	lb (N)	in (mm)	psi (MPa)
1	1342 (5970)	1342 (5970)	0.0089 (0.226)	234 (1.61)
2	737 (3280)	1184 (5270)	0.0071 (0.180)	128 (0.89)
3	1015 (4510)	1854 (8250)	0.0145 (0.368)	177 (1.22)
4	1774 (7890)	2494 (11090)	0.0092 (0.232)	309 (2.13)
5	1772 (7890)	3198 (14230)	0.0069 (0.175)	309 (2.13)
6	1554 (6910)	2510 (11170)	0.0115 (0.292)	271 (1.87)

Each FRM tested in this study carried maximum loads that are significantly higher than the cracking load. The first crack and maximum loads of all specimens are also higher than that of the plain mortar mixture with the exception of Mixture 2. All mixtures, including the lower strength mixture 2, has shown high ductility and extended post first-crack behavior (Figure 2).

Figure 2 compares load-displacement curves for plain, macro-fiber only, micro-fiber only and hybrid PVA-FRMs. The FRM load-displacement curves show two distinct regions. Elastic material behavior is characterized by a relatively linear curve from the initial load application up to the formation of the first crack. The first crack is illustrated by a sharp decrease in load with no corresponding increase in deflection. The remainder of the curve is considered plastic material behavior. In this region, reinforcing fibers crossing the first dominant crack and micro-cracks inside the beam begin to carry tensile stresses (Figure 3).

The post-crack capacity of FRMs containing macro fibers (Mixtures 2, 5 and 6) are greater than the FRMs reinforced with only micro fibers (Mixture 3 and 4). This result is expected because macro fibers carry tensile stresses across larger cracks and sustain more deflection than micro fibers. Hybrid fiber-reinforced mixtures presented the most significant increases in performance after cracking. However, this extended post-crack behavior, or ductility, may not be necessary for *masonry applications*. In the case of the masonry joint, the bond between the unit and the mortar would fail before reaching the strain and the deflection capacity of extremely ductile FRMs, such as mixture 2. Consequently, the maximum ductility and residual strength inherent to FRM are not achieved. Therefore, relatively high volume fraction macro fiber-reinforced mortars are not suitable to masonry applications. The authors suggest

that FRMs with micro fibers or with hybrid fibers at low volume fractions are more appropriate choices for masonry applications.

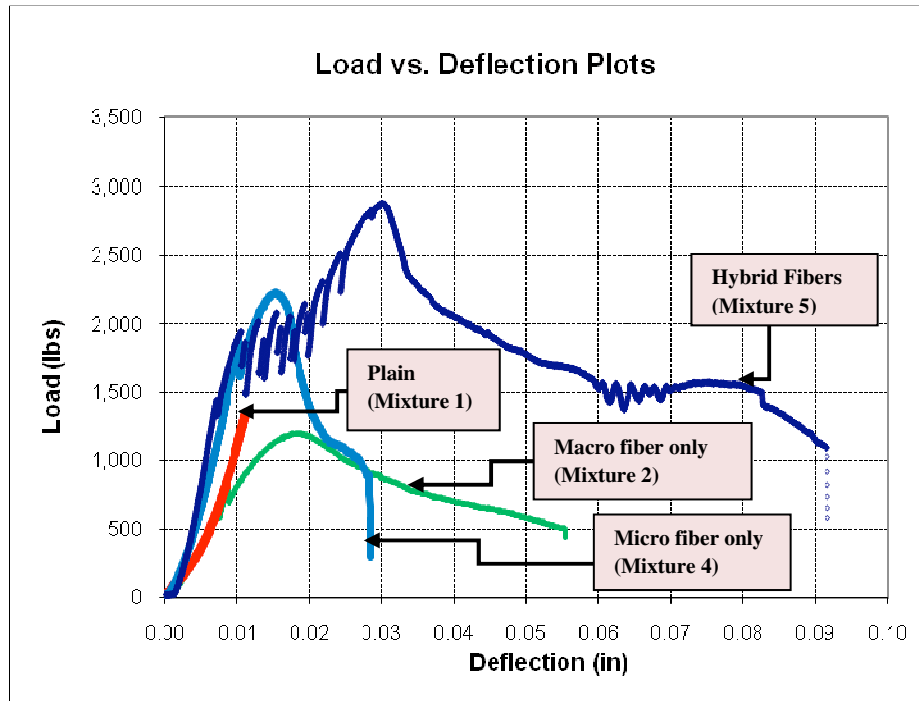


Figure 2: Comparison of Load Displacement Curves between Plain Mortar and PVA- FRMs



Figure 3: ASTM C1018 Specimens after failure: Plain mortar specimen (left), PVA- FRM specimen (right)

Toughness indices, residual strength factors and ductility values for all of the specimens are also calculated, however, they are not presented here due to space limitations. More detail of the study can be found in Skourup and Erdogmus (2008).

In summary, ASTM C1018 results show that the inclusion of fibers increases the toughness, ductility, energy absorption, and flexural load-carrying capacity of the mortars. However, the level of this increase must be optimized to provide feasible and effective mortar mixtures for masonry applications in the case of rehabilitation and reconstruction. Future studies should consider micro-fiber only or hybrid mixtures with total volume fractions of fiber less than or equal to 0.6%.

3. Flexural Bond Strength

Fifteen prisms are prepared with seven units each, using the seven mortar mixtures. These specimens are tested in bending according to ASTM E-518. The results of the experiments are summarized in Table 4. Each prism's modulus of rupture ($f_{r-prism}$) and failure load are recorded (P_{max}).

Table 4: ASTM E518 Report (Average values for each mixture)

Mixture ID	$f_{r-prism}$	P_{max}
	(psi)	(lbs)
Mixture 1- Plain	78	400
Mixture 2- PVA 1	92	479
Mixture 3- PVA 2	83	428
Mixture 4- PVA 3	111	581
Mixture 5- PVA 4	88	456
Mixture 6- PVA 5	78	398
Mixture 7- Corn silk	181	1,130

The average $f_{r-prism}$ values for all specimens meet the MSJC specified minimum f_r value of 75 psi, for prisms constructed with Type N mortar bending perpendicular to the bed joints (MSJC 2005). It is promising to note that the inclusion of fibers in the cementitious matrix does not significantly reduce the bond between the masonry unit and the mortar even if the largest fibers are used. Conversely, bond strength may be improved if the smallest size micro fibers are used in an optimized volume fraction, as in Mixture 4.

Corn fibers show a surprising increase in bond strength. The authors attribute this increase to the high moisture capturing behavior of the fresh corn fibers. During specimen preparations, the corn silk FRMs naturally increased the moisture content of the cementitious material, and made it almost too hard to work with due to the increased flow of the mortar. This increased moisture content, undoubtedly increased the absorption by the units and therefore improving the bond between the unit and the mortar. The authors deduce that with further study, an optimization of the observed advantages and disadvantages can be achieved. If processed, dried, or pre-soaked

corn fibers are used with a lower w/c ratio of the mortar mixture, the mortar can be made more workable while some of the bond-enhancing benefits of the organic fibers can be maintained. Processed and dried corn fibers are surely to be more durable in the long term, as well. Pre-soaking of PVA fibers should also be studied, as this may enhance the bond between PVA-FRMs and the masonry units.

Conclusions

Several conclusions are drawn from this preliminary study:

1. The addition of fibers to the mortar did not significantly affect the mortar compressive strength (except Mixture 5), which is the desired result.
2. Mixture 5, a hybrid of two longer fibers, provided significant increases in compressive strength and excessive increases in post-crack strength; however, also was the least workable mixture. Therefore this mixture is found to be ineffective for masonry applications.
3. All tested FRMs showed increases in post-crack ductility, toughness and energy absorption compared to the plain mortar. These are obvious benefits for blast-resistant masonry structures and masonry structures in seismic areas.
4. Both micro fiber PVA-FRMs showed higher first crack strength than the macro FRMs (Mixture 2). This is likely the result of the percent volume of fibers included in the mixtures. The macro FRM mixture contained twice the amount of fiber as the two micro FRM mixtures. The amount and size of these fibers required slightly more water to maintain workability, increasing the water-cement ratio and decreasing the final strength.
5. The hybrid fiber-reinforced specimens displayed the highest amount of post-crack strength among all FRM specimens.
6. Macro fibers can project from masonry joints and create an undesirable aesthetic in exposed structural elements. Mortar joints can be ground flat and smooth with an ordinary disc grinder once the mortar has cured to address this problem.
7. The post-crack capacity and residual strength of the FRM mixtures exceeded published ranges of expected values (ASTM 1997). Fanella and Naaman (1985) showed that improvements in FRM performance are more significant for lower matrix compressive strengths, a result that is verified here. This result suggests that even further economy can be achieved with FRMs because smaller amounts of reinforcing fibers than those used in this study will result in increased material performance at a lower overall cost.

8. Increased flexural capacity was observed in masonry prism tests. Micro PVA-FRM (Mixture 4) and the corn silk FRM (Mixture 7) provided the best results and should be further studied.
9. Micro PVA- FRMs and organic FRMs seem to be most appropriate for repointing and repairing mortar joints due to the workability and finishing characteristics of the material. Hybrid fiber-reinforced mortars are more suitable for reconstruction projects where the slightly decreased workability and concern of macro fibers protruding from joints are not primary considerations. Macro fiber only mixtures are not recommended for masonry applications.
10. A general understanding of how organic fibers affect the mechanical properties of mortars in masonry structures is gathered. The findings of this pilot study present the potential of organic fibers, which can be extrapolated to other similar organic fibers.

Further Work

Some empirical material property relationships for the PVA-FRMs are also developed as a result of this study, which can be found in Skourup and Erdogmus (2008).

Currently, further experiments with processed corn fiber FRMs and optimized volume fractions of PVA-FRMs are ongoing by the research team. Results of these findings will be added to the pilot database presented in this paper and will be published in the near future.

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IMPACT OF FASTENER-DECK ATTACHMENT ON THE WIND UPLIFT RESISTANCE OF MECHANICALLY ATTACHED ROOFING SYSTEMS

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Abstract

A roofing system (RS) consists of a waterproof membrane, mechanical attachments, cover board (if present), insulation, and vapor or air barrier (retarder – if present). A roof assembly (RA) is defined as an RS that includes a structural deck. Wind uplift ratings are obtained by subjecting RA mockups to dynamic wind loading. Mechanically Attached Roofing Systems (MARS) are one particular type of roof assemblies in which the membrane is attached to the structural deck using mechanical fasteners. The strength of the fastener-deck interface is an important aspect in the successful design of the wind uplift resistance of mechanically attached roof systems. To quantify the fastener-deck interface influence on the wind uplift performance of MARS, seven different roofing assemblies were constructed and tested under dynamic conditions. The experimental investigation identified three parameters namely deck grade, deck gauge and fastener type that have influence on the wind uplift resistance of MARS. Based on this component characterization, fastener pullout resistance (FPR) is identified as a verification factor for system wind resistance estimation.

Key words: *Steel decks, tensile strength, fasteners, pullout resistance, wind uplift, dynamic testing.*

1. Introduction

In conventional roof assemblies, the waterproof membrane is located at the top of the insulation and is directly exposed to environmental elements. The conventional roof systems may be either the Single-ply roof (SPR) or a Built up roof (BUR). Mechanically Attached Roofing System (MARS) represents one type of single-ply roofing system. In North America over 50% of low slope applications are made with MARS. Market survey indicates that there is a continuous growth for MARS. Nevertheless, MARS continues to face the greatest challenge of resisting the variable wind uplift forces that act upon them. Figure 1 illustrates the wind dynamics on MARS along the Resistance Link Diagram, a concept developed to evaluate its wind uplift resistance. As shown in Figure 1, in MARS, the insulation is located above the

deck. The waterproof membrane such as a thermoset, thermoplastic or modified bitumen is placed on top of the insulation and is attached to the structural deck using mechanical fasteners. The membrane attachment locations are then overlapped with the adjacent membrane to form seams. Wind-induced suction lifts the membrane between the attachments and causes membrane elongation and billowing. As shown in Figure 1, Force Resistance Link Diagram, all resistance links should remain connected for the system to be durable and in place. Failure occurs when the wind uplift force is greater than the resistance of any one or more of these links.

Mechanically Attached Roofing Assembly is defined as MARS that includes a structural deck. Henceforth, for easy reference the mechanically attached roofing assembly will be addressed as MARS throughout the paper. As seen from the force resistance diagram in Figure 1, the fastener-deck interface represents one of the resistance links to wind uplift forces. Its strength is characterized by a parameter known as Fastener Pullout Resistance (FPR).

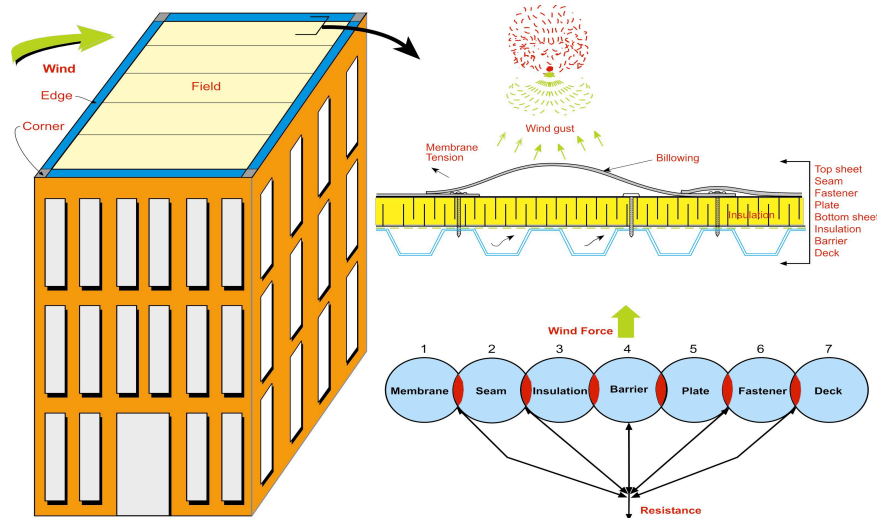


Figure 1: Wind Effects on Mechanically Attached Roofing System (MARS)

In MARS, the structural deck could be concrete or steel, however, the steel decks have been used for years and is the primary decking material. ASTM A1008 and ASTM A653 classifies steel deck for roofing applications, and the most common grades of decking in the roofing industry are Grade 33 -20, Grade 33 -22, Grade 80 -20 and Grade 80 -22 (Baskaran and Smith, 2005). The numbers, 33 (227 Mpa) and 80 (550 Mpa) represent the yield strength of the steel deck in kilopounds per square inch (kisi) while the number, 20 (0.91 mm) and 22 (0.76) represent the gauge/thickness of the steel deck. It is this difference in deck grades or minimum yield strength that is causing a lot of concerns and confusion with the roofing industry from the wind uplift perspective. Similarly, the roofing industry has many choices regarding the roofing fasteners available. Designing with the appropriate fastener-deck interface is essential for achieving the required wind uplift resistance of MARS.

To address the strength issue of fastener-deck interface, Baskaran et. al (2007) conducted fastener pullout experimental testing using three common roofing fasteners; namely; #14 [4 mm (0.17 in) dia.], #15 [7 mm (0.281 in.) dia.] and #21 [8.5 mm (0.331 in.) dia.], and the above mentioned four different steel decks. The results indicated that FPR increases with increase in the gauge and yield strength of steel deck, and fastener thread diameter.

Analyzing it from the component level, the present research extends to the assembly level by constructing seven different roofing assemblies with different fastener-deck combinations, and by testing under dynamic conditions. This experimental program has been initiated at NRC's IRC. The roof assembly responses are measured by two design indicators: pressure and force. By presenting a comparison between these seven roofing assemblies, the paper quantifies the role of fastener-deck interface on the strength of low slope roof assemblies.

2. Experimental Approach

Dynamic Roofing Facility and Wind Test Protocol

Experimental work was carried out at the Dynamic Roofing Facility (DRF), established at NRC's IRC. Baskaran and Lei (1997) provide a detailed documentation of the facility's features. As shown in Figure 2 (a), the DRF consists of a bottom frame of adjustable height upon which roof specimens are installed and a movable top chamber. The bottom frame and top chamber are 6100 mm (240 in.) long, 2200 mm (86 in.) wide and 800 mm (32 in.) in height. The top chamber is equipped with six windows for viewing, and with a gust simulator, which consist of a flap valve connected to a stepping motor through a timing belt arrangement. Wind suction as high as 10 kPa (209 psf) over the roof assembly is produced by a 50 HP (37 kw) fan having a flow rate of 2500 L/sec (5300 cfm).

All the constructed test specimens were subjected to the CSA A123.21-04 dynamic load cycle as shown in Figure 2 (b). Details of the method by which the load cycle was developed can be found in Baskaran et al (1999). As shown in the figure, it has five rating levels (A to E). To evaluate a roof assembly for a specific wind resistance, testing starts at Level A and continues by moving from one level to the next. To obtain a rating, all specified numbers of gusts in each level must be completed without any resistance link failure. As discussed in the Figure 1, the system is considered to have "Failed", if any one-resistance link fails.

Test Assemblies Setup

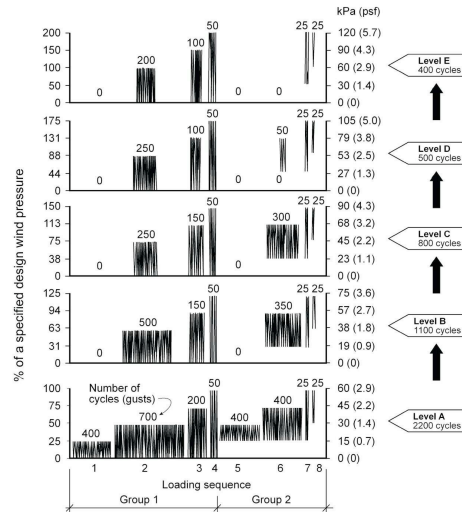
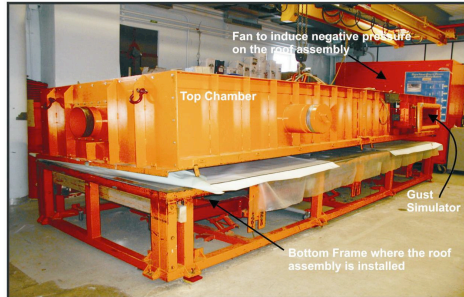
Using the above test facility (Figure 3) the present study evaluated seven assemblies with different fastener-deck combinations. These seven assemblies can be categorized into three sets based on the roof membrane type as:

- **Set 1: Thermoplastic Assemblies**

TP₂₂₋₈₀₋₁₅: Steel Deck = 22 Ga - 80 ksi, Fastener = #15, FPR = 3kN (685 lbf)

TP₂₀₋₈₀₋₁₅: Steel Deck = 20 Ga - 80 ksi, Fastener = #15, FPR = 3.9kN (893 lbf)

TP₂₂₋₈₀₋₂₁: Steel Deck = 22 Ga - 80 ksi, Fastener = #21, FPR = 5kN (1148 lbf)



(a) Dynamic Roofing Facility

(b) Dynamic Wind Test Protocol: CSA A123.21-04

Figure 2: Test Approach for Wind Uplift Testing of Roofing Assemblies

▪ **Set 2: Thermoset Assemblies**

TS₂₂₋₃₃₋₁₄: Steel Deck = 22 Ga - 33 ksi, Fastener = #14, FPR = 1.8 kN (420 lbf)

TS₂₂₋₈₀₋₁₄: Steel Deck = 22 Ga - 80 ksi, Fastener = #14, FPR = 2.9 kN (660 lbf)

▪ **Set 3: Modified-Bituminous Assemblies**

MB₂₂₋₃₃₋₁₄: Steel Deck = 22 Ga - 33 ksi, Fastener = #14, FPR = 1.7 kN (382 lbf)

MB₂₀₋₈₀₋₁₄: Steel Deck = 20 Ga - 80 ksi, Fastener = #14, FPR = 2.7 kN (651 lbf)

To evaluate the strength of the fastener-deck interface, the present study evaluated three different fastener types: #14, #15 and #21 with the combination of 22 Ga –33 ksi, 22 Ga – 80 ksi, 20 Ga – 80 ksi steel decks. The main difference in these fastener types, as shown in Figure 3, exists in the fastener head, tip, thread, and shank diameter. To reference each tested assembly type, the three parameters namely the deck gauge, yield strength and fastener type have been used.


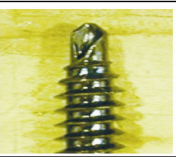
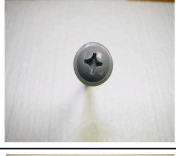

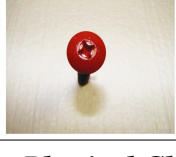
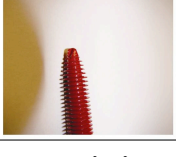
Fastener Type	Fastener Head	Fastener Tip	Head / Shank Diameter, in. (mm)
# 14			0.43 (11) / 0.170 (4.7)
# 15			0.44 (11.2) / 0.281 (7.1)
# 21			0.66 (16.8) / 0.331 (8.4)

Figure 3: Physical Characteristics of the Tested Fasteners

Apart from the variation in membrane type and fastener-deck interface, the other components of the roof assembly, namely, air/vapour barrier and insulation were similar for all the seven assemblies. As the roof installation is similar for all the tested assemblies the construction procedure can be classified into five steps:

1. **Deck Installation:** One full sheet of 914 mm (36 in.) wide and two cut pieces of 610 mm (24 in.) and 483 mm (19 in.) wide steel sheets were installed along the table length as shown in Figure 4. All the tested steel decks as mentioned above had a profile height of 38 mm (1.5 in.) and a flute width of 150 mm (5.9 in.). The black dotted lines as shown in Figure 4 indicate the deck overlaps. The decks were fastened to the steel joists at every flute.
2. **Barrier/retarder Installation:** For Set 1 and 2, a 150µm (6-mil) polyethylene film was used as a vapour barrier. To support the polyethylene film, a 12 mm (½ in.) gypsum board is laid above the steel deck and upon which polyethylene film was loose laid as shown in Figure 4. In Set 3, a self-adhered film (SAF) was used as a vapour barrier. The SAF is a 75µm (3 mil) thick sheet, which is adhered to the steel deck. It is composed of SBS modified bitumen and is surface-reinforced with trillaminate woven polyethylene. Figure 4 also illustrates the installation setup of SAF.

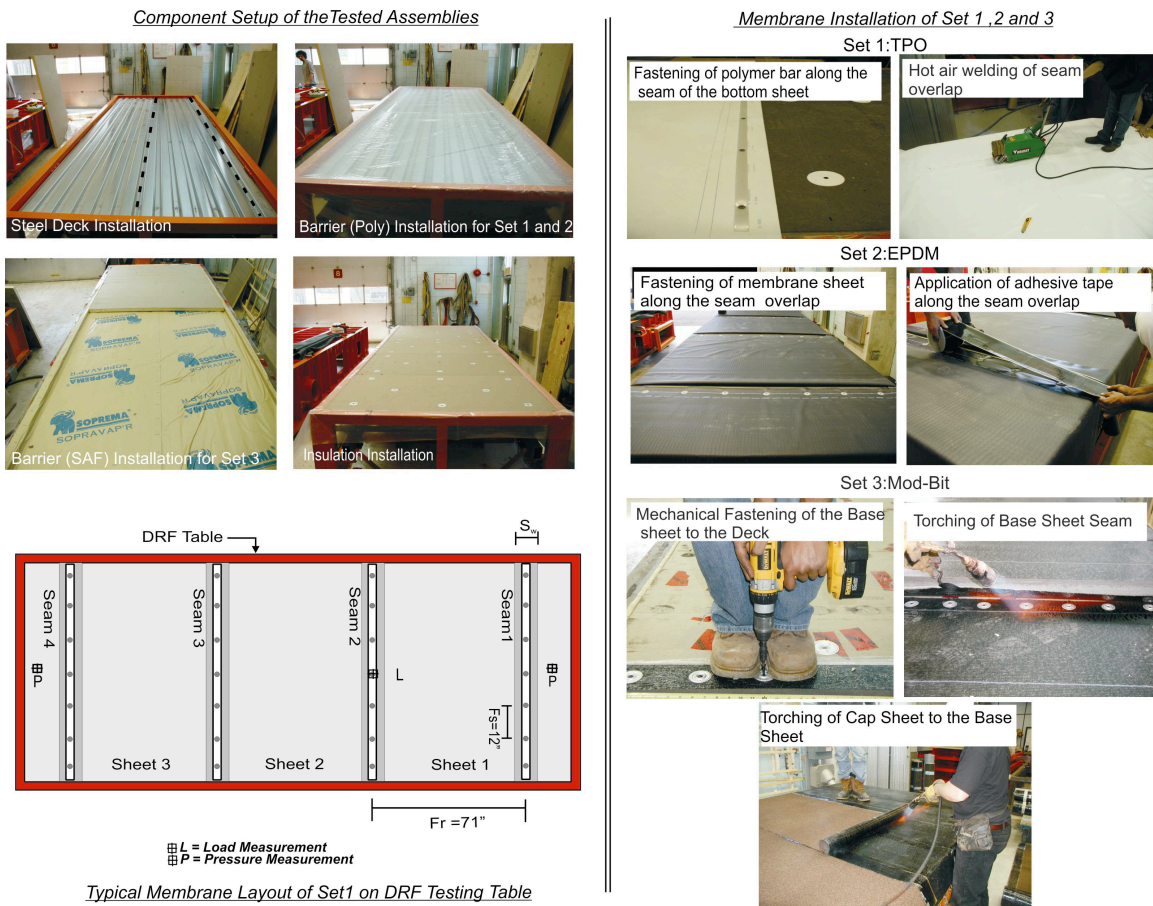


Figure 4: Assembly Setup of the Tested Roofing Assemblies

3. **Insulation Installation:** As shown in Figure 4, 51 mm (2 in.) thick polyisocyanurate insulation boards with a compressive strength of 170 kPa (25 psi) were used. The layout comprised of a single layer of insulation of four full boards of 1219 mm x 2006 mm (48 in. x 79 in.) and one partial board of 1118 mm x 2006 mm (44 in. x 79 in.) installed with the long edges perpendicular to the steel deck flutes. The insulation boards were mechanically fastened to the steel deck with eight fasteners per board.
4. **Waterproof Membrane Installation:** Figure 5 shows the typical installation procedure of the roofing membrane for the three sets. In Set 1, thermoplastic assemblies, a reinforced Thermoplastic Olefin (TPO) membrane sheet was used as the waterproof membrane. The membrane has a thickness of 1125 μm (45 mil) and a width of 1981 mm (78 in.). Geometric details of a typical tested assembly are also described in Figure 4. As shown in Figure 4, the layout comprised of three full-width sheets and a dummy sheet at each end. The four seams shown in Figure 4 indicate the fasteners' locations. The membrane attachment was done by fastening the sheets at fastener row spacing (Fr) of 1790 mm (71 in) and a fastener spacing (Fs) of 305 mm (12 in.) along the seam. For these assemblies, # 15 and #21 fasteners were used with 57- mil (0.057 in.) thick polymer batten bar having a width of 25 mm (1 in).

For Set 2, thermoset assemblies, Ethylene Propylene Diene Monomer (EPDM) membrane was used. Being a rubber polymer, EPDM cannot be hot air welded, therefore, its application follows the method of "In-seam attachment". As shown in Figure 4, the membrane seam is mechanically fastened to the deck using 50 mm (2 in.) diameter plastic plates and 83 mm (3 1/4 in.) long #14 fasteners, and the overlap seams were primed with adhesive and sealed by using seam tape to make a symmetrical joint. The overall membrane assembly configuration consisted of three seams with two full sheet widths having a fastener row spacing (Fr) of 1980 mm (78 in.) and fastener spacing (Fs) of 300 mm (12 in).

In Set 3, mechanically attached modified bituminous roof assemblies, the waterproof membrane typically consists of a minimum of two layers: the base sheet and the cap sheet. Both the base and cap sheets come in width of 990 mm (39 in.). The base sheet is installed as the first layer over the insulation. It is affixed to the steel deck along the seam using 83 mm (3 1/4 in.) long # 14 fasteners and 50 mm (2 in.) diameter metal plates with Fs of 150 mm (6 in.). The seam's overlaps of the base sheet are then torched as shown in Figure 5. With the base sheet installed, the cap sheet is installed by torching it onto the base sheet. The overall membrane layout comprised 5 full sheets with six seams having fastener row spacing (Fr) of 900 mm (35.6 in.)

In figure 4, P, and L respectively represent the pressure and force measurement locations.

3. Results and Discussion

The strength of the fastener-deck interface is characterized by the physical and mechanical properties of the deck and fasteners. Through this experimental testing, the present study identified three parameters that characterize the strength of the

fastener-deck interface and have an influence on the wind uplift resistance of MARS. The results are quantified based on these three investigative parameters and are discussed below:

Effect of Deck Gauge

Thermoplastic assemblies $TP_{22-80-15}$ and $TP_{20-80-15}$ represent the two assemblies which had variation in the deck gauge. $TP_{22-80-15}$ used gauge 22 steel deck, while $TP_{20-80-15}$ used gauge 20. Figure 5 shows the typical pressure measured and load time histories of $TP_{22-80-15}$. The measured wind uplift resistance data indicates that $TP_{22-80-15}$ passed the first four levels (90, 113, 135 and 157 psf) of the CSA load cycle. Since it failed at Level E sequence 4, it obtained a wind uplift rating of 7.5 kPa (157 psf). In the case of $TP_{20-80-15}$, the wind uplift rating is 8.6 kPa (180 psf), as it successfully sustained all five levels (90, 113, 135, 157 psf and 180 psf) without any failure of the assembly. The test had to be stopped as it reached the maximum pressure level of the CSA dynamic cycle. The measured fastener loads of $TP_{22-80-15}$ and $TP_{20-80-15}$ at their sustained pressures were 2.8 kN (640 lbf) and 3.1 kN (706 lbf) respectively.

With all similar components, except the steel deck gauge, $TP_{20-80-15}$ showed a 13% increase in wind uplift resistance compared to $TP_{22-80-15}$, and this can be explained or related to the strength of the fastener-deck interface. $TP_{22-80-15}$ and $TP_{20-80-15}$ with # 15 fastener have a FPR of 3 kN (685 lbf) and 3.9 kN (893 lbf) respectively. The FPR comparison indicates 23% increase with the increase in the gauge of the deck. During wind uplift testing, this higher fastener-deck attachment strength of $TP_{20-80-15}$ offered higher pullout resistance to the membrane-induced load, which allowed the assembly to sustain higher wind uplift pressures compared to $TP_{22-80-15}$. Observing the failure modes as shown in Figure 5 can also draw similar inference. In $TP_{22-80-15}$, the membrane load exceeded the resistance offered by 22 Ga deck, which allowed the fasteners to pull out from the deck. Whereas in $TP_{20-80-15}$, the capability of 20 Ga deck to resist high pullout loads allowed it to sustain higher uplift pressures without any sign of failure.

Effect of Fastener Type

Using all the components in $TP_{22-80-15}$, including the gauge and yield strength of deck, the # 15 fastener in $TP_{22-80-15}$ was replaced by #21 for the assembly $TP_{22-80-21}$. The measured pressure and load time history of $TP_{22-80-21}$ indicated that it sustained a maximum wind uplift pressure of 8.4 kPa (175 psf) with a corresponding fastener load of 2.9 kN (672 lbf).

Fastener # 21 has a shank diameter of 8.4 mm (0.331 in.), which was almost 15% greater than the shank diameter of the # 15 fastener [7 mm (0.281 in.)] used in $TP_{22-80-15}$. The #21 fastener when connected to 22 Ga -80-ksi deck yielded a FPR of 5050 N (1148 lbf), which was 23% greater than the FPR of $TP_{22-80-15}$. Due to the larger diameter of the #21 fastener, it provided greater contact strength with the deck, which translated into higher FPR and higher wind uplift resistance. The failure mode, as shown in Figure 5, indicates membrane tear along the plastic batten bar in the seam. In this case, the fastener-deck combination was the strongest link, while the membrane was the weakest link.

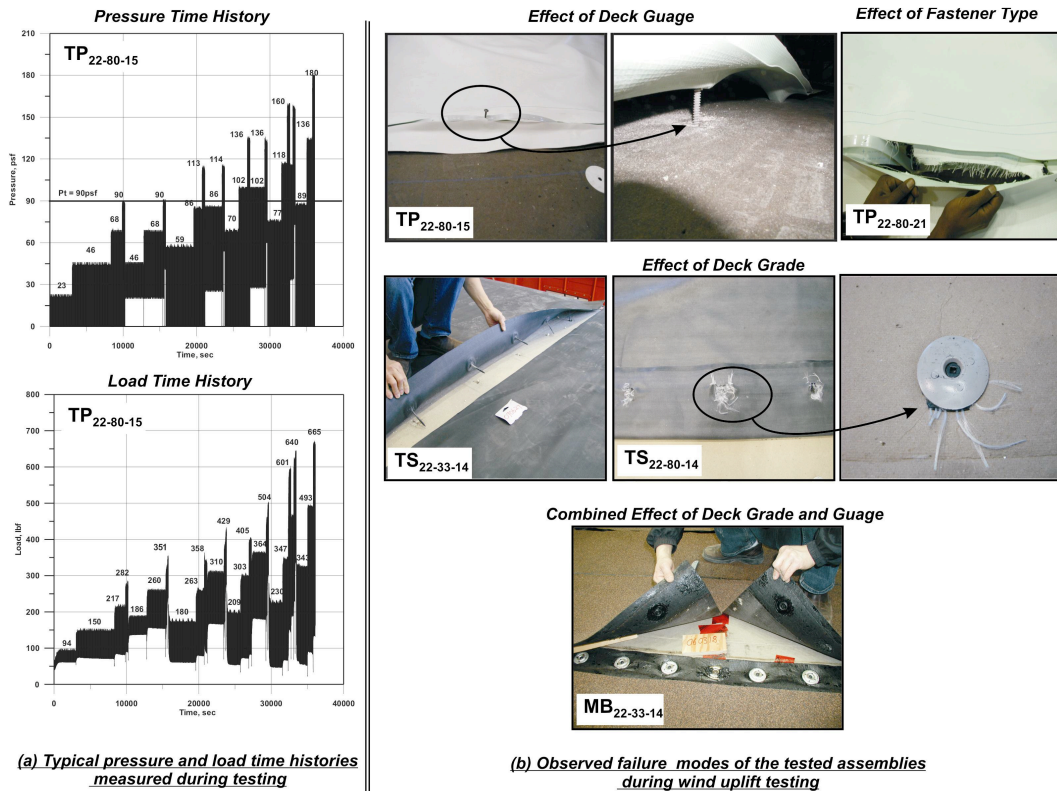


Figure 5: Results Obtained from the Wind Uplift Investigation

Evaluating the wind uplift performance of the above three assemblies ($TP_{22-80-15}$: 7.5 kPa; $TP_{20-80-15}$: 8.6 kPa; $TP_{22-80-15}$: 8.4 kPa) also provides the opportunity for assembly optimization. More discussion about assembly optimization can be found in Baskaran et al (2006). In order to improve the performance of $TP_{22-80-15}$, two options exist, one is to increase the thickness of the deck to 20 Ga as in $TP_{20-80-15}$, and the other is to use a larger diameter fastener (# 21) as in $TP_{22-80-21}$. As discussed above, both options could contribute to improving the wind uplift performance of $TP_{22-80-15}$. However, considering a scenario of re-roofing, option 1 may cost the building owner more in comparison to option 2. This is due to the fact that replacing a deck tends to be more expensive in terms of both material and labour as compared to replacing fasteners.

Effect of Deck Grade

$TS_{22-33-14}$, and $TS_{22-80-14}$ of the thermoset assemblies represent the two assemblies, which had variation in the deck grades (33 vs.80). When subjected to dynamic wind load cycles, $TS_{22-33-14}$, sustained a wind uplift pressure of 4.3 kPa (90 psf) with a fastener load 1.5 kN(345 lbf), while $TS_{22-80-14}$ sustained a wind uplift rating of 5 kPa (105 psf) with corresponding fastener load of 1.3 kN (293 lbf). Correlating the wind uplift resistance with the strength of the fastener-deck interface indicates that $TS_{22-80-14}$ with FPR of 2.9 kN (660 lbf) sustained a 14% higher wind uplift resistance when compared to $TS_{22-33-14}$, which had FPR of 1.8 kN (420 lbf). This increase in FPR could be attributed to the higher yield strength of the 80-ksi deck., which during the

wind uplift testing also offered high resistance to the membrane induced fastener load.

The wind uplift failure modes of these assemblies as shown in Figure 5, indicate that $TS_{22-33-14}$, which had 33-ksi deck failed due to fastener pullout from the deck, whereas in $TS_{22-80-14}$, the membrane became the weakest link and failed due to membrane tear around the fastener plate. It can be concluded that the greater the yield strength of the deck the higher will be its fastener pullout resistance, and ultimately, it would have higher wind uplift resistance.

Combined Effect of Deck Grade and Thickness

Mod-bit assemblies, $MB_{22-33-14}$ and $MB_{20-80-14}$, represent the combination for variation in both deck grade and deck thickness (22 vs. 20 and 33 vs.80). With the similar fastener type, but variation in the deck properties, $MB_{22-33-14}$ sustained an uplift pressure of 4.3 kPa (90 psf) with fastener load of 0.6 kN (137 lbf), and $MB_{20-80-14}$ without any failure sustained an uplift pressure of 5.7 kPa (120 psf) with fastener load of 0.7 kN (178 lbf). Examination of the failure mode of $MB_{22-33-14}$ as shown Figure 5, indicated that the seam of the base sheet delaminated, which caused the premature failure of the roof assembly. In addition to this primary failure mode, it also had symptoms of secondary failure mode of fastener plates loosening with fasteners slightly pulled. Similar exercise of FPR correlation with the wind uplift performance indicates that $MB_{20-80-14}$, which had higher FPR of 2.9 kN (651 lbf) sustained 25% higher wind uplift rating compared to $MB_{22-33-14}$, which had a lower FPR of 1.7 kN (382 lbf). This increase in the wind uplift resistance is the result of the high pullout strength of 20 Ga-80 ksi deck, which is a combination of higher gauge and higher yield strength. Even though the fastener had a smaller shank diameter, when connected to the 20 Ga-80 ksi deck, it provided a strong connection strength, which enabled $MB_{20-80-14}$ to withstand high wind uplift pressures.

Figure 6 presents the results summarizing the research findings from the present investigative study. The plotted data has been generalized with respect to the three assembly sets. As shown in the figure, the data indicates that by increasing the fastener diameter, deck grade or deck gauge an increase of 10 to 15% in the wind uplift performance can be expected. Whereas, if a steel deck having higher gauge and grade is used an increase of 25% and beyond can be expected in the wind uplift performance of that roof assembly. As the present study did not investigate all the possible combinations of fastener-deck interfaces within each assembly set, this generalized data only provides the estimation of how much each parameter could influence the wind uplift performance of the roof assembly. However, data variation in the improvement of the wind uplift performance can be expected based on the assembly set being tested.

4. Conclusion

Fastener-deck interface strength constitutes an important parameter in the successful design of mechanically attached roof assemblies. To investigate its effect on the wind uplift resistance of MARS, seven mockups of mechanically attached assemblies with three different membrane types and having different fastener-deck combinations were tested using the dynamic wind load cycle. From the obtained results, three parameters

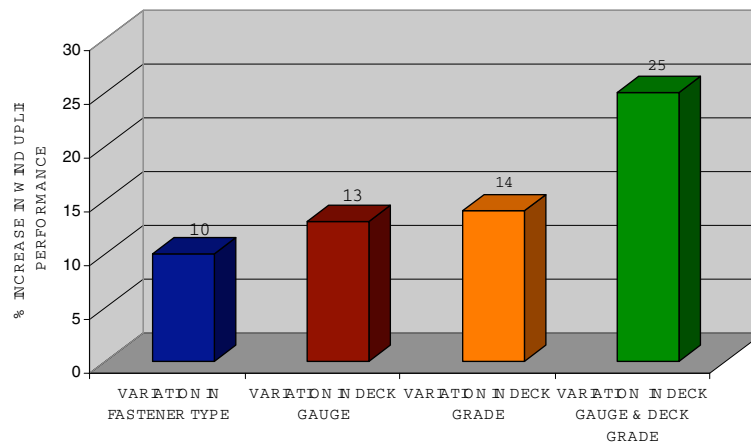


Figure 6: Effect of Fastener-Deck Interface Properties on the Wind Uplift Performance of MARS

characterizing the fastener deck interface strength namely, deck gauge, deck grade and fastener type were identified as the influencing parameters. Based on the presented results and discussions, the following conclusions can be drawn:

- Assemblies tested on higher gauge (20Ga) or higher yield strength decks (80 ksi) sustained higher wind uplift pressures compared to assemblies of lower gauge and lower yield strength decks.
- Assemblies tested with different fastener types showed that the larger shank diameter fasteners provided greater connecting strength, thus enabling the assembly to sustain higher wind uplift pressures.
- Within each assembly set, the FPR comparison indicated that the assembly with higher FPR sustained higher wind uplift ratings compared to the assembly with lower FPR. Therefore FPR could be used as a verification factor for system wind resistance estimation.

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Acknowledgements

The presented research is being carried out for a consortium - Special Interest Group for Dynamic Evaluation of Roofing Systems (SIGDERS). SIGDERS was formed from a group of partners who were interested in roofing design. These partners included:

<i>Manufacturers</i>	Atlas Roofing Corporation, Canadian General Tower Ltd., Carlisle Syn Tec., GAF Materials Corporation, GenFlex Roofing Systems, Firestone Building Products Co., IKO Industries Ltd., ITW Buildex, Johns Manville, Sika-Sarnafil, Soprema Canada, Stevens Roofing, Tremco and Trufast
<i>Building Owners</i>	Canada Post Corporation, Public Works and Government Services Canada.
<i>Industry Associations</i>	Canadian Roofing Contractors' Association, Canadian Sheet Steel Building Institute, National Roofing Contractors' Association and Roof Consultants Institute.

Numerical Modeling of Adhesive Applied Roofing Systems for Wind Uplift Resistance Evaluation - A Pilot Study

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Abstract

Adhesive Applied Roofing Systems (AARS) are compact roof systems that use cold adhesives to integrate their components. Differences in adhesive types and method of applications can influence the wind uplift performances of AARS. Full scale experimental studies have been conducted and it revealed that majority of failures occurred at the insulation level and its interface. Based on this observation, a simplified three-dimensional finite element (3D-FE) model is developed to assess the uplift resistance of AARS. However, at this stage, the modeling simulation does not incorporate the studies due to the independency effects (e.g. # of meshes, # of elements and type). The model is assumed as isotropic elastic materials consisting of three parts (insulation, adhesive and insulation). To benchmark the model, new testing specimens were fabricated and tested. The results showed good agreement in comparison with the model. Using the benchmarked model, the effects of the adhesive thickness and its methods of application were investigated. Overall stresses distributions were computed at each level of the models as an indicator of AARS performances. This paper presents the results from this ongoing numerical study.

Keywords: Roof, AARS, 3D-FE Model, Failure, Adhesive, Insulation, Wind uplift and Building

Introduction

Roof is a key component of the building envelope that provides an environmental separation between the indoor and outdoor environmental conditions thereby protect occupants from directly exposes to wind, sun, temperature, snow, rain, precipitation and other hazards. Adhesive Applied Roofing Systems (AARS) represent a new generation of Built-Up Roofing Systems (BURS) that utilize adhesives to integrate its components together (e.g. deck, vapour barrier, insulation board, cover board, membrane). Due to this integration technique, AARS can reduce common roof problems that would be related to moisture transports, air intrusions (Hens et al.

1995), thermal bridging and fastener penetrations (Phalen 1993). Presently the continuous development in roofing technology and products permit roofing manufactures to continually develop different types of AARS. The introduction of new components and application techniques could result in different levels of uplift performances. Consequently, when different components are used, the method of applications and the performances obtained would be different. Therefore these phenomena will vary AARS performances in term of their wind uplift resistances. Figure 1 illustrates typical roofing components used to form AARS.

Figure 1 AARS Typical Isometric Diagram.

Although AARS have been commercially in use in the North American roofing market, there is no existing test standard to quantify their wind uplift resistance performance. Recent investigation from the hurricane Charley identified major AARS's wind uplift resistance failures (Graham, 2004). To study common AARS wind uplift resistance performances, a series of experimental investigation has been conducted and showed that most failures occurred at the insulation level, either at the insulation itself or/and at adhesive interface. Further information about the experimental investigation results (mock-up fabrication, testing protocols, failure mode and criteria, static and dynamic uplift rating performances) can be found in Baskaran et al 2007. Based on failure modes obtained from experimental investigation, a simplified numerical analysis model using finite element (FE) modelling technique aided by commercial ABAQUS software was developed. The model developed is considered as an early stage to further study AARS performance using a simplified FE model simulation. The results obtained from the modelling simulation are expected to help AARS roof designers to better understand AARS wind uplift performances in terms of the effect of various elements such as adhesive method of application, adhesive thickness, temperature, roof size and the present of joint insulation in the components. However, the present research only focuses on a development of the simplified model and its validation by using benchmarked experimental data. In addition, the benchmarked model was enhanced to investigate the effects of adhesive method of application and its thickness. This paper presents and discusses the results from these numerical studies.

Numerical Model

Model limitation

The three dimensional finite element (3D-FE) model presented herein will work only with the assumption if the AARS weakest link happen to be either at the insulation itself or and at its adhesive interface as documented by Bas et al. 2007 and Current et al. 2008. The numerical analysis performed in the model is based on three dimensional linear elastic analysis simulations.

Modeling Detail

The model consists of three parts namely, Bottom Insulation, Adhesive and Top Insulation. All parts were modeled as three dimensional eight nodes continuum elements (C3D8I) having the same length and width, 305 mm x 305 mm (12 in x 12 in), but different thicknesses, 51 mm (2 in) , 2 mm (0.08 in) for the insulation and the

adhesive respectively. Figure 2 illustrates the detail of the model and the mechanical properties used for the FE simulation. The adhesive and insulation mechanical properties were extracted from Henry 2006 and Baskaran & Borujerdi 2001 respectively. The bottom insulation was fixed with no movement allowed (Encastre) and uniform pressure loads were applied in four steps at the top insulation surface while, the other edges of the three parts were released as shown in Figure 2. The adhesive and insulation parts were modeled as homogenous isotropic elastic material with the adhesive having 100 % contact with the two insulation surfaces at the interface. To simulate these aspects, tie constraints were used to establish connections between the three parts. The model has in total 108 numbers of elements and 654 nodes. The simulation was performed under three-dimensional linear elastic analysis using a static stress procedure provided in ABAQUS software. The AARS uplift performances are indicated in term of their stresses correspondence to four different steps load input.

Figure 2 Modeling Detail and Properties

Stress Concepts

Stresses in ABAQUS are measured in term of Cauchy (true) stresses. Cauchy's stress principle demonstrates that the state of stress at a point in a body is completely defined by the nine components of a second-order Cartesian tensor called the Cauchy stress tensor, given by

$$\sigma_{ij} = \begin{bmatrix} \mathbf{T}^{(e_1)} \\ \mathbf{T}^{(e_2)} \\ \mathbf{T}^{(e_3)} \end{bmatrix} = \begin{bmatrix} \sigma_{11} & \sigma_{12} & \sigma_{13} \\ \sigma_{21} & \sigma_{22} & \sigma_{23} \\ \sigma_{31} & \sigma_{32} & \sigma_{33} \end{bmatrix} \equiv \begin{bmatrix} \sigma_{xx} & \sigma_{xy} & \sigma_{xz} \\ \sigma_{yx} & \sigma_{yy} & \sigma_{yz} \\ \sigma_{zx} & \sigma_{zy} & \sigma_{zz} \end{bmatrix} \equiv \begin{bmatrix} \sigma_x & \tau_{xy} & \tau_{xz} \\ \tau_{yx} & \sigma_y & \tau_{yz} \\ \tau_{zx} & \tau_{zy} & \sigma_z \end{bmatrix}$$

Where, $\mathbf{T}^{(e_1)}$, $\mathbf{T}^{(e_2)}$, and $\mathbf{T}^{(e_3)}$ are the stress vectors associated with the planes perpendicular to the coordinate axis, σ_{11} ($\sigma_{xx}=\sigma_x$), σ_{22} ($\sigma_{yy}=\sigma_y$), and σ_{33} ($\sigma_{zz}=\sigma_z$) are normal stresses, and σ_{12} ($\sigma_{xy}=\tau_{xy}$), σ_{13} ($\sigma_{xz}=\tau_{xz}$), σ_{21} ($\sigma_{yx}=\tau_{xy}$), σ_{23} ($\sigma_{yz}=\tau_{yz}$), σ_{31} ($\sigma_{zx}=\tau_{zx}$), and σ_{32} ($\sigma_{zy}=\tau_{zy}$) are shear stresses.

The first index i (σ_{ij}) indicates the stress acts on a plane normal to the x_i axis, and the second index j (σ_{ij}) denotes the direction in which the stress acts. A stress component is positive if it acts in the positive direction of the coordinate axes, and if the plane where it acts has an outward normal vector pointing in the positive coordinates direction. For the case where cross-sectional area is assumed constant during deformation under certain loads, the stress is called engineering stress or nominal stress. ABAQUS stress output considers that the change in cross-sectional area are significant, therefore cross-sectional area at the deformation are used instead of the initial cross-sectional area. The true stresses in Model is calculated as follow,

$$\sigma_{true} = (1 + \epsilon_e)(\sigma_e)$$

Where, ϵ_e is the nominal (engineering) strain, and σ_e is nominal (engineering) stress.

Benchmarked Experiment

A series of experiment have been conducted to benchmark the model. The test specimens consist of three parts (Insulation, Adhesive and Insulation) that have the same size with the model simulation (Fig. 2). Top and bottom insulations were 100% connected by using adhesive at the interface. In total six specimens were fabricated and cured for 28 days prior to testing. As shown on the right side of Figure 2, two plywood dummies were attached to the top and bottom insulation parts using adhesive. The dummies later will be used to attach the test specimens to the testing machine. The test was conducted using an Instron testing machine model 5566 and the specimens were loaded in tension at a rate of 5 mm/min. Figure 3 shows the test set-up used for the testing in comparison with the model simulation boundary condition.

Figure 3 Typical Experimental Setup

Model Validation

This section discusses the validation of the 3D-FE model developed with the benchmark experimental data. The validation of the model is indicated in term of stresses. The stresses obtained from the model were 2.33E-3 Mpa, 6.22E-3 Mpa, 7.94E-3 Mpa, 1.12E-2 Mpa for the input loads of 138 N, 372 N, 470 N and 670 N respectively. The indicated stresses were then compared to the stresses obtained from the benchmark experiment. The maximum stress for the benchmark experiment was calculated by dividing the average maximum load (After averaging maximum loads from the six specimens tested) with the surface plane area of the specimen (7.2E-3 Mpa). To plot a linear “Stress vs Load” curve for the benchmark experiment, three data points (three average loads) were randomly selected from the average test data (before the maximum load achieved). Adopting the same procedure for calculating the average maximum stress, three other stresses (1.48E-3 Mpa, 4E-3 Mpa and 5.06E-3 Mpa), were calculated correspondence to the three data points selected. Comparison of stresses between the model and the benchmarked experiment are presented in Figure 4. Result shows that the model was able to simulate the test data performance (Figure 4) as well as failure mode (Figure 5). Typical model stress distribution contour diagram indicated in Figure 5 (left side) shows that maximum stresses are concentrated at the corners of insulation part while at the test, failure mode happen at the corner of insulation facer (Right side). This aspect illustrates that the model predict well the test failure performances. However, model was overestimated maximum capacity of the test data as result of stress distribution complexities at the interface of test specimen that depends mainly on adhesive application and its performance. For example, during the failure mode investigation, it was noticed that the adhesive surface at the interface did not have the same thickness and the presence of adhesive bubbles were observed. These phenomena will definitely reduce the performance capacity of the test specimens due to the fact that there were not 100 % covered by adhesives, despite the precautions have been taken during test specimen

preparations. Nevertheless, in this study, the authors are not interested in predicting the capacity at the maximum load but rather the study focuses investigate the *trend* performance of the specimen at the elastic stage. For this reason, the model was enhanced and used to further study the effect of adhesive method of application and thickness at 50% of its maximum capacity performance. It should be noted that the current modeling analysis does not consider the independency effects such as number of meshes, number of element and type. However, these independency effects will later be addressed on the next stage of the research.

Figure 4 Stresses Performances - Model vs Test

Figure 5 Failure Phenomena - Model vs Test

Effect of Adhesive Method of Application

In field, two methods of adhesive application (fully coated (100% adhesive coverage) method and ribbon (beading) method) are used. The selection on using either one of the methods depends on the manufacturer's specifications. Regardless which method is being used, it can contribute to different stress performance and behavior of AARS due to reduction in the adhesive contact area. In addition to the method of application, adhesive mechanical and physical properties vary from one roof manufacturer to another. This aspect also affects the AARS performance. Hence, a study to investigate the effect of the adhesive application methods is needed. One ways to accomplish this would be by using FE model simulation.

Based on the previously developed FE model (100% adhesive coverage), three other models were created to investigate the effect of common method of adhesive applications in term of one, two and three beading applications that represents 21%, 42% and 62% areas reduction respectively. The three models utilized the same boundary condition applicable to the previous model as well as the size of the insulation parts and element type (C3D8I) for the simulation. The change in the three models was made in terms of adhesive (bead) width because of beading application. The beading width of 63.5 mm (2.5 in) was used instead of 305 mm (12 in) while length and thickness remained the same. Figure 6a illustrates un-deformed geometry diagrams of the three models that show different beading applications. Three-dimensional finite element linear elastic analyses were conducted for the four models (the new three models + previous model). Each analysis consists of four loading steps with the pressure input on each step being 50% lower in comparison with the previous model simulation. This condition was taken in order to ensure that the obtained stress performance remains in an elastic range.

Figure 6b shows the system ability performance due to the effects of four different types of adhesive application (100%-Fully Coated, 62% area covered-3 Beads, 42% area covered- 2 Beads and 21% area covered – 1 Bead). The stresses output results from the three beading application models were normalized with the fully coated adhesive application model. These normalized stresses are plotted in Y axis of Figure 6b while the X axis illustrates the performance in term of each load increment inputs. The figure shows that the system ability performance will be nonlinearly reduced as

the adhesive contact area is reduced. The difference of system ability performance reduction between one bead application and two beads application is roughly about 30 to 40%. The system performances of the three bead application decreased significantly (about 4 to 5 times) compared to one or two beads application. These situations occurred because of the stress distribution phenomena that were caused by system response due to different placement of adhesive application. Figure 6c illustrates the stress distribution contour diagrams for the models at the 335 N input loads. Figure 6d presents the average normalized stress from four different input loads. This curve can be used to predict approximately the reduction in performance due to the difference of adhesive application methods if two extreme cases are known, fully coated and one bead application.

Figure 6. a) Undeformed Geometry Diagrams, b) Normalized Stress vs Input Load, c) Stresses Contour Diagrams for the Three Beading Applications and d) Average Normalized Stress vs % Surface Area Covered.

Effect of Adhesive Thickness

Finding the effect of adhesive thickness is crucial to provide clear understanding of its impact on uplift system capacity performances. To achieve this, new models were created based on the fully coated (100% adhesive coverage) FE model. All variables, such as boundary condition, type of analysis and element used, remained the same as before. Modifications were made only with respect to the adhesive thicknesses. Nine different thicknesses (0.5 mm, 1 mm, 2 mm, 3 mm, 4 mm, 5 mm, 10 mm, 20 mm and 30 mm) were selected and used in the model simulations. Stress output results are presented in the Figure 7. Results indicate that there are no significant differences in stress level between the 0.5 mm to 5 mm adhesive thicknesses. A significant difference in the level of stress was noticed in the case of adhesive thicknesses ranging between 5 mm and 30 mm. The numerical study suggests that adhesive thickness used in field application should not be more than 5 mm because of adhesive rigidity decreases when larger thickness is used. This actually reflecting the fact that field adhesive thickness is normally within this range. It must be noted that in relation to the interpretation of these test results that this analysis was only performed using one type of adhesive. As the fact that Young Modulus (E) and Poisson ratio (ν) can be different between one manufacture to another, this will provide variation in term of nominal stress value obtained from the model simulation. Therefore it is important to emphasize that the study here does not attempt to evaluate nominal stress value of the specific adhesive properties rather than how it affects the overall uplift capacity performance when different adhesive thicknesses are used by roofing manufactures in field application.

Figure 7. Adhesive Thickness vs Stress

Conclusions

This ongoing numerical analysis study provides some useful information that later can be used to further numerically analyse AARS uplift capacity performances. There are several conclusions that can be drawn from this study as follows,

1. A preliminary simplified three dimensional finite element model was established and verified using benchmarked experiment. It is proven that the simplified model can be used to predict AARS uplift performances if the weakest link of AARS lies on insulation and its adhesive interface. At a later stage, another study for the model will be performed to account for dependency effects due to number of meshes, number of element and type.
2. The effect of adhesive method of application have been illustrated by using the enhancement of the simplified FE model and a curve (Fig. 6d) to approximately predict the AARS uplift performances in term of its stress has also been developed. However, in the future study this curve will need to be verified in term of other effects such as surface temperature, roof size and existence of joints in insulation components.
3. The effect of adhesive thickness on AARS uplift performances has also been studied and it is proposed that the thickness used at the field application should not be more than 5 mm.

Acknowledgement

The authors acknowledge financial support provided by the Natural Sciences and Engineering Research Council (NSERC) project under Grant # CDR 305819, Roofing manufacturers: *BAKOR Inc.*, *IKO Industries Ltd*, *SOPREMA Inc.*, *TREMCO Inc.* and Roofing Contractor Association of British Columbia (RCABC).

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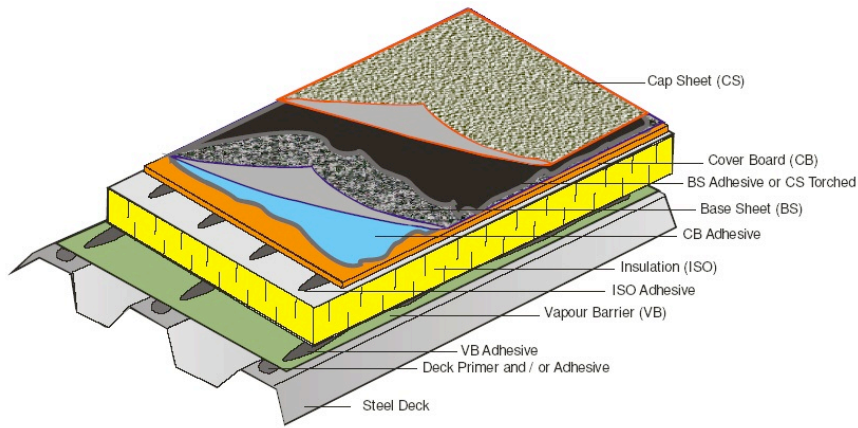


Figure 1. AARS Typical Isometric Diagram

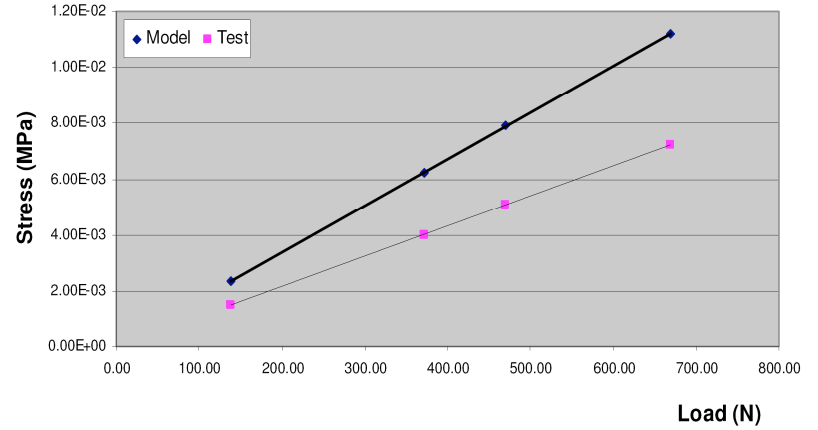


Figure 4. Stresses Performances – Model vs Exp.

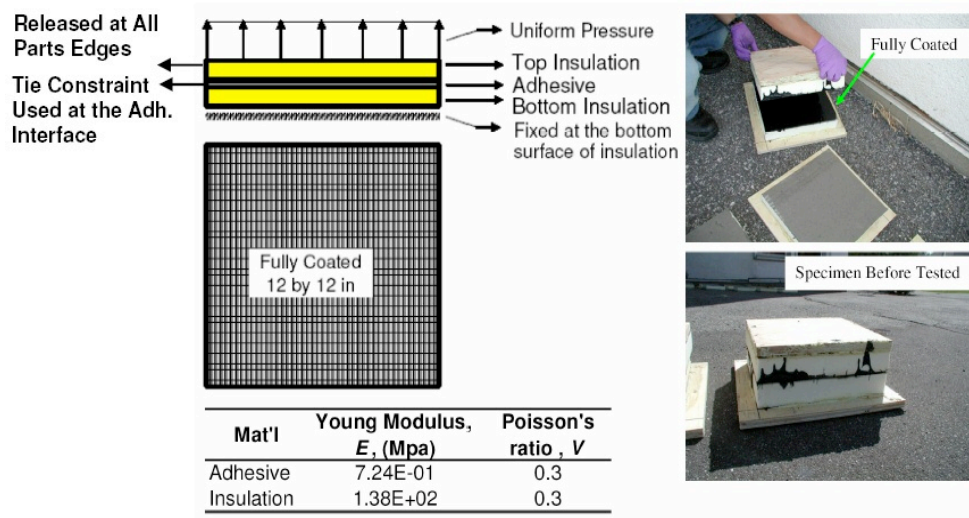
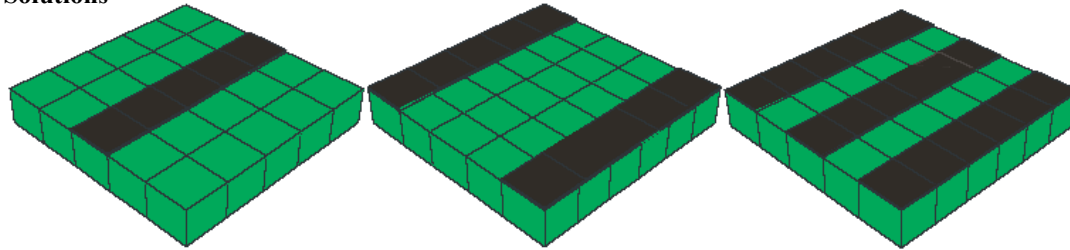


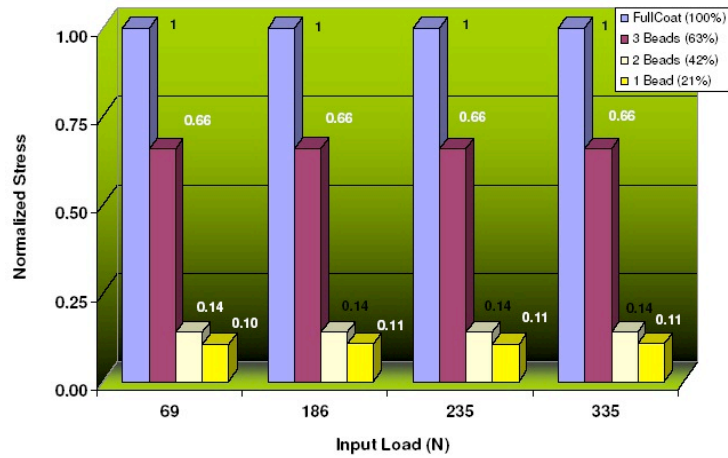
Figure 2. Modelling Detail and Properties



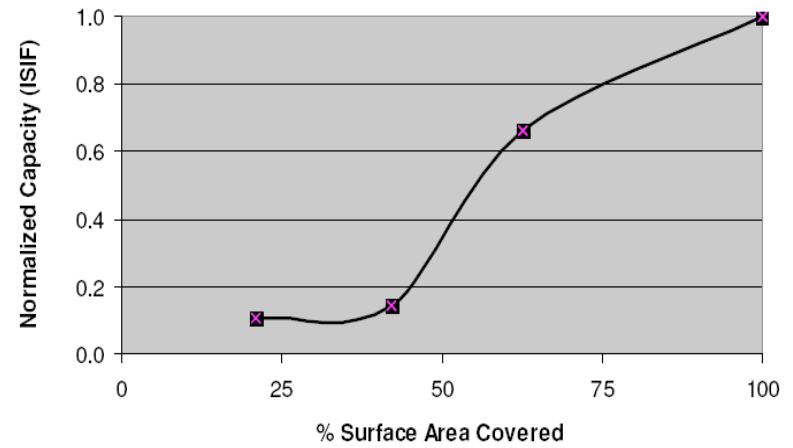
Figure 3. Typical Experimental Setup



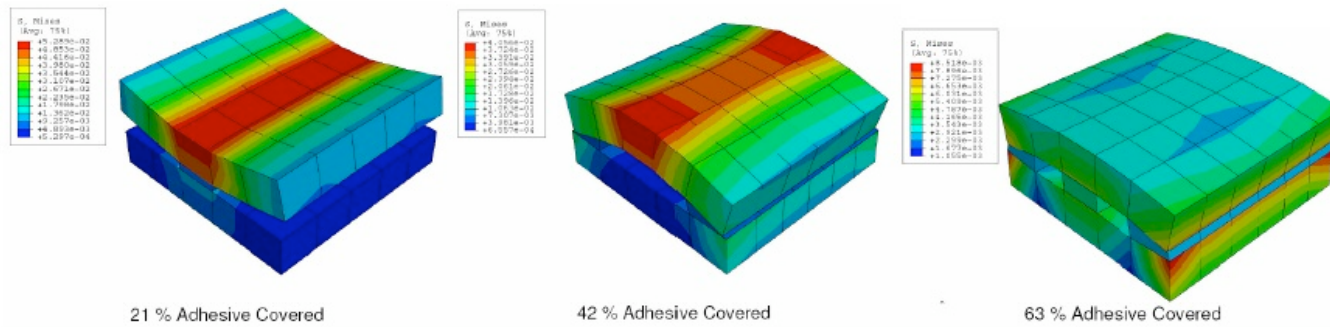
(6a)



(6b)



(6d)



21 % Adhesive Covered

42 % Adhesive Covered

63 % Adhesive Covered

(6c)

Figure 6. a) Undeformed Geometry Diagrams, b) Normalized Stress vs Input Load, c) Stresses Contour Diagrams for the Three Beading Applications and d) Average Normalized Stress vs % Surface Area Covered.

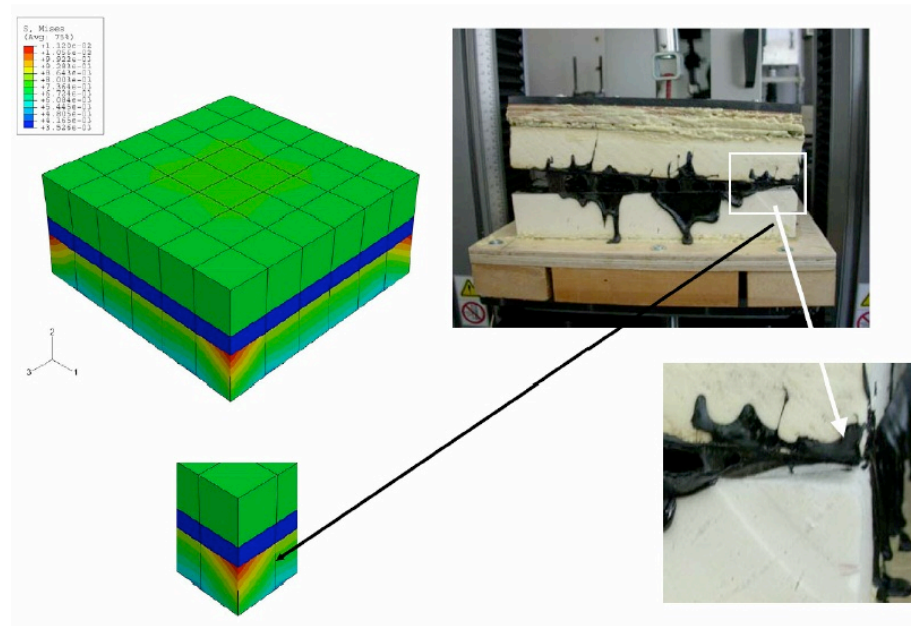


Figure 5. Failure Phenomena – Model vs Exp.

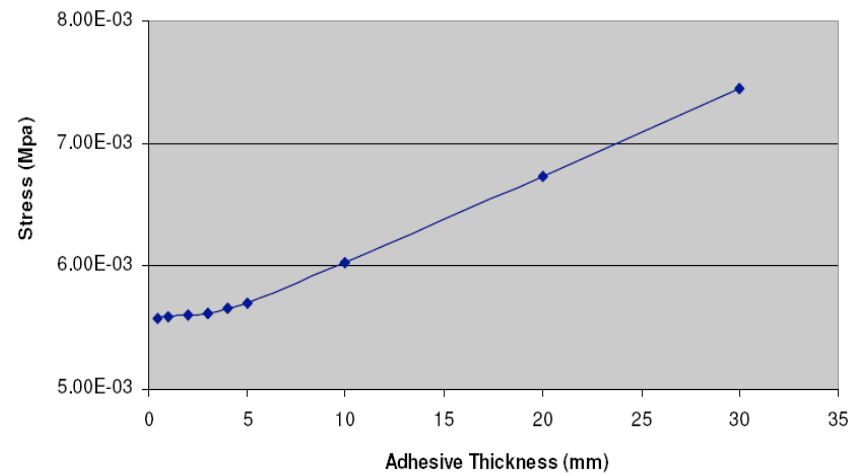


Figure 7. Adhesive Thickness vs Stress (at 669 N Input Load)

Practical Design Guidelines for Steel Diagrid Structures

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Abstract: Diagrid structures have been prevalently used for tall buildings worldwide due to their structural efficiency and aesthetic potential. This paper provides design guidelines for diagrid structures for tall buildings. Optimal grid geometry is discussed for diagrids of various configurations. A stiffness-based design methodology for determining preliminary member sizes for the diagonals is presented. Guidelines are presented for determination of optimal bending and shear deformations depending on grid geometries and height-to-width aspect ratios of diagrid structures.

Introduction

Diagrid structures have emerged as a new design trend for tall buildings. The recognition of structural efficiency of diagonals and use of them combined with conventional orthogonal structural members date back to the time of very early tall building developments in the late nineteenth century. However, the emergence of the current form of diagrid structures typically without vertical columns at the building perimeters is very recent phenomenon. Examples include the Swiss Re Building in London, the Hearst Headquarters in New York (Figure 1), both by Sir Norman Foster, the Guangzhou Twin Towers in Guangzhou, China by Wilkinson Eyre, and the Lotte Super Tower in Seoul, Korea (Figure 2) by Skidmore, Owings and Merrill.

While design methodologies and the structural performance of other tall building structural systems have been investigated by many researchers and engineers, those for the relatively new diagrid structural system has not been extensively studied and presented. This paper provides design guidelines for diagrid structural systems to assist engineers and architects in the early stage of design. Optimal grid geometry of diagrid structures is discussed first. In terms of diagrid angle configurations, contemporary diagrid structural systems can be categorized as those with uniform angles, as is the case with the Hearst Headquarters, and with varying angles, as is the case with the Lotte Super Tower. Optimal uniform angles as well as optimal configuration of varying angles for diagrid structures are discussed.

A stiffness-based design methodology for determining preliminary sizes for the diagonals is introduced and applied to a representative set of steel buildings of various heights. Guidelines are presented for determination of bending and shear deformations in conjunction with grid geometries for optimal design of diagrid structures depending on their heights and height-to-width aspect ratios. With these design guidelines, structural and architectural decisions at the early stage of tall building design employing a diagrid structural system can be made in a more integrative and efficient way. Further, the optimal design strategies discussed in this paper will contribute to constructing built environments using a minimum amount of resources.



Figure 1: Hearst Headquarters
(Courtesy of Adam Gimpert)



Figure 2: Lotte Super Tower
(Courtesy of SOM, Chicago)

Optimal Grid Geometry of Diagrid Structures

The optimal angle for diagonal members for the maximum shear rigidity can be estimated using the simple braced frame model shown in Figure 3 by assuming that members carry only axial forces. Then, shear rigidity D_T can be expressed as

$$D_T = A_d E_d \sin 2\theta \cos \theta \quad (A_d : \text{Area of Diagonal}) \tag{1}$$

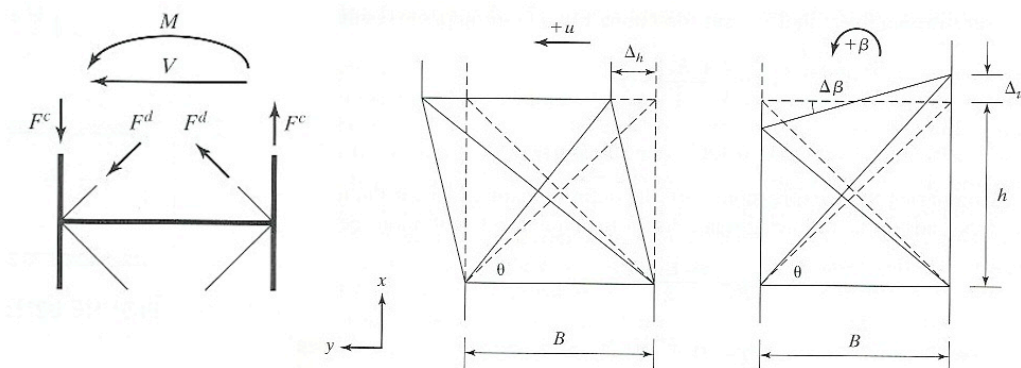


Figure 3: Braced Frame Model

The maximum value of $\sin 2\theta \cos \theta$ occurs when the angle is about 35 degrees, indicating the optimal angle for maximum shear rigidity of the system. In typical braced frames, the bending moment is carried by the axial forces in the vertical columns.

However, for diagrid structures which do not have vertical columns, bending is carried by the axial forces in the diagonals. Since the optimal angle of the columns for maximum bending rigidity is 90 degrees and that of the diagonals for maximum shear rigidity is about 35 degrees, it is expected that the optimal angle of the diagonal members of diagrid structures will fall between these angles. Short buildings of low height to width aspect ratios behave more like shear beams, and tall buildings of high aspect ratios tend to behave more like bending beams. Thus, it is expected that as the building height increases, the optimal angle also increases (Moon et al. 2007).

Diagrids of Uniform Angle

In order to find the actual range of optimal angles, sets of 40, 50, 60, 70, and 80 story diagrid structures, with height to width aspect ratios ranging from 4.3 to 8.7 and having various diagrid angles (Figure 4), are optimally designed to meet the typical displacement design parameter of a five hundredth of the building height.

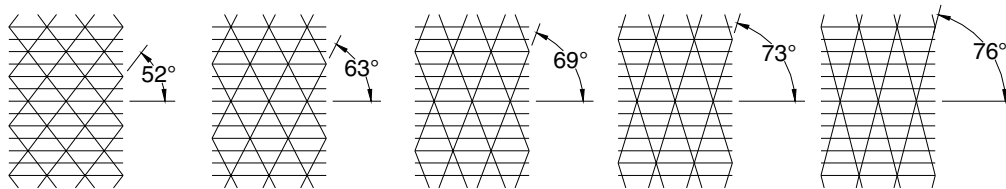


Figure 4: Diagrids of Varying Angles

The building’s typical plan dimensions are 36 x 36 meters with an 18 x 18-meter gravity core at the center and typical story heights of 3.9 meters. The document, SEI/ASCE 7-05 (Minimum Design Loads for Buildings and Other Structures) is used to establish the wind load. The buildings are assumed to be in Chicago and within category III, which implies a substantial hazard to human life in the event of failure. Based on the code, the basic wind speed is 40.2 meters per second (90 miles per hour). One percent damping is assumed for the calculation of the gust effect factor.

For 40 and 50 story diagrid structures, it was found that the buildings with an angle of 63 degrees meet the stiffness design parameter with the minimum amount of diagonal material. For 60, 70, and 80 story diagrid structures, the buildings with an angle of 69 degrees meet the design parameter most efficiently with the minimum amount of diagonal material. The influence of changing angles for 60 story diagrids is shown in Table 1. As can be seen in the table, at near optimal cases, the efficiency is not too sensitive to the change of angles. However, when the diagrid angle deviates substantially from the optimal, the efficiency of the system is drastically reduced.

Table 1: Influence of Changing Angles for 60 Story Diagrids

Story Module	Diagrid Angle	Diagrid Steel Mass (Ton)	Percentile Difference
2 stories	52 degrees	5700	+50.0%
3 stories	63 degrees	3930	+3.4%
4 stories	69 degrees	3800	Near Optimal
5 stories	73 degrees	4200	+5.3%
6 stories	76 degrees	4960	+30.5%

This study thus far is based on diagrid structures without vertical corner columns. The optimal angles of diagrids with corner columns are shallower than what is discussed here by about five degrees. However, diagrid structures without corner columns create valuable columnless corner spaces, which provide better views as well as higher rents. Thus, it is expected that diagrids without corner columns are preferred by architects as is the case with the Hearst Building.

Diagrids of Varying Angles

Incremental rates of shear forces and bending moments toward the base of a tall building are different. While shear forces increase almost linearly, bending moments increase drastically toward the base of the building. Thus, in a properly designed diagrid structure, the design of the upper portion of the building is governed by shear, and the lower portion is governed by bending. Considering this fact, it can be presumed that diagrid structures with gradually stiffer angles towards the base will have potential for structural efficiency. The structural efficiency of varying angle diagrids was investigated through designing varying angle diagrid structures and comparing them with uniform angle diagrids presented earlier. For completeness of the study, the case in which gradually changing angles of diagrids become steeper toward the top is studied as well. Varying angle as well as uniform angle diagrid configurations for 60 story diagrids is shown in Figure 5.

It was found that the optimal uniform diagrid angle produces more economical design in terms of material usage than the gradually changing diagrid angles for diagrid structures, 40, 50, and 60 stories tall, with height-to-width aspect ratios ranging from 4.3 to 6.5, as is the case with the 48 story Hearst Building in New York. In the varying angle design, with angles varying from 63 degrees at the top to 73 degrees at the base of the building, in the 60 story structure for example, the structure is configured to better resist bending at the bottom and shear at the top. Thus, in terms of bending rigidity, the varying angle design is a more economical angle composition than the diagrid of uniform angle. However, the negative effect of the reduced shear rigidity caused by the steeper

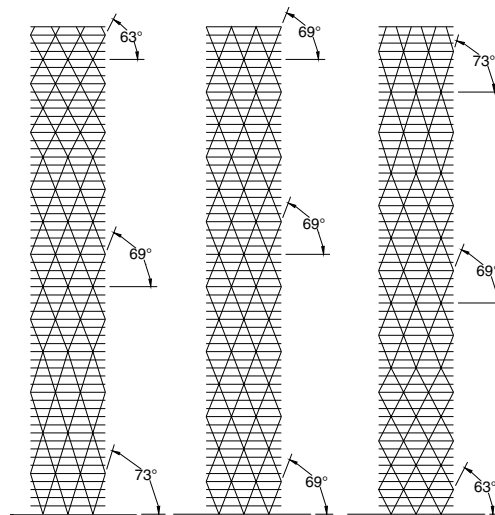


Figure 5: 60 Story Uniform vs. Varying Angle Diagrids

angle at lower levels of the structure is greater than the positive effect of the increased bending rigidity up to the 60 story diagrid structures studied here. (Since the member size becomes drastically larger towards the base of the building, member sizes at lower levels govern overall structural efficiency.) This negative effect of reduced shear rigidity becomes even greater in shorter structures because shorter buildings tend to behave more like shear beams.

However, this is no longer true for the 70 and 80 story diagrid structures in this study and those even taller than 80 stories. For the diagrid structures, with height-to-width aspect ratios bigger than about 7, gradually changing diagrid angles with the uniform optimal angle as a median angle value produces more economical design in terms of material usage than the uniform angle design cases, as is the case with the design of the 112 story Lotte Super Tower in Seoul. Steeper angle configuration towards the base of the structure in the 80 story diagrids, for example, favorably affects the bending rigidity and adversely affects the shear rigidity. However, unlike the 60 story case, the positive effect of the increased bending rigidity caused by the steeper angle at lower levels of the structure is greater than the negative effect of the reduced shear rigidity in the 80 story structure. This positive effect of increased bending rigidity becomes even greater in taller structures because taller buildings tend to behave more like bending beams. Therefore, diagrids of varying angles meet the same stiffness design criteria more economically than those of uniform angles for very tall diagrid structures with aspect ratios bigger than about 7.

As was expected, the design in which the gradually changing angles of diagrids become steeper toward the top produces the least efficient system among the alternatives. A summary of these study results is shown in Table 2 for the 40, 60 and 80 story diagrid structures.

Table 2: Efficiency of Uniform and Varying Angle Diagrids

Diagrid Height	Height/Width	Angles (Bottom → Top)	Steel Mass (Ton)
40 Stories	4.3	52, 63, 69	1906
		63	883
		69, 63, 52	1009
60 Stories	6.5	63, 69, 73	4482
		69	3820
		73, 69, 63	4104
80 Stories	8.7	63, 69, 73	16627
		69	15611
		73, 69, 63	11574

Stiffness-Based Design Methodology for Steel Diagrid Structures

Two most important design requirements for any building structural design are strength and stiffness, and for very tall buildings with a large height-to-width aspect ratio, the stiffness constraint generally governs the design. With the development of higher strength structural materials, stiffness of tall and slender structures has become an even more serious design issue. For example, the strength of structural steel has increased with almost no change in its modulus of elasticity. Thus, the strength requirements can

be met with smaller members, resulting in potential stiffness problems. One of the most important stiffness design parameters to consider in any tall building design is its maximum deflection, which is usually in the neighborhood of a five hundredth of the building height. Two modes of deformation, bending and shear, primarily contribute to the total deformation. This paper now presents a stiffness-based design methodology for diagrid structures for tall buildings and discusses how to allocate bending and shear deformation for optimal design.

A diagrid structure is modeled as a vertical cantilever beam on the ground, and subdivided longitudinally into modules according to the repetitive diagrid pattern. Each module is defined by a single level of diagrids that extend over ‘n’ stories. Figure 6 illustrates the case of an 8 story module. In order to more accurately estimate the lateral rigidity provided by diagrids, all the required lateral stiffness is allocated to the perimeter diagrids, and consequently core structures, omitted in Figure 6, are only gravity systems in this study. Depending upon the direction of loading, the faces act as either web planes (i.e., planes parallel to wind) or flange planes (i.e., planes perpendicular to wind). The diagonal members are assumed to be pin-ended, and therefore resist the transverse shear and moment through axial action only. With this idealization, the design problem reduces to determining the cross-sectional area of typical web and flange members for each module. Following the design methodology developed by Moon et al. (2007), member sizes for the modules can be computed using Equations (2) and (3) customized for each design case.

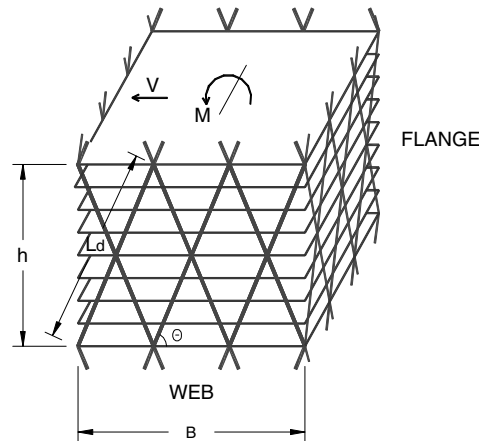


Figure 6: Typical Diagrid Module

$$A_{d,w} = \frac{VL_d}{2N_{d,w}E_d h \gamma \cos^2 \theta} \tag{2}$$

$$A_{d,f} = \frac{2ML_d}{(N_{d,f} + \delta)B^2 E_d \chi h \sin^2 \theta} \tag{3}$$

$A_{d,w}$: Area of Each Diagonal on the Web
 $A_{d,f}$: Area of Each Diagonal on the Flange
 V : Shear Force

M: Moment
 L_d: Length of Diagonal
 E_s: Modulus of Elasticity of Steel
 θ: Angle of Diagonal Member
 γ : Transverse Shear Strain
 χ : Curvature
 N_{d,w}: Number of Diagonals on Each Web Plane
 N_{d,r}: Number of Diagonals on Each Flange Plane
 δ: Contribution of Web Diagonals for Bending Rigidity
 B: Building Width in the Direction of Applied Force

Optimal stiffness-based design corresponds to a state of uniform shear and bending deformation under the design loading. Uniform deformation states are possible only for statically determinate structures. Tall building structures can be modeled as vertical cantilever beams on the ground, and uniform deformation can be achieved for these structures. (Connor, 2003) Then, the deflection at the top, $u(H)$, is given by

$$u(H) = \gamma^* H + \frac{\chi^* H^2}{2} . \tag{4}$$

H: Building Height
 γ* : Desired Uniform Transverse Shear Strain
 χ* : Desired Uniform Curvature

where $\gamma^* H$ is the contribution from shear deformation and $\chi^* H^2 / 2$ is the contribution from bending.

The design begins by specifying the desired bending deformation and shear deformation of the structure. In order to specify the relative contribution of shear versus bending deformation, a dimensionless factor ‘s’, which is equal to the ratio of the displacement at the top of the structure due to bending and the displacement due to shear, is introduced. The ‘s’ value can be calculated by

$$s = \left(\frac{\chi^* H^2}{2} \right) / (\gamma^* H) = \frac{H \chi^*}{2 \gamma^*} . \tag{5}$$

The maximum allowable displacement, one of the most important stiffness-based design parameters for tall buildings, is usually expressed as a fraction of the total building height.

$$u(H) = \frac{H}{\alpha} \tag{6}$$

Determination of a value for α is an engineering decision. Typical values for α are in the neighborhood of 500. Noting equations (4) and (5), equation (6) expands to

$$u(H) = (1 + s)\gamma^* H = \frac{H}{\alpha} \tag{7}$$

Then,

$$\gamma^* = \frac{1}{(1 + s)\alpha} \tag{8}$$

Also, χ^* is determined using equation (5).

$$\chi^* = \frac{2\gamma^* s}{H} = \frac{2s}{H(1 + s)\alpha} \tag{9}$$

It remains to establish a value for ‘s.’ Then, the design of diagonals in each module can be performed using Equations (2) and (3) customized for each design case. This paper investigates the impacts of specifying different values of the desired bending and shear deformation of the structure – selection of different ‘s’ values – toward the optimal stiffness-based design, which uses the least amount of material to meet the design requirements.

Design Studies: Uniform Angle Diagrids

Member sizes for the modules were computed using Equations (2) and (3) customized for each design case. Both $N_{d,w}$ (Number of Diagonals on Each Web Plane) and $N_{d,f}$ (Number of Diagonals on Each Flange Plane) equal 6 in the particular design study case shown here. An estimate of the contribution of the diagonals on each web to the bending rigidity is made by adding one extra diagonal on each flange, resulting in $\delta \approx 2$. To carry the shear forces and bending moments calculated using the code loadings, diagonal member sizes are increased from the top toward the bottom. As an

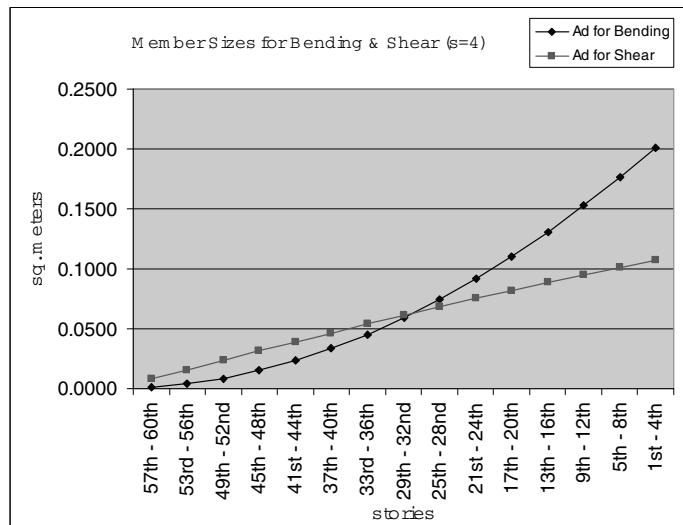


Figure 7: 60 Story Uniform Angle Diagrid Design Example with s=4

example design, profiles of the required areas for the typical diagonals in the web and flange planes for the 60 story diagrid structure having a 69 degree diagonal angle are plotted with $s = 4$ in Figure 7. Since the wind can blow in either direction, the role of a plane can be either a flange or a web. The building considered here has a square plan and the preliminary design value for the module is taken as the larger of the two values. When 's' is taken as a much smaller number such as 1, the areas required to limit displacement due to bending govern the design for square plan buildings. When 's' is taken as a much larger number such as 10, the areas required to limit displacement due to shear govern the design for square plan buildings. These cases represent the extreme limits for 's', and do not lead to economical design. Choosing 's' to be about 4 and selecting the greater of the member sizes required for the bending and shear criteria, leads to the most economical design. In this case, the bending deformation requirement governs for approximately the lower half portion of the building, and the shear deformation requirement for the upper half.

Regarding the relatively short diagrid structures, with an optimal angle of about 63 degrees, the optimal 's' for the 40 story structure is about 4 and for the 50 story structure, about 6. Regarding the relatively tall diagrid structures, with an optimal angle of about 69 degrees, the optimal 's' for the 60 story structure is about 4, and those for the 70 and 80 story structures are about 5 and 6 respectively. These results clearly demonstrate that bending deformation governs more as a building becomes taller. In addition, it can be noted that as the angle of diagonals becomes shallower, the optimal 's' value becomes larger. This is because the shear rigidity provided by the shallower angle diagonals increases, while the bending rigidity provided by them decreases, and consequently there will be more bending deformation. Table 3 summarizes these study results.

Table 3: Optimal 's' for Uniform Angle Diagrids

Diagrid Height (Aspect Ratio)	Near Optimal Uniform Angle	Near Optimal 's'
40 Stories (4.3)	63 Degrees	4
50 Stories (5.4)	63 Degrees	6
60 Stories (6.5)	69 Degrees	4
70 Stories (7.6)	69 Degrees	5
80 Stories (8.7)	69 Degrees	6

Design Studies: Varying Angle Diagrids

The same design methodology presented for uniform diagrids can be used for the design of varying angle diagrids. In terms of structural efficiency, varying angle diagrids with steeper angles towards the base of the building are recommended for the structures with height to width aspect ratio bigger than about 7. The optimal 's' for the 60 story varying angle diagrids is about 2.2, and those for the 70 and 80 story structures are about 4.2 and 4.9 respectively. (Table 4) Note that decimal fractions are used now instead of integers as 's' values because the design with changing angles is more sensitive to the change of 's' values than with uniform angles. As an example design, profiles of the required areas for the typical diagonals in the web and flange planes for the 80 story diagrid structure having varying angles of 63, 69, and 73 degrees from the top to the bottom are plotted with near optimal $s = 4.9$ in Figure 8.

Table 4: Optimal 's' for Varying Angle Diagrids

Diagrid Height (Aspect Ratio)	Diagrid Angles (Top → Bottom)	Near Optimal 's'
60 Stories (6.5)	63, 69, and 73 Degrees	2.2
70 Stories (7.6)	63, 69, and 73 Degrees	4.2
80 Stories (8.7)	63, 69, and 73 Degrees	4.9

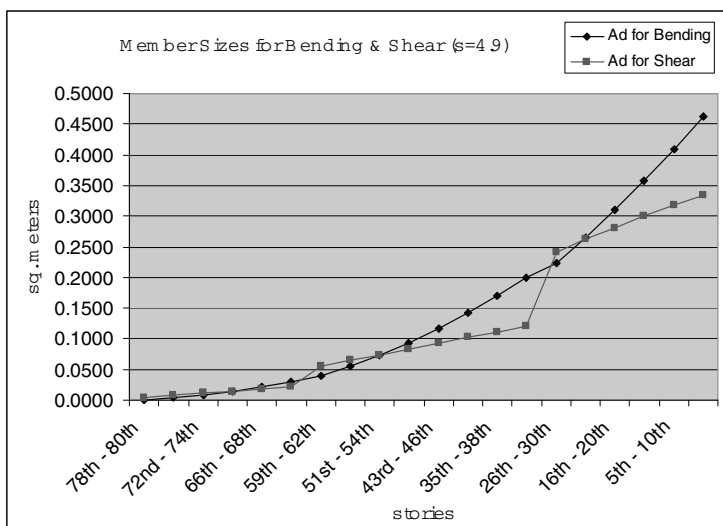


Figure 8: 80 Story Varying Angle Diagrid Design Example with s=4.9

Conclusion

This paper provides design guidelines for diagrid structural systems for tall buildings. A taller structure with a large height-to-width aspect ratio tends to behave more like a bending beam. Thus, as a building becomes taller, the optimal diagrid angle increases because steeper angle diagonals resist bending moments more efficiently by their axial actions.

For uniform angle diagrid structures, with aspect ratios ranging from about 4 to 9, the range of the optimal angle is from approximately 60 to 70 degrees. For very tall diagrid structures with an aspect ratio larger than about 7, a varying angle diagrid configuration which has gradually steeper angles towards the base of the building generates a more efficient design with less material than the uniform angle configuration. However, for diagrid structures with an aspect ratio smaller than about 7, the uniform angle diagonals produce more efficient designs.

A stiffness based design methodology for diagrid structures was presented to help preliminary member sizing of diagrid structures with less iteration. Guidelines for selecting the bending to shear deformation ratio for optimal design was also presented. The author expects that the design methodology presented in this paper will be very useful to both engineers and architects for preliminary design of tall buildings employing diagrid structural systems. Based on these guidelines, structural and architectural decisions at the early stage of design can be made in a more integrative and efficient way.

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An Overview of the State-of-the-Art Report for Glass Structures

Abstract

This paper is an overview of the report “State-of-the-Art Report for Glass Structures” written to fulfill the requirements of Independent Study in Civil Engineering CIEN E9101, Columbia University. The report was done under the guidance of Professor Christian Meyer. The objective was to survey the current trends in the innovative uses of structural glass or glass structures. Most recent printed material was researched in the topic to understand material glass and details, components, and systems for glass structures. Many of the most innovative and contemporary structures built with glass were examined. The current codes and engineering standards offering guidance in the design of glass structures were summarized. Design philosophies, stress and deflection criteria were discussed. Finally, future trends were investigated, and areas of pending and possible new research were outlined. This report was not intended to be a design guide.

Introduction

Within the recent 15 years glass has been developed from a simple cladding material in the façade to a multi-purpose material in architecture. The reasons for this can be primarily found in developments in glass technology, which have led to higher mechanical strengths, an increase in the processing options and crucially, considerable improvements to the properties relevant to building science, while at the same time maintaining transparency.

Glass as Material

Owing to its strong atomic bonding forces, glass with an intact microstructure and perfectly smooth surface possesses a very high mechanical strength [13]. Nevertheless, “Griffith Flaws” occur from the damage to the microstructure within the body of the glass and scratches and checks on the surface. These flaws cause high stress peaks once a mechanical load is applied. Being a perfectly elastic-brittle material as seen in the Fig. 1 of comparative stress-strain curves for different materials, these stresses can not be dissipated by means of plastic deformation.

Sobek [13] further explains that the failure of glass related to imperfections occurs as follows. Upon exceeding a “critical (tensile) stress,” the crack grows at the tip of a notch or a crater due to initial impurities. (Fig. 2) In some circumstances this occurs in small steps, coming to a halt in between. In fracture mechanics, this slow or “stable” crack growth is regarded as subcritical, and is primarily determined by the duration of the load. So, greater loads can be allowed for glass when the load is short term. However,

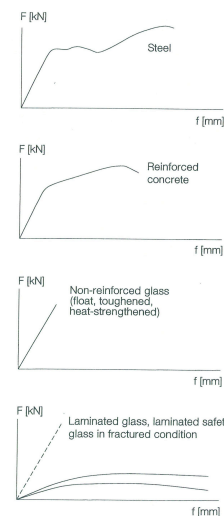


Figure 1. Stress-strain for glass and other materials [13]



Figure 2. a) Stress peak under tension b) Member under compression [13]

However,

it must be clearly distinguished that the strength reduction caused by the accumulated surface damage over a long period of time is different from the strength reduction due to subcritical crack growth resulting from permanent loads.

Modern glass technology allows several types of glass to be manufactured for architectural use in the building today.

Float Glass or Annealed Glass is the common glass that is manufactured today to be used in framed windows and many common applications but not structural uses. The manufacture of float glass is a fairly modern technique. It has increasingly replaced sheet and plate glass production since 1960. The annealed glass can be fabricated up to 45MPa strength.

In structural applications, plain float or annealed glass is always enhanced with some additional process to improve after-breakage behavior and to increase strength. Float glass breaks into large pieces. (Sim. to Fig. 4) Three types of glass can be manufactured from float glass to be used in structural applications: Tempered glass, heat-strengthened glass, and laminated glass [13], [9], & [21].

Tempered Glass (a.k.a. toughened, pre-stressed glass) is the type of glass where compressive stresses are introduced at the surfaces to raise the strength against mechanical and thermal loads. This type of glass is tempered by heating and then quenching with air jets. This glass can not be cut, and failure of this glass results in a formation of many small pieces of glass.

The stress distribution in a tempered glass sheet with drilled holes is shown in Fig. 3. During tempering, a protective layer of compressive pre-stress is created around the holes and along the edges. This layer helps load to be supported at zones where high stresses occur due to contact from linear and pin supports.

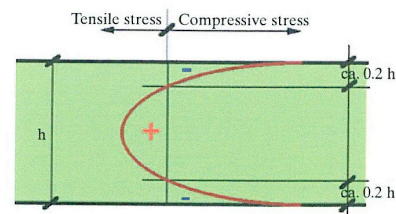


Figure 3. Stress profile over the thickness [21]

Tempered glass can be manufactured up to 19mm thickness and up to 120 MPa typically, although much stronger tempered glass is available. The toughening mentioned here mostly refers to the glass strengthened or prestressed by thermal toughening, which gives a stress profile over the thickness, as shown qualitatively, in Fig. 3. There also are the processes of mechanical toughening and chemical strengthening of glass [21].

Heat-Strengthened Glass is like tempered glass but the cooling process is slower. The strength developed is almost half of that of the tempered glass, approximately up to 70 MPa. Such glass breaks into larger pieces resembling the failure of the annealed glass. Heat strengthened glass can be produced up to 12mm thickness. Particularly

when the glass is designed for an overhead structure, heat strengthened glass is preferred due to its strength coupled with good post-breakage behavior. (Fig. 3)

Laminated Glass (or Laminated Safety Glass) is the assembly of two or more pieces of tempered, heat-strengthened, or float glass. The more shear is transferred between two layers, the more they will act monolithically. Composite action can result in enormous efficiencies structurally. In fact, assuming thickness of laminations are the same, when calculated using elastic theory, the deflections can be reduced by a factor of 4 while the stress can be reduced by a factor of 2 for a simple beam with a concentrated load in the center.

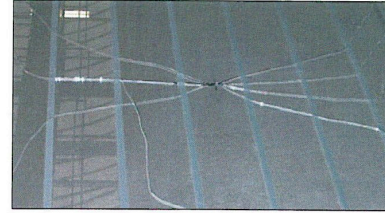


Figure 4. Crack patterns in typical heat strengthened glass [21]

Commonly, these laminates are bonded together with an interlayer of polyvinyl butyral (PVB). Extensive studies are being made to evaluate the performance of the PVB layer under different load durations and temperature conditions. The shear modulus of the PVB interlayer is highly influenced by the change in the temperature, showing viscoelastic behavior at temperatures close to 140°C [16]. As an example, at 50°C, and for a load duration of 1 month, the shear modulus will be reduced to 0.052 MPa. Recently, DuPont has been reporting success with its SentryGlas laminated product using a 100% Polymer interlayer. The shear modulus for the same duration and temperature can measure 2MPa, which is a significant improvement [41].

Detailing and Connections

The detailed report written to The structural connections can be made through holes in the panes, clamped connections either as point supports or edge supports, friction connections, and glued or welded connections. Compression, tension, and shear specific connections or combinations thereof can be made.

Point Supports

Point supported glass panes are a good example for stress transfer via contact. The contact in this case is provided by an interim compressible material between the glass and the usually patented pinned supports. This material is usually an elastic resin.

Linear Supports and Clamps

Forces in a glass element will be transferred by way of friction, i.e. mechanical interlocking of the microscopic surface imperfections of both contact faces. Besides the mechanical interlock, adhesive forces also occur at the contact face. When the bottom edge of the glass pane is supported linearly, the weight is transferred both by friction and bearing. Clamps also transfer force mainly via friction. They are however, not linear edge supports but point supports that compress the glass normal to the plane of glass to achieve friction. Linear supports and point supports are the most common connection types for glass structures.

Structural Silicone

A flush surface can be achieved when structural silicone is used in-between glass panes. In this way of supporting, the silicone at the bottom edge takes the gravity and the silicone-applied side edges will help with lateral load transfer. Additional lateral supports are generally required. Visual discontinuity at the façade in the whole can be minimized with this type of connection.

Glued and Welded Connections

Acrylate glues have been used for many years in glass furniture production, but they generally carry only minor loads and are only used internally (with minor temperature changes only and little environmental impact such as UV-light). Furthermore, per [40], glued connections are already applied to safety relevant parts in mechanical engineering. In modern cars the windows are used to stiffen the body. The stiffening forces are transmitted by glued connections with high-modulus polyurethanes. In order to develop a durable and suitable glued connection for external structural use, proper glue compositions are still being tested and are in general very seldom used. From a structural point of view the aspects of any adhesive to be used as a permanent glazing connection shall be examined by means of testing, calculations and further careful examinations with regard to durability, strength, dynamic loading and ease of application.

Currently, epoxy and acrylic based glues are being tested in various load bearing connection configurations in compliance with the aging and loading conditions of the European Technical Approval for Structural Sealant Glazing Systems (ETAG 002-P1:2001). Below are sketches for possible glued connections. The results of one such test Sedlacek [40] confirmed that the durability of the glued connections may be limited by environmental influences like UV-radiation, humidity, or high temperatures. Better results are expected with a toughened epoxy-adhesive and a larger application thickness which could balance stresses due to thermal expansions.

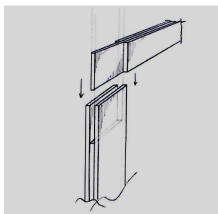


Figure 5.a Connection of two glass elements by gluing [24]

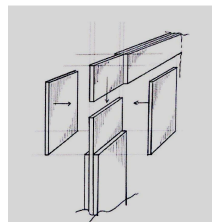


Figure 5.b. Connection with separately glued-on panels [24]

Both of the structures built with glued connections show that the best way to make a glued connection is by overlapping parts of the connections (Fig 5). Nijssse [24] states that three kinds of glue are appropriate for a structural connection of glass elements, the UV-hardening glue, the epoxy based glue, and the silicone based glue. The first two are strong and stiff glues, the silicone based glue

is weak but very elastic, which is in some cases the best choice. In the Sonsbeek Pavillion [24] project a silicone-based glue was used since the length of the glued joint was sufficient to produce a strong-enough joint with a silicone weak in shear. In an other project the in Netherlands, the Leiden Conservatory [24], and the Broadfield House Glass Museum in England [26], the other two types of glue were used.

Structural Components and Systems

4.1 Understanding Glass Structural Systems

To understand and classify the glass structural systems of today, a re-thinking of traditional structural systems is necessary. The traditional philosophies will be determined and where the structural glass would fall within these categories will be discussed in the first part of this chapter. This section will introduce the fail-safe and safe-life concepts which are critical to the understanding of glass structural systems.

Werner Sobek, in his Glass Construction Manual [13], presents a great way of classifying structures to understand the complex relationship between the parts that form any structure. The following paragraphs are summarized from [13].

First is the Hierarchical System where the structures that compose a building follow a scheme of primary, secondary, and tertiary structural scheme. The primary structure would be the load bearing core, all columns, walls, floors, and bracing to carry horizontal and vertical loads. The failure of such members might cause total collapse of the structure. In other words, there is a demand for an unconditional guarantee against failure of the individual member without considering the role of this member within the structure as a whole. This design philosophy is called safe-life concept. The secondary structures in such a system would be floors which are not part of the primary system, such as partitions, roof structures, annexes, and façade elements. The tertiary systems would be all those that are part of the secondary ones, but whose stability is not critical to the stability of the secondary structures. e.g a window within a façade element.

The second category contains Discrete-Work systems. These are systems where individual parts or substructures in each case perform only a clearly defined function. Such systems are frequently encountered in glass construction because this prevents total collapse following failure of a single component. Although after such a failure the element can no longer carry loads, this is acceptable for a short period of time.

The third category contains Redundant Systems. A system is considered redundant when it contains elements whose functions can be taken over by other elements if they fail. In this case the failure mechanism corresponds to a “fail-safe mechanism” where a local failure is allowed as long as it does not lead to a disproportionate failure of the whole construction.

Load Bearing Elements

Linear elements: beams, bars, rods, columns

Glass girders and beams

Flat Glass Beams, Glass Fins

These are beam members composed of only glass webs attached usually to façade glazing vertically or horizontally as reinforcing ribs for the glazing. These members are often too long to be produced monolithically, therefore structural splices are required. These members will be further discussed in the façade systems section.

Glass Beams combined with other materials

If the brittle glass, strong in compression, is complemented by a ductile material with high tensile strength, then the tensile forces present in the beam, in the broken condition in particular, can be safely carried. In Figure 38 below, three types of glass reinforced beams are shown as well as a very recent application by Schlaich-Bergerman [35].



Figure 6.a Glass beams with tension rods [35]

There is current research underway to test glass-reinforced composite girders as well. A recent composite beam construction involving glass are timber-glass girders for a hotel in Switzerland. In this project timber was used for ecological and manufacturing reasons. Timber flanges were glued to a glass web, forming an I section.

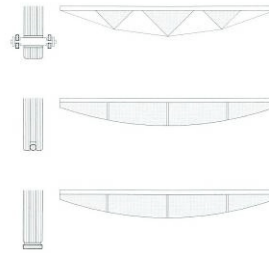


Figure 6.b Possible glass beams with steel tension element [13]

Glass columns (compression members)

Glass columns were used for the first time in an office building at St. Germain en Laye in France (1995), and then further developed for the Rheinbach Pavillion in Germany (2000) [35]. Current research is underway to establish stability calculations for glass-only columns. The maximum load on a compressed element depends mainly on the initial deformation, the glass thickness, and the PVB interlayer for laminated material. In contrast to other materials, failure occurs when the maximum tensile stress due to loading exceeds the embedded tensile strength of the annealed float glass. See figures below for the buckling study of laminated panes. A simplified design can be done by using an “effective thickness” for a monolithic cross section. Other, more complicated, methods of design are given by Luible in [23].

Another compression member utilizing glass structurally is the concentrically prestressed glass tube. These tubes have been used as restraining members for the atrium wall of the Tower Place in London [13]. They are comprised of glass tubes with steel end blocks and internal tension rods anchored in the end blocks [18], [38]. Recently, all-glass pipe columns were used in a prototype All-Transparent Pavilion in the Netherlands [29] made out of laminated glass with a clear resin interlayer.

Two interesting projects realized recently in Japan utilizes glass panels as façade elements supported on the ground at the bottom level and supported solely on each other as the façade height increases. These panels are working in compression as well as bending against the wind and earthquake forces [17].

Glass slabs.

Single-layer and multi-layer glass slabs with and without bond can be constructed. In addition, laminating glass panes with only edge seals is possible. In this kind of slab, the flat hollow box creates an internal air or gas cushion which behaves structurally,

according to the Mariotte-Boyle law also called a catheter effect. Usually, it is recommended by most manufacturers that unsupported glass shall span up to only 3'-0" if used horizontally [42], [43].

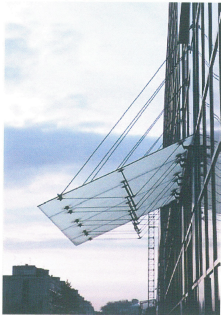


Figure 7. A bottom-trussed canopy [36]

A very popular system is the trussed slab (Fig. 7). The clear spans of panes could go up to 15' in some cases. In this type of construction, the glass pane carries the compressive forces, the truss, provided along one or two axes, the truss forces [13]. The compressive stresses superimposed on the bending stresses are beneficial for glass, provided that no buckling problems are introduced.

Glass shear panels

Structural shear panels were used in various projects mostly as an integral part of the overall glass framing. In some of these projects by Schlaich [28] (Figs.49-51), the glass panels act both as vertical load bearing members as well as lateral stabilizers. Essentially, glass boxes are created with panels in both orthogonal directions to take the lateral forces and stabilize the panel perpendicular to it. This system was created by using a grid of prestressed steel cables within the glass joints that are grooved into the edges of the panes.

Glass shells or curved glass members

These can be made by cold or warm bending. Omitting the compression members of a grid shell and introducing the load directly into the glass created a real glass shell. This was for the first time realized at Maximilian museum in Augsburg (Fig. 8). Another approach for shell

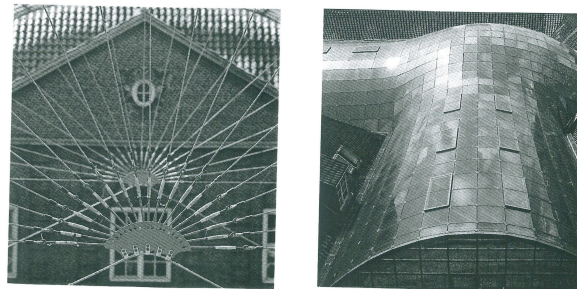


Figure 8. Maximilian Museum in Augsburg [13]

structures with cold-bent laminated glass panes, now realized at the façade of City Hall of Alphen aan de Rijn, Holland. The laminated glass panes were heated to approx. 60 C° to reduce the shear modulus of the PVB interlayer to almost zero, then the glass panes were cold bent on site. In this particular project a steel cable takes the thrust. Other applications of this new and very promising technique are found in the Netherlands such as the Canopy of 's-Hertogenbosch Station [27]. Compared to the warm bent glass, cold bent glass produces a pre-stress in the glass pane that is favorable for carrying downward loads, allowing for small glass thickness.

The cost of warm bent can be up to twice as high as that of cold bent due to the load carrying behavior and the bending method used for warm bending. Another advantage is the increased visibility through the cold-bent glass, since warm bent members are subject to surface distortions during the process, thereby decreasing visibility and optical quality [20], [27].

Glass panes bent in two directions are even more favorable than panes bent in one direction because in this case uneven loads can be carried as axial forces without bending. Heinz Isler has already postulated the one-piece glass dome. This topic will be further discussed in the Freeform Glass Structures section below.

4.3 Structural Systems for Glazing

Freeform Glass Domes or Lattice Shells

These structures are comprised of plane glass panes using triangular or quadrangular meshes. These are steel-and-glass structures taking advantage of both mesh units. The design of these structures requires efficient mesh geometry and results in a light but robust and very redundant structure [34].

Freeform glass domes are the heirs to the delicate glass greenhouses of the mid 19th century. Domes of iron-and-glass were a hallmark of this era. Countless small glass panes brace their slender metal primary structures. The individual panes of these constructions are embedded in a hard yet still slightly resilient putty and so form a myriad of interconnected load paths. The residual stability and redundancy of such a system is immense. The high degree of statical indeterminacy, and impossibility (so far) of correct computer modeling would involve a huge amount of work, if one tried to analyze such structures [13]. Therefore, the metal framing is accounted for, but not the glass panes, in the structural analysis of these systems.

One important point with regard to these meshes is that if a multiple curvature is needed to create the form, triangular meshes work better to give the desired geometry. However, triangular meshes are in general less efficient than the quadrangular ones. The quadrangular mesh normally leads to square-mesh systems with one diagonal brace in every bay capable of taking tension and compression, or X-bracing for tension only. If the diagonal members are undesirable, then the nodes must be designed to resist bending [13].

Cylindrical lattice shells, vaults, and arches are also a viable and elegant solution used in some acclaimed projects around the world. These can be cable-guyed arches (Fig. 8) or cable-guyed barrel vaults. Cylindrical shells curved in one direction and spanning over long spans suffers from stability problems, which cannot be solved without additional measures. Therefore, cable guying or trussing has been a viable solution to mitigate the buckling problem. Jorg Schlaich in particular has pushed the boundaries of this construction particularly with his design of the cylindrical arched roof over the Maximilian museum in Augsburg (Fig 8). In this shell, the steel compression members were omitted and the load was directly introduced to the glass.

Pre-stressed Cable-Net Facades and Roofs

Cable net structures are composed of horizontal and vertical cables anchored at a stiff perimeter structure and glass panes directly connected to the cables at the intersections. Cable-net facades can be designed to withstand blast loads in addition to function as mere enclosure. With sufficient cable length, the pre-stressed cable

itself is an excellent distributor of the displacements in the façade plane uniformly across all glass joints. One of the first projects realized with this structural concept was the Hotel Kempinski Façade in Munich Germany [28] (Fig. 9).



Figure 9. Hotel Kempinski façade, Munich, Germany

The load bearing behavior is directly related to the amount of pre-stress in the cable. The cable load is inversely proportionate to the cable sag. An increase in the pre-stress decreases the cable deformation. The key is to limit the deflections to a desired and acceptable level. Too little deflection will result in overstressing the connections, and too much deflection will not be tolerated by glass panes [28]. A detail of the connection is presented in Figure 9.

The glazing in the net could be called upon to stiffen the network, and this is being recently studied especially for blast loads [35]. One example is the Time Warner Center in New York where the main atrium's 32 m x 8.7 m (105' x 28.5') façade is a pre-stressed cable net façade structure designed to withstand to a peak pressure of 4 psi. The total calculated out of plane deflection of the glass was calculated as 320 mm after 0.4 sec.

The structural calculations for a glazed cable-net always proceed on the basis of determining the warping and racking of each individual pane and comparing these values with the permissible deformation values.

Other structural systems for façade glazing

The glazed facades can be made of glass "fins" which can span the façade horizontally, vertically, or both. These would be continuous or rigidly connected for continuity. Also, possible is the cable trusses which may be located both inside and outside the façade for taking tension due to positive and negative lateral wind loads.

Engineering Design and Relevant Codes and Standards for Glass

So far in the worldwide engineering community, there have been various ways of describing the mechanical behavior of glass and deriving design methods from these descriptions. However, the developments seem to have only led to a very limited number of design codes and standards which can be easily picked up and applied by engineers who are not very familiar with the topic. Moreover, these codes do not appear to cover all the possible structural applications with glass. A lot has been accomplished in the field, and some spectacular structures have been designed and built, but most of these examples seem to be experimental and limited to the expertise of specific structural design offices around the world. The patented glazing systems for pin connections, cable-trussed panes, glass handrail and guardrails, and other various systems offered by manufacturers such as Pilkington and BGT provide simple design tables systems, but these are usually used by the manufacturer's engineers, and they are often proprietary in-house design guidelines obtained over

time by empirical testing by the firms and by the individual firm’s experience in the field.

Specialty plate engineering software such as SJ MEPLA, and in-house software by ARUP were developed to help with the design of single panes with different support conditions. The engineer usually designs the system using a commercial software package and develops more detailed models with the specialty programs. In addition, laminated glass panes under various load conditions can be designed with “Window Glass Design 2004” [41].

$$\sigma_{eff} \leq f_{g,d}$$

$$f_{g,d} = \left(k_{mod} \frac{f_{g,k}}{\gamma_m k_a} + \frac{f_{b,k} - f_{g,k}}{\gamma_V} \right) \cdot \gamma_N$$

σ_{eff} : effective stress
 $f_{g,d}$: design resistance
 k_{mod} : load duration influence
 $f_{g,k}$: characteristic glass strength
 γ_m : partial safety factor for glass
 k_a : influence of the glass surface size
 $f_{b,k}$: characteristic glass strength of pre-stressed glass
 γ_V : partial safety factor for pre-stress
 γ_N : national coefficient

Figure 10. Design tensile strength formula based on [6]

In many situations designing with glass, especially for structures that “push the envelope”, there is no agreed method or precedent for design, and in these instances an approach based on first principles of mechanics material is necessary. The draft code prEN 13474, [6] provides a valuable starting point, (Fig. 10), dealing with the fundamental issue of strength and stiffness of glass in various conditions as a general basis for design in glass. In many instances the designer uses a fundamental approach of anticipating scenarios where the glass fails and levels of redundancy are provided by duplication or by alternative load paths that do not rely on the integrity of the glass [18].

Over the years various design philosophies emerged. Out of three methods, Allowable Stress, Probabilistic Approach, and Limit States Design, the most common approach, Limit States Design will be summarized below.

Limit States Design

This design takes into account various probabilistic distributions both on the load side and on the material side. Furthermore, this method considers various modes of failure in by taking into account the probability that a particular load case occurs or that a component fails. This method is really important in the design of glass structures because with its underlying design philosophy of “fail-safe design,” which was discussed in Chapter 3, it allows for the post-breakage behavior considerations in design. This in turn provides an inherent “residual stability” concept. A crucial concept for an ideally elastic material such as glass, this provides possibilities for analyses of the structure in the damaged condition, including the residual load-bearing capacity of the damaged glass element itself, behavior upon failing, residual stability of the structure as a whole, tolerances of large deformations in the damaged component and the structure as a whole.

It must be noted that even though there might be relevant design guidance from the IStructE publication[7], BS 6262 [2] and BS 5516 [3], the manufacturer’s design guidance is required for the last phase of the design. The manufacturer might have an off-the-shelf system already designed and available to order, which can suit the

application at hand. Otherwise, a customized system needs to be designed, still requiring a lot of input from the manufacturer to make the system work with the characteristics of the glazing fabricated by that particular manufacturer.

prEN 13474 and the very few referenced design codes, incorporate the limit design philosophy. Strength verification is made by comparing a so-called effective stress which is a weighted average value of the distributed main stresses on the glass surface, with the maximum tensile resistance of glass. The maximum tensile strength of the glass is given as a function the influence of size and quality of glass surface, accumulated load duration, and environmental conditions:

Serviceability is also a major concern in glass structure design. Table 1 is from the German National Code, [4] as the technical rules for the use of continuously supported vertical glazing. This table is one of the only guides that makes reference to deflections.

type of glass	type of linear supports	Deflection limit	Definitions
single glass		$f \leq l/100$	l: span in main load-carrying direction d: glass thickness *)
insulated glass unit	four sides	$f \leq l/100$ and $f \leq d$	
	two or three sides	$f \leq l/100$, $f \leq d$ and $f \leq 8 \text{ mm}$	l: length of free glass edge d: glass thickness *)
*) the nominal glass thickness of a laminated safety glass unit is $d = \sqrt[3]{d_1^3 + d_2^3}$			

Table 1. Allowable deflections according to [5] & [9]

However, according to Laufs [9] the maximum allowable value should be more restrictive than those given values in Table 1. The deflection of the main steel frame that the glazing is attached to should be one of the main reference points in the glazing deflection allowance. The supports of the glazing also must be flexible enough not to create secondary effects in these areas. Relative displacements between glass panels have to be another concern which is not generally addressed by the codes. Moreover, architectural considerations further limit the deflections in addition to the structural requirements. For example, panes with excessive deflections create negative lens effects on the façade surface, an undesired effect on glazed façades.

Future Trends

As more and more transparency coupled with structural performance is demanded, one area to be improved and better understood will be the connecting structural glass components with welding with melted glass or gluing. The current research shows that the short term structural performance of glued or welded connections is quite satisfactory. More studies will be done to improve the long term behavior and durability of glued connections. Currently, the few options in glue materials such as epoxy resins, UV hardening glues, and silicone based glues, do not respond really well to environmental influences. This curtails their use in exterior applications. Visibility problems with glued connections and difficulties of applying glue to tight corners are some other concerns with this process. Recently, DELO Industries, a German company, developed a family of adhesives, PHOTOBOND, for glass connections. Case studies in collaboration with the University of Delft in the Netherlands revealed that this glue family might overcome the many handicaps of earlier glued connections. [39]

The composite behavior of laminated glass is another area to be improved in the future. At the present, the PVB interlayer shows very small shear resistance under long term loadings, hence the capacity of the composite can not be fully utilized for loads such as self weight or other dead loads. Moreover, delamination of the interlayer under long term loadings and environmental conditions such as moisture must be studied further to improve the composite behavior. Dupont SentryGlas product can offer significant improvement in interlayer technology.

Applications of cold-bending of glass are very promising for the future. With the patented process from Holland, sheets can be bent on site preventing the problems with delivery of the plates bent in the shop. The behavior of the cold-bent glass needs to be studied further for industry wide applications. Experimental and numerical research on cold-bent glass sheets resulted in a model to predict the pattern of deformation, to be used for design purposes. More research is necessary to derive design rules for the stresses in cold bent doubly-curved glass sheets with a rectangular shape [20].

The pressure is also on the glass manufacturing industry to produce larger sheets of glass to minimize the number of connections and increase visibility. Hence, perhaps, studies to enhance the material characteristics of glass to introduce some ductility to the material or to enhance the manufacturing processes for allowing thicker and larger panes might be a future trend [21].

The stability of the glass components is a topic being studied by the universities that are advanced in the glass structure research. Due to their high slenderness, glass structural components, although strength-wise adequate, are prone to stability problems. An all encompassing safety concept for glass structures based on probabilistic methods, taking into consideration the stability problems such as column buckling, lateral torsional buckling and plate buckling of glass panels is being prepared at Swiss Federal Institute of Technology [23].

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New Modern Structure with Historic Appearance

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SCOPE OF THE PROJECT

The project encountered is a remodeling of a historic building with major structural improvements. The design is intended to preserve the outer appearance of the building and to re-build modern offices inside. This paper presents comparisons of floor layouts, structural connections, and major considerations for practical structural engineers.

The project is located in Las Animas County in the City of Trinidad in Colorado. The building used to be the Las Animas County jail. The building is a historic building. It was built about one hundred years ago. The building was designed in the Victorian-style of the 19th century. It is four stories tall with basement. Today, it is a vacated building of the county. The county now intends to utilize this building as an office building. The elevation of the building are shown on Figure 1.



Figure 1 Building Front Elevation



Figure 2 Building w/surround buildings

The City of Trinidad is considered one of historic sites in Colorado. This building is surrounded by urban buildings with the County Courthouse nearby (Figure 2). A corridor connects the jail to the Las Animas County Courthouse. To preserve this historic building, the county requests that the exterior wall of the building should be kept and its interior structure will be remodeled so that it can be utilized as a modern office building.

The county specifications require using IBC 2003 (International Building Code 2003) [1] to process the design. According to IBC2003, this building is in the Seismic Category B zone. Seismic design will be taken into consideration.

EXISTING AND PROPOSED STRUCTURE

This building is four stories above the ground. The exterior wall of the building was constructed with stone. The thickness of the wall varies: 24" (approx.) on the first floor, 19" (approx.) above. Since the building was built a long time ago, no historic documentations can be referred to. The existing foundation and floor structure design information is also unknown.

The existing floor system consists of concrete floor slabs with concrete columns. The age of these existing concrete members is unknown. Since the building was used as a jail, the spaces of the concrete columns are small and do not meet the requirements of a modern office. We decided to remove the existing floor structures and columns and to replace these structures using the proposed structure system.

Since the proposed inner structure is surrounded by the existing structure, the building/floor drift of the proposed structure will be limited. The building/floor drift will be controlled by space between the proposed structure and the existing structure at the floor level, and the building drift of the proposed structure will be limited by the space at the roof.

Design of connection detail is critical. The proposed structure shall not be against the existing structure when it is under seismic force. The existing structure shall not affect the proposed structure when it is under windy conditions.

COMPARISON OF STRUCTURAL DESIGN

The proposed design will remove the entire existing support structure that is inside of the building and replace it by a four-story proposed structure. SAP200 [4], and EnerCalc [5] will be used to analyze and design the structure members. SAP2000 and EnerCalc are common applications in the structural engineering field.

Model A and Model B are established for structural analysis. Model A is a relative rigid structure system, which uses concrete shear-wall and framing. Model B is a relatively flexural structure system, which uses composites of concrete/metal floor with steel columns.

Building drift is major object when these two models are under simulated seismic forces. When the building is under lateral seismic force, drift of building/floor will affect or damage the existing exterior wall system.

MODEL A

To reduce the horizontal movement of the building, concrete shear walls and columns will be used to resist lateral seismic load[7]. Due to the greater rigidity of this type of

structure, the drift of the building will be reduced so that it will reduce damage on the existing exterior wall.

The Model A is established by SAP2000. The model is a concrete shear-wall/fram system. Five shear-walls are displayed on the plan (Figure 3). This model is used to analyze the drift of the interior structure. Figure 4 shows Model A in 3D view. The floor weight excludes the weight of the exterior existing wall.

EnerCalc is used to calculate rigidity center of floor and the mass center of floor, and distribute the seismic force to each of the shear-walls on each of floors. Figure 5 presents the accidental eccentricity, the distance between the center of mass and the center of rigidity. It is less than $3.15 \text{ ft} = 0.05L_x$ (perpendicular length of building).

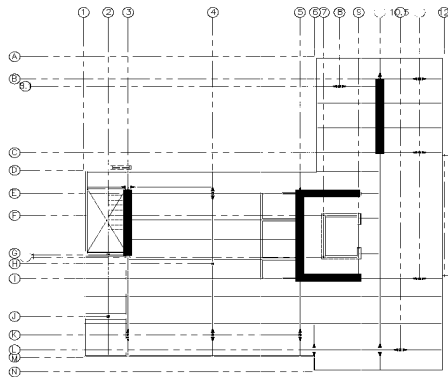


Figure 3 Building Plan View (havey line are shear walls)

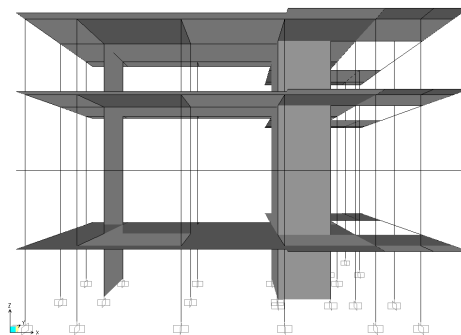


Figure 4 Model A in 3D view

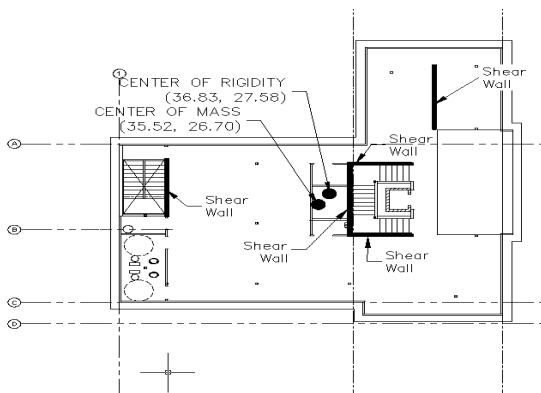


Figure 5 Location of the accidental eccentricity

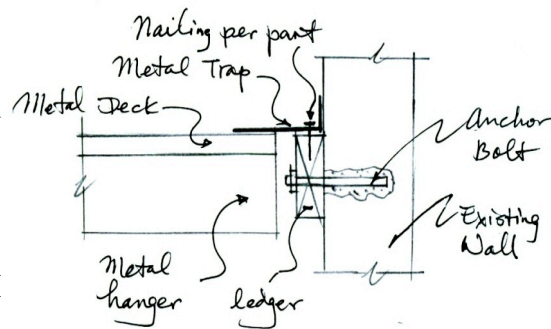


Figure 6 Connection Detail

The results of SAP2000 show that the maximum drift is on the roof of Model A. The maximum drift is 0.23 in, that is less than 2% of total building height[2].

Connection detail is a key design for this model. This connection not only allows the vertical movement of proposed structure when the proposed structure settles, but also allows the horizontal movement when the proposed structure is under the lateral seismic force. A suggested connection detail is shown in Figure 6.

Due to a lack of information regarding the existing foundation, footing design and construction may be challenge jobs. To reduce the disturbance on the existing structure, we consider conventional pile foundation or the static-pushed pile foundation that uses static-pressure to push the piles into place when the proposed shear-wall is close to the existing exterior wall.

MODEL B

Using steel material will reduce the weight of the floor structure, which can reduce the seismic forces on each of the floors. Therefore, it will reduce the drift of the building/floor and thus reduce damage on the existing exterior wall.

Model B is a steel column/braced steel framing system and is analyzed by SAP2000 and EnerCalc. Five braced frames are layout on the plan (Figure 7). The type of braced frames are “inverted V chevron” braced frames [2]. Figure 8 shows the elevation of Model B. The weight of the structure excludes the existing exterior wall weight.

EnerCalc analyzes the rigid center of the floor and the mass center of the floor (Figure 5), and distributes the seismic forces to the floor levels on the each of braced frames. These seismic forces are applied to Model B (Figure 9). The deformation of the braced frame under the seismic forces is shown on Figure 10.

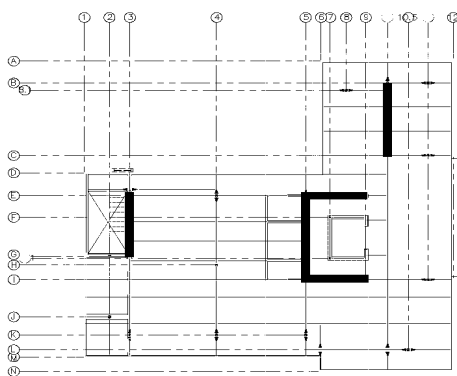


Figure 7 Building Plan View
(havey lines are braced framing)

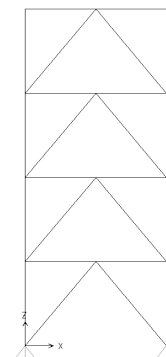


Figure 8 Model B Braced Frame

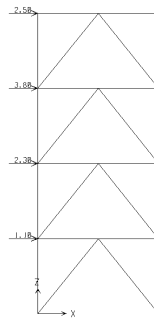


Figure 9 Model B w/Seismic Force

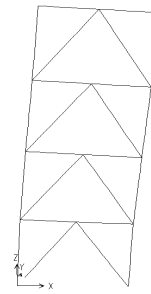


Figure 10 Deformation under Seismic Force

The results of SAP2000 show that the maximum drift is on the roof of Model B. The maximum drift is 0.15 in, which is less than the result that is derived from Model A.

The design of connection detail is also important. The connection allows the proposed structure to sustain vertical movement when the proposed structure settles. Horizontal movement of the proposed structure will be reduced since we use steel structure. Since the proposed steel columns can be away from the existing exterior wall, the proposed footing construction will not disturb the existing footing of exterior wall. A suggested connection detail is shown in Figure 11.

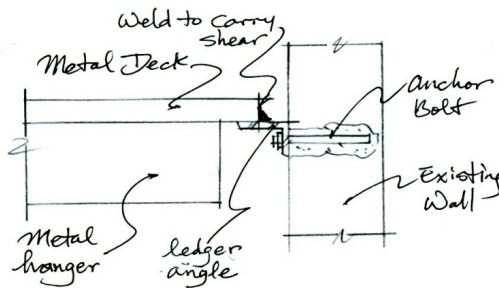


Figure 11 Connection Detail

ASSUMPTIONS

These models have simulated fixed connections to the ground.

Since these designs are preliminary designs, the dead loads on these models are simplified. No wind load and loading combination are taken into account.

The concentrated seismic loads on each of floor masses are converted into uniform loads that act on each of the floor levels.

CONCLUSIONS

This project is in low seismic zones. Therefore, these two types of structures are not significantly different. The separation between the existing and the proposed structure is necessary. For this project, the separation will be at least 0.3 in. This separation may be established by connection details, or flexural materials. By establishing this separation, the proposed structure can move slightly vertically and horizontally.

The renovation of a historic building is a challenging job for structural engineers. It requires analyzing the strength of existing materials, structural layout, effect upon adjacent existing structures, connection details, and construction ability and procedures.

Setting of a center of mass and a center of rigidity is very important for reducing torsional shear on the proposed structure.

ACKNOWLEDGEMENTS

I would like to thank JR Engineering and my co-workers at JR Engineering for being a consistent source of support and encouragement.

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The Renewal of Grant Hall: An Original Netsch Designed Building on the Campus of the University of Illinois at Chicago



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Abstract

In 2005, a \$2M gift to the University of Illinois at Chicago by Sid Port in memory of his daughter provided the starting point for the first whole building renovation of one of the original Walter Netsch designed buildings on the UIC campus. Constructed in 1965, Grant Hall was representative of the “Brutalist” architectural style which is the defining theme throughout the master planned campus designed by Netsch. With the building envelope deteriorating and the opportunity to invest in a significant renovation, UIC leadership and architectural consultants proposed a new, modern, energy efficient design for the building which would serve the campus through the middle of the 21st century while blending in with and paying tribute to the original architecture. Now serving as the Sandi Port Errant Language and Culture Learning Center, the \$5.5M project includes a geothermal well field and heat pumps as well as numerous other energy saving features. This paper will discuss the planning and challenges associated with the renovation as well as the considerable debate around such a radical departure from the original design.

The History of Grant Hall

The east campus of the University of Illinois at Chicago (UIC), originally referred to as the Chicago Circle Campus after the interstate highway interchange located in the Loop area of downtown Chicago, is a master planned campus dating



Figure 1. Walter Netsch

from the 1960's designed by the internationally recognized architect Walter Netsch of Skidmore, Owens, and Merrill. Other than some landscaping and the removal of second story walkways between buildings, the original award winning Netsch designs are untouched from the day the campus opened in 1965. Although the campus has remained true to the brutalist architectural philosophy that Netsch created, certain aspects of the buildings have not worn well over the passed forty years. As described in a Time Magazine article from the day, "Netsch used the cold, durable materials of the city—concrete, granite, hard-surfaced brick—to build his university (Time 1966)." While earning Netsch much acclaim, this harshness has, in recent years, been recognized as a factor that has dated the campus appearance. Probably the most often heard comment from students and faculty is their desire for more natural light, which is not surprising in that the pre-cast concrete window walls were designed to transmit only 2% of daylight into the classrooms through the center openings and 11% and 27% through the top and bottom openings respectively (Gueft 1966). Occupants also asked for more openess, softer finishes and more uniqueness in their spaces. The original design specified only beige as the color for floors in the classroom buildings and all walls and exposed surfaces were concrete with exposed aggregate (Gueft 1966).



Figure 2. Grant Hall in its original configuration

Student lockers lining the hallways of the classroom buildings gave the campus the feel of transience as opposed to the more residential experience that UIC leadership has been trying to foster for the past decade. Exposed structural and architectural concrete has exhibited significant spalling and many windows have cracked and faded with time. The campus has long imagined a way to update its facilities, but has lacked the resources necessary to undertake such an endeavor.

So when, in 2005, Sid Port, a major benefactor to UIC, donated \$2 million toward the establishment of the Sandi Port Errant Language and Culture Learning Center to memorialize his daughter, the campus undertook a study to determine how best to use the funds. The facility decided upon to house the center was a small classroom building whose mechanical, electrical, and plumbing systems were essentially unaltered from their original condition. The HVAC system was, in effect, uncontrollable, consisting of mostly broken and leaking Nesbitt units, and the electrical system could not support modern requirements for a Language Laboratory. Safety code compliance was well out of date. There was no sprinkler system and the fire alarm system was original to the 1960's construction. Entrances and restrooms were inaccessible.

At first analysis it was obvious that the entire \$2M gift plus a substantial campus contribution would be absorbed in just adapting the outdated infrastructure to accommodate a state-of-the-art language lab and addressing the code issues. Following that initial look, a feasibility study was commissioned to determine the cost of a complete renovation of the building to bring it up to 21st century standards and provide a high tech environment for the next 40 years. When the study showed that the cost of a whole building renovation was not much more than the cost of correcting the deficiencies in the existing building, the campus undertook to raise the additional funds from a variety of sources. In total, \$5.5M dollars (including furniture, computers, and advanced audio visual equipment) was secured to commission a complete gut and renovate project for Grant Hall.

Design

A competition for the right AE was held and SmithGroup was selected as the architect. As planning for the project proceeded, one major goal of the architect was to increase the natural light in the building which was, due to Netsch's original concept for the campus buildings, insignificant. The glass in the concrete infill panels was practically opaque and the precast concrete sections were not insulated. Additionally, the exposed concrete columns and floor slabs were showing signs of severe spalling from reinforcing steel corrosion caused by inadequate cover. UIC Architects considered replacing the infill panels with clear glass panels in their current location within the building frame and repairing the concrete spalls as part of the project. At the schematic level design review, SmithGroup proposed the bold idea of encapsulating the entire building in a glass and metal façade reflecting a more modern design. This solution would simultaneously solve the problems of the exposed reinforcing steel and the outdated and energy inefficient wall panels and increase the useable square footage of the classrooms.



Figure 3. Concrete spalls

Since this was a radical idea which represented a clear departure from the standard theme of the campus buildings, the proposal was presented to the UIC Design Review Committee, a group comprised of architects from the Department of Architecture faculty and distinguished members of the Chicago Architectural community. The reaction of the architects was immediately favorable, even though the scheme represented a significant change from Netsch's design. The group struggled with how to bring the building into the 21st century but still retain a link to its origins. Also, the design team recognized that whatever



Figure 4. Columns visible through glass

theme we decided on would provide the basis for all future renovations of the eight other small classroom buildings on the campus. With that in mind, the Design Review Committee suggested that the new façade be designed in a manner such that the existing columns, which had been exposed and would now be part of the building interior, be visible through the glass and that the color of the mullions and cladding match the color of the concrete from the surrounding buildings. The group recommended continuing the 2.5 foot exterior grid by specifying a 2.5 foot spacing of the vertical window wall mullions. The design was subsequently presented to the President of the University and to the Board of Trustees who embraced the idea of transforming the campus' 40 year old buildings into more modern facilities while still paying homage to their original design. The design team also undertook to create a facility that reflected the best technology available in education, energy efficiency and green construction.

Engineering Technology

Early in the design process, it was clear that to meet the objective of the language center, the building environment would need to be regulated year round, which is not our standard. Campus buildings are served by central plants which deliver high temperature hot or chilled water depending on the season and cannot adjust to Chicago's unpredictable climate during transition months. Because the building was to be full of electronic equipment, the heat loads would be such that during those months, the interior spaces had the potential to be intolerably hot. Rather than adding supplemental systems and bearing the expense and maintenance cost, the team decided to investigate a geothermal heating/cooling system. Very preliminary calculations were made and a request for a grant from the Illinois Clean Energy Community Foundation was submitted in July 2006. A \$153,000 grant was awarded later in the year. Because UIC is an urban campus with minimal green space, this decision was fairly bold and success could not necessarily be guaranteed.

A test bore was drilled and the initial calculations resulted in a design calling for 55 wells at 300 feet. While this was a favorable outcome, the number of wells was still significant for the land available. A second consultant was retained who recommended increasing the depth to 500 feet even though the test bore only went down to 250 feet. Further analysis of the calculations led the UIC in-house design team to believe that more capacity than necessary had been called for in the geothermal well field. The UIC team recommended that the number of wells be reduced to 14 wells at 500 feet which reduced the cost and the disruption to the campus. As a backup, the team specified tapping into the campus-wide hot and chilled water distribution system in case we realized extremes in temperature beyond the capacity of the geothermal field.



Figure 5. Location of geothermal field

The well field was designed to be expandable so that the other two buildings in the three building cluster could be modernized and also draw on the geothermal system. The project team also researched the newest, most efficient heat pumps for the heating/cooling system, actually waiting for the best products to be delivered just in time to install before completing and opening the building. As an additional technology application to save energy, a high efficiency glass was specified for the curtain wall system which reduced the projected heat load even further and cut the requirement for the number of geothermal wells. Also, the entire building was updated with T8 fluorescent fixtures to provide the most efficient lighting possible.

Code Updates

Because the entire east campus was designed and constructed in the 1960's, compliance with current codes is a significant issue. Since this was to be a complete gut/renovate project, addressing code deficiencies was an important aspect of the work. The scope included installation of a sprinkler system, a new fire alarm system, enclosing interior staircases and installing exits to the outside. Additionally, the front entrance and rest rooms were made ADA compatible and accessible. An elevator had been added to the building in the 1990's making all three floors of the building accessible. A two story walkway enabled



Figure 6. ADA accessible entrance

access to the second and third floors of the adjoining Douglas Hall, and a new third story walkway is planned to link Douglas Hall to the adjoining Lincoln Hall when that building is renovated thus making all floors completely accessible. All restrooms have either been or will be modified according to current accessibility design standards. Also, based on a study of the number of total restroom facilities by gender, renovation of each of the two other buildings in the complex will include increasing the number of facilities so as to reduce crowding during inter class periods.

Learning Environment

The project also included packing the new facility with state-of-the-art



Figure 7. Writing Center

technology to enhance the learning experience. Rooms full of computers to aid in foreign language learning were part of the design package as was a writing laboratory where students who needed tutoring could walk in at any hour and receive assistance in a classroom wonderfully lit by natural light and climate controlled year round by the building's new

geothermal system. On the third floor of the new Grant Hall, two wide screen TV's receive television broadcasts from around the world at all hours of the day, and language students can relax in the lounge area while watching their favorite show from China or Germany (among many others). To assist in the energy efficiency of the building a shade system was designed to cut down on the direct thermal load at peak hours during the day. The shades were designed specifically for the exposure of the building so that north and west elevation shades filtered more of the direct light than those on the south and east. Traditional blackboards were replaced by whiteboards thus eliminating chalk dust as an irritant and distracter in the class room. A centrally monitored security system that would detect any piece of electronic equipment being brought outside of a classroom or the building was installed at the door of each classroom and at the exits from the building.



Figure 8. TV from around the world

In addition to formal learning spaces in the facility, a series of student areas known as “oases” were designed for hallways and otherwise open, but unused areas. This has been a common feature in UIC classroom buildings to make up for the starkness of the architecture and Grant Hall lent itself very nicely to accommodating several of these areas. The concept of the oasis is very much fitting for the purpose of Grant Hall as a language learning center.



Figure 9. Student Oasis

Plans for the future

As a follow on to the very successful Grant Hall project, the campus decided to fund renovations of the other two buildings in the complex – Lincoln and Douglas Halls – following the same design concept as Grant Hall. With minor modifications to the window wall design to save cost, planning for Lincoln Hall proceeded. Again, with consultation from the UIC Design Review Committee members, the question of introducing color into the other buildings was discussed. This was a far-reaching consideration for the campus as every building on the master planned east campus is essentially identical in architectural style and color. When Grant Hall was renovated, the architects took great pains to find a way for the mullions and metal panels to match the concrete color as closely as possible. The thought of changing exterior



Figure 10. Rendering of Grant, Douglas, and Lincoln Halls in the future

colors was presented to the Chancellor, the President of the University, and with their approval, to the Board of Trustees. The new scheme was adopted and the result will be the first ever application of a color other than the original concrete on the east campus area. Future mullion and cladding colors will remain neutral so as to be in

harmony with Grant Hall and original construction. More bold colors will be introduced in stairwells and other areas that are visible from the outside of the building.

Lessons Learned

Several significant lessons have been learned from the design and construction of Grant Hall. We have incorporated many of those into the planning and design of Lincoln Hall.

Curtain Wall Design

In the Grant Hall renovation plan, the curtain wall was attached to the existing building floor slabs and extended outward from the perimeter of the building by approximately 18 inches. This necessitated that a new foundation be constructed to support the load from the 3-story high glass and metal structure. The cost of this construction and the time involved in constructing it was significant. When the design team planned the Lincoln Hall project, they explored solutions that would reduce this cost and construction time, but would still be aesthetically pleasing. The result was modifying the curtain wall design by bringing the first story window wall back flush with the perimeter of the floor slab and leaving the first story columns exposed. The second and third story window walls were to be brought in closer to the second and third story floor slabs and cantilevered with mounting brackets. By bringing the window wall in closer to the floors slabs the stress on the mounting brackets would be reduced.

Geothermal Well Field Design

Post commissioning analysis of the performance of the geothermal well field in comparison to its design appears to show that the well field and heat pumps are performing better than expected. As previously described, the number of wells was reduced from that originally specified by the engineer after review by the in-house UIC design team and consultant. It was felt that not enough credit was being taken for natural shading, diversity of loading, and glass efficiency. Subsequent designs show that the optimum well field design is actually smaller than first anticipated, reducing the cost and the land area needed for the expanded well field. In the design process for Lincoln Hall, engineers are taking into account the observed efficiency of the well field and have reflected that efficiency in the number of wells called for. Since eventually all three buildings in the complex will be served by the same geothermal well field, the bid package for the Lincoln Hall geothermal field includes an option for additional wells to serve the full capacity of Douglas Hall, the third building in the complex. With favorable bids, UIC hopes to avoid additional cost and disruption by completing the entire well field for both remaining projects during Lincoln Hall construction.

Planning and Construction

Numerous lessons were learned as bidding and construction proceeded. We learned that the concrete infill panels were much more difficult to remove than would have been implied by the as built drawings and that some concrete sections that were noted as having construction joints were actually integrally poured with the floor slab. Overall, the greatest lesson learned was the construction cost of the building systems that had been specified and the best way to bid the project to optimize efficiency and reduce expense. Estimating the cost for renovating Lincoln Hall has been much simpler, allowing the users to understand more precisely what they can expect to afford in the way of program improvements and what must be invested in infrastructure. This more informed estimating has also enabled a multi-year capital improvement plan with a high degree of confidence in the projected budget. Additionally, by analyzing the results of the Grant Hall design and costs, we determined that we could afford to design Lincoln Hall to LEED Silver standards and seek accreditation from the United States Green Building Council. Design Development documents indicate that we are well on our way toward achieving this first ever LEED certification for UIC.

Energy efficiency data

Although a preliminary study comparing a geothermal system to a unit ventilator system was performed during the Grant Hall feasibility phase, a more detailed analysis was undertaken when planning proceeded for the Lincoln Hall renovation so that there would be a basis for determining whether the geothermal system would, indeed, save significant energy in comparison to its now better defined costs. The Lincoln Hall study showed that the geothermal system was, by far, the most energy efficient of any system for year round heating and cooling. Since August 2007, UIC engineers have been recording energy consumption data from Grant Hall. In January 2008 a more advanced metering system was installed which measured energy consumption from the building HVAC system only. The data in Table 1 show that readings have been very close to the predictions of energy use in our studies.

	Measured Geothermal	Study Geothermal	Study Unit Ventilator System
Grant Hall	633*	610	910
Lincoln Hall**	NA	991	2182***

Notes: *Based on annualized reading adjusted for degree days
 **Includes two Make up Air Handling Units vs. one for Grant Hall
 ***4-pipe system with supplemental chiller vs. 2-pipe system for Grant Hall
 Lincoln Hall is approximately 50% larger than Grant Hall

Table 1. Comparison of actual and predicted energy use (MMBtu/yr)

Despite extremely hot and cold days since Grant Hall's commissioning, the geothermal wells have been able to supply all of the heating and cooling demanded by the building without using the backup system linked to the campus central hot and chilled water loops.

Conclusion

The project has been an overwhelming success and has been extremely well received by the students, faculty, and administration of the university. At the ribbon cutting ceremony in September 2007, over twenty members of the Port family attended as did the President of the University and the Chairman of the Board of Trustees. As an additional gesture of recognition to the architectural origins of UIC, the University of Illinois Board of Trustees voted to award an honorary Doctor of Architecture degree to Walter Netsch at the May 2008 commencement exercises.



Figure 11. Grant Hall at night

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Method of Using Weather Information for Support to Manage Building Construction projects

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Abstract

Weather conditions have a great effect on building construction projects. The projects would avoid the risks from weather conditions if the weather information were used. This paper proposes to use weather information to support to manage more effectively building construction works on a construction field. A project management software is used to show how the real-time weather information is integrated to schedule management by coding in VBA. Further studies are necessary to complete the supporting system for the utilization of historical weather data and the prediction of weather effects on construction schedules.

Key Words: Construction Management, Weather Information, Schedule

INTRODUCTION

Lots of climate changes have occurred in the whole world and the amount of damage is considerable in Korea (Jeong, Y. M. 2006). The climate has a great effect

on construction projects. In general, project managers consider rainy season of summer and cold weather season of winter, but now they must consider climate changes. Thus, it is necessary to predict weather and to use weather information in order to prevent safety accidents in construction fields, then to decrease cost, and to increase profits of the contractors.

Weather information is provided by not only KMA(Korea Meteorological Administration) but also private companies such as Weather-I or K-Weather. The KMA provides the information every 3 hours, but the private companies provide the customized information demanded by a person or a company. If the weather information is utilized in managing construction project, the risks occurred in construction process will be removed or reduced. Therefore, this research proposes a method to manage construction schedules using real-time weather information provided by KMA.

USE OF WEATHER INFORMATION

Literature review

Thomas et al.(1999) describes the loss of productivity to erect the steels and the effect of weather from snow and cold temperatures. It seems that the method is not fit for frequent weather change in these days. El-Rayes and Moselhi(2001) developed a knowledge-based system that was quantified the impact of rainfall on productivity and duration of common highway construction operations. The system used weather information from the closest weather station, analyzed weather claims, and then generated realistic schedules. But, it seems that it is difficult to adapt the system to building projects.

Researches to use weather information are studied in these days in Korea. Jeong and Lee(2000) simulated weather and estimated non-working days for each weather factor, and reflected them to a network. They forecasted the construction duration by the probability of relations between weather factors and activities. However, the system needs how to use real-time weather information needs and needs to consider the relations of activities. Kim et al.(2004) analyzed the relation of weather factor and productivity in RC works of a multi-story housing project, using regression analysis of 30 years' weather factors such as temperature, humidity, length of daylight, rainfall, and wind speed. The research presented the possibility to use the

results to planning phase or schedule management of building projects. Shin et al.(2007) proposed a method to calculate the construction duration for high-rise buildings by using the regression analysis of weather factors and productivity, and then simulated virtual weather by using Crystal Ball in order to estimate the duration of structural works.

Previous researches focus on loss of productivity or non-working days resulted by unfavorable weather. This means the importance of weather information increase as climate change occurs. So, more precise prediction is needed to estimate non-working days and construction duration and a method to use the weather information is required in the construction phase.

Cases using weather information

Weather information is used in most of industries. Shipbuilding industry has many outdoor works and its productivity and quality are affected much by weather condition. STX Ship Company makes use of shipbuilding-specific weather information to control efficiently the production time and reduces cost to prevent risks occurring in tasks such as cost painting, loading, and so on (MK Economy 2006).

In construction industry in Korea, Hyundai Development Company runs on-line network for weather and provide weather information to about 30 construction sites, which reduce labor cost, equipment cost, and etc. by applying it to the planning of tasks, equipments, and resources. Also, the quality of in-situ concrete is ensured by controlling mixture ratio in consideration of weather (KMA 2002).

SCHEDULE MANAGEMENT AND WEATHER INFORMATION

Construction and Weather

Because buildings are constructed on diverse sites, construction schedules must be managed considering the weather of local area. In Korea, weather changes according to location, and the 4 major climate changes that are the yellow-sand winds, extremely hot weather, heavy rain, and strong wind become big problems (Jeong, Y. M. 2007). So, it is necessary to analyze long-term historical weather information and the 4 major climate changes as well and to understand their features

in order to find impacts on building construction.

Risk from climate is negative factor that does damage to construction projects, but can be positive factors doing good when it is used actively. It is necessary to review methods to use weather information in diverse industries and to search a new method to adopt them to construction industry. The new method seems to decrease construction costs or construction duration.

Though it is reasonable to manage construction plans considering both long-term historical weather data and real-time weather change, the scope of this study is restricted to the utilization of real-time weather information.

Construction schedule management

Construction works on site may start, continue or stop as weather condition is good or bad. So, when work plans are made, construction manager makes a decision of construction duration after calculating non-working days by predicting weather on site. Mostly, the manager's experience or historical weather data is used in order to predict weather.

Weather information is reflected in planning phase. However, weather condition of the work day makes practically tasks go or not in construction phase. That is, it means that weather information cannot affect the schedule change of forward tasks in performing tasks. This is because construction managers cannot be sure of the accuracy of weather information from KMA and it is difficult for the managers to change schedules affected by weather. In this research, we propose a method to change schedules by using weather information in a project management software in order to resolve the problems.

DEVELOPMENT OF A SYSTEM USING WEATHER INFORMATION

Method for system development

One of methods using historical weather information is to evaluate non-working days to calculate construction duration or to decide start date. However, there are few reviews to use weather information in performing tasks.

Project management softwares such as P3(Primavera Project Planner),

MSP(Microsoft Project) are much used for construction management. P3 is mainly used in general contractors, but it has weakness of inconvenience for making use of user demand information. MSP is used not only by construction companies but also by IT(Information Technology) related companies in order to manage software projects. Also it has advantage that a user can easily change information in user format by using VBA(Visual Basic for Application).

This paper proposes a method to connect the weather information of KMA with schedule network in MSP and to use it in performing tasks. Also the paper includes a manner to provide construction managers with severe weather alert such as typhoon, yellow-sand winds, and so on.

System development

(1) Weather information of KMA

Weather information on KMA homepage is transferred to the network in MSP by codes written by VBA. That is, the weather information is saved to a text file, and then the code in MSP loads the file and applies it to the network to decide whether each day of the network is working or non-working.



Fig. 1 Weather information on KMA homepage (www.kma.or.kr)

(2) Weather impact

A number of activities are affected by weather condition, but some activity is never affected. Thus, weather impact on each activity is identified first, and then the activity needs to be rescheduled. In this paper, the weather impact is defined as the intensity of weather effect on activities only by rainfall. The weather impact 1.0 means that when it rains an activity cannot be performed ever. Similarly, the impact 0.5 means any activity can be performed only in the morning or in the afternoon. Table 1 is the activity list and the weather impact of each activity. Fig. 2 shows the network diagram for the activities.

Table 1. Activity list and weather impact

ID	Activity	Duration	Start	Finish	Predecessor	Weather Impact
1	A	3.d	2007-11-26	2007-11-28		0.0
2	B	2.d	2007-11-29	2007-11-30	1	1.0
3	C	3.d	2007-11-29	2007-12-01	1	0.5
4	D	3.d	2007-12-02	2007-12-04	3	0.0
5	E	2.d	2007-12-01	2007-12-02	2	1.0
6	F	2.d	2007-12-05	2007-12-06	4	0.0
7	G	3.d	2007-12-07	2007-12-09	6,8	1.0
8	H	2.d	2007-12-03	2007-12-04	5	0.5
9	I	5.d	2007-12-07	2007-12-11	6	0.0
10	J	3.d	2007-12-12	2007-12-14	9,8,7	0.0

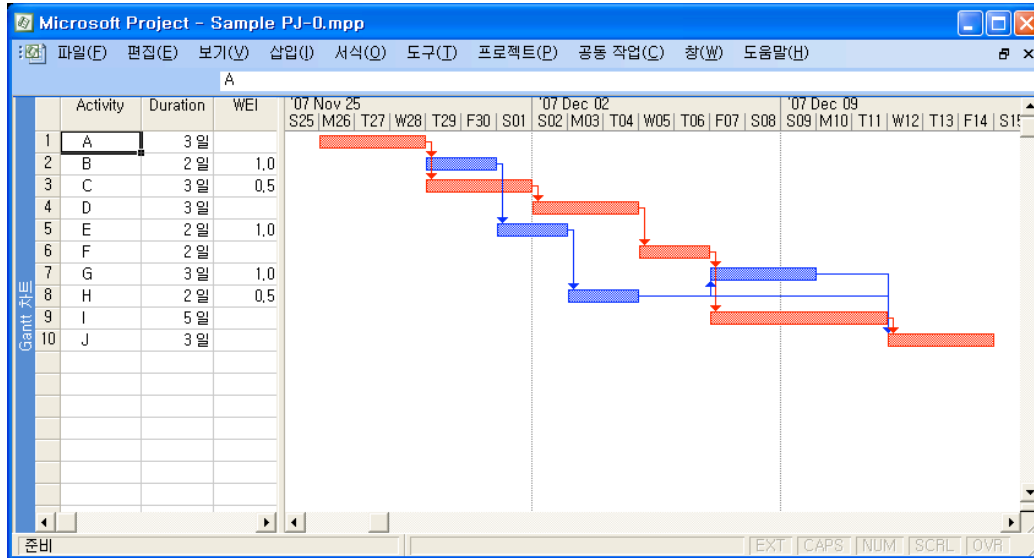


Fig. 2 Sample Network

(3) Activity Rescheduling

The weather impact reflecting the rainfall probability of one day from weather information reschedules working days. An activity in progress is divided into two activities by the date when the weather information is reflected. The first activity is not moved, but the second activity needs to be rescheduled in some days later. Activities not starting can be delayed or can be divided and then part of them can be postponed by 'Split Task' menu in MSP.

Fig. 3 shows the updated sample network considering the weather impact. If today is Nov. 30 (Friday) and it rains on Dec. 2 and on Dec. 3, activity D and E are considered to be rescheduled. Activity D has 0 weather impact, so it must be not rescheduled. But activity E is performed on the first day, and the task of second day is delayed in 2 days.

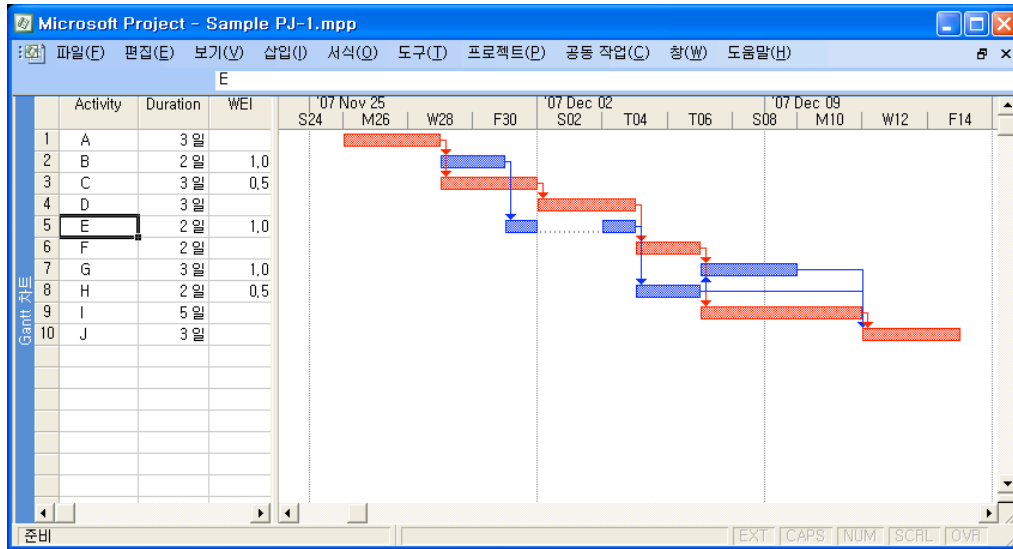


Fig. 3 Updated Network (Activity E is split into 2 sub-activities)

(4) Update of Construction plans

When the schedule of an activity is changed, critical path may also change. So, a construction manager should review the construction plans. And he must confirm whether the network rescheduled by the proposed system is reasonable. In the sample network, though activity E is divided by the effect of weather, the critical path does not change. Considering the efficiency of performing works, the activity E has better be delayed in 2 days (shown in Fig. 4) rather than be divided (shown in Fig. 3) because it is good to perform works without break. Then the construction plans can be updated.

When overall construction plans are updated and new network is set up, the construction manager repeatedly performs the works and checks the weather impact.

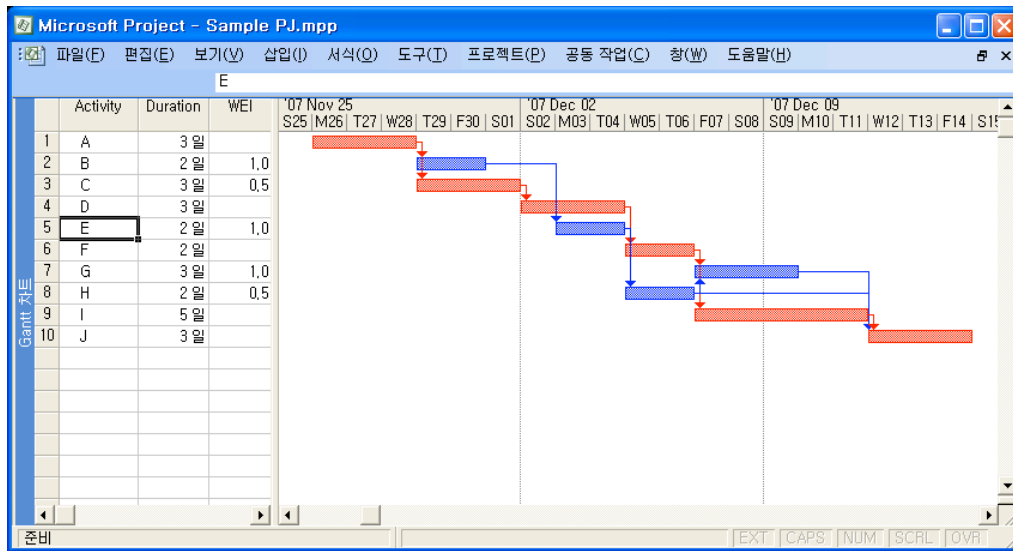


Fig. 4 Updated Network (Activity E is delayed in 2 days.)

CONCLUSIONS

Weather condition is regarded as a risk causing construction duration delay or safety accident in construction projects. The risk may be an opportunity if construction plans are made considering weather information in planning phase, and the plans can be updated on the basis of weather forecast.

This study proposed a method to update the schedules of network using weather information. The results of the study are the following.

(1) Real-time weather information can be obtained in the Internet, and so this study links the information with MSP for construction management.

(2) Working or non-working depends on weather such as rainfall or wind etc. In this paper, activities are rescheduled by the intensity of weather effect only when it rains.

(3) A system is developed by VBA in MSP in order to update effectively the schedules of activities affected by the weather. But, considering the efficiency of performing works, the critical path must be reviewed and then the overall construction plans can be updated.

(4) A sample network is used for verification of the usability of the real-time weather information. In further study, it is also necessary to investigate the effect and intensity of weather factors except rainfall.

Acknowledgments

This research was supported by a grant(05CIT-D05-01) from Construction Technology Innovation Program funded by Ministry of Construction & Transportation of Korean government.

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Use of Historical Overhead Costs for Estimation and Control Purposes

By

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Abstract: The competitive environment in the construction industry has been causing contractors to bid with a very small margin of profit. At the same time, overhead costs are ever increasing relative to the project direct costs, as a result of increased requirements for trades coordination, works supervision, and the provision of temporary site facilities. This paper examines the components of the project overhead costs through a thorough study of actual cost data from building-type projects undertaken by a major contracting firm. To this end, the paper makes a comprehensive classification of such costs and presents statistics that can be useful in assisting cost professionals in both estimating and controlling this project's crucial cost component. Finally, the paper discusses three sub-processes aimed at (1) maintaining an updated database of previously incurred overhead costs, (2) estimating the overhead costs for new projects, and (3) controlling the same throughout the construction period.

Introduction

Nowadays, the construction industry is becoming highly competitive, causing general contractors to opt for lower margins of profit (Shash and Zamel 1993, Assaf et al. 1999). Taylor (1994) indicated that "in the competitive environment of today's construction industry, the difference of a single percentage point can be the difference between winning and losing a bid, making a profit, or bankruptcy". Under such an environment, the logical way to remain competitive is to attempt to control and decrease costs, without jeopardizing the quality of provided services.

Overhead costs represent a strong candidate for control as these could act as a "silent harvester of retained earnings" (Snodgrass 1991) and have the property of never going down on their own (Assaf et al. 1999). In fact, Gandhi (1996) mentioned that, during the last 20 years, overhead costs have steadily crept upward due to many reasons, and controlling such costs has become an issue that can make or break a

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company. On the other hand, as construction projects are becoming more complicated with more work trades involved, increased coordination, supervision, and temporary site facilities are contributing to overhead costs becoming of greater significance relative to the overall cost of construction. That said, the research described in this paper aimed at (1) clearly identifying the components of a building project's overhead costs, (2) investigating the project's characteristics affecting overhead costs, (3) and devising a mechanism for accurately estimating project overhead costs during the bidding phase and effectively controlling these costs during the construction phase. The adopted research methodology relied on the use of actual overhead costs data collected from six recent building-type projects undertaken by one of the top-five general contractors based in Lebanon.

Cost Control in the Construction Industry

It is important to note that the construction industry in the United State has the second highest rate of bankruptcy among all other industries (Tavakoli and Utomo 1989). By the same token, no other business probably needs an effective cost control system more than the construction business does (Perera and Imriyas 2003). Regardless of the type of contract, contractors are bound to control construction cost in order to decrease the possibilities of cost overrun and to attain a maximum possible profit. The cost control process could be defined as “the process by which a cost objective is used as a reference value against which an actual cost is compared for timely decisions” (Shash and Zamel 1993). Kerzner (2003) defines cost control as “not only monitoring costs and recording data, but also analyzing the data in order to take corrective action before it is too late”. Heinze and Westney (1997) stated that “no one can control anything by concentrating on what has already happened”. Recording and analyzing cost data is one step, but the crucial step is to control the source of this cost. Construction management must therefore implement a proactive rather than a reactive attitude of cost control (Opfer 1992).

The growing idea of “labor-broker-type” general contractors makes understanding and controlling all aspects of a company's overhead extremely essential. When following such a practice, general contractors are likely to receive comparable quotations from prospective subcontractors, thereby making the markup for overhead and profit a more significant factor in deriving to a bid price (Taylor 1994). According to Nassar (2003), most major general contractors are self-performing a decreasing amount of the work contracted for while subcontracting the majority of it. Hinze and Tracey (1994) reported that, especially in building projects, 80 to 90 percent of the work is executed by subcontractors. The same is confirmed by Nobbs (1993), who reported the share of specialist and trade subcontractors to account for as much as 90 percent of the total value of construction.

Many important reasons underlie the observed trend of subcontracting in the construction industry. Arditi and Chotibhongs (2005) are of the opinion that “the operations of the average general contractor are not sufficiently extensive to afford full-time employment of skilled craftsmen in each of the several trade classifications needed in the field, nor is it feasible for these companies to own, operate, and maintain specialized equipment that may have only limited use during a project.” On

the other hand, Jamieson et al. (1996) ascribed the increasing subcontracting practice to the continuous increase in the complexity of building construction and organizational relationships. Furthermore, experienced subcontractors are generally able to perform the sublet work more quickly and at a lesser cost than the general contractor can (Arditi and Chotibhongs. 2005, Kumaraswamy et al. 2000). Nassar (2003) confirmed that profitability is the main reason behind the decision of buying out significant part of the work. Based on the above, general contractors have concentrated their efforts more on managing general site operations, doing an efficient procurement, and improving their subcontractors' selection and partnership practices than on hiring direct labor to execute trade or specialty construction work (Kumaraswamy et al. 2000).

Overhead Costs

The overhead category of cost is commonly estimated or expressed by contractors as a percentage of total direct costs, direct labor costs, direct labor hours, or direct labor costs and equipment expenses (Shelton and Brugh 2002). These costs are believed to vary between 8 and 30 percent of direct costs, or between 12 and 50 percent of the labor costs, depending on project parameters (Pulver 1989). The percentage of overhead costs to direct costs is reported to be more than 10 percent (Jones 1996). To be noted is that the unsteady nature of the construction market makes it difficult for contractors to choose the optimal level of overhead costs which allows them to win and manage large projects and at the same time make a reasonable profit (Assaf et al. 2001). Here comes the need to clearly define, estimate, and control all possible elements of overhead costs.

Study Findings

In this paper, the focus is on issues related to the project overhead costs component, which accounts for project and site expenditures which cannot be charged directly to a specific work trade or package. Historical overhead figures from six projects have been examined for the purpose of this study. These costs have been classified into four main groups as shown in Table 1. Costs related to the site supervision or representation teams of the consultant (Engineer) and client are shown in separate groups in Table 1, for classification purposes, as these can be highly dependent on the contract requirements spelling out the minimum to be furnished by the contractor in terms of such facilities and services (but grouped under Table 2 for statistical purposes). The effects certain project-related factors may have on the determination of overhead amounts, per item, have been assessed as being high (H), medium (M), low (L), or none.

Table 2 summarizes the calculated percentages of each overhead item amount to the amount for the overhead group that it belongs to (Item/Group) and of each overhead item amount to the total project overhead amount (Item/Total Overhead). Based on the results, but excluding Project 6 data (involving only concrete works for a special large spiritual facility), it can be deduced that there is a high degree of similarity in the proportional breakdown of overhead costs across projects at the item (within each group) level and at the group level.

Table 1. Project Overhead Costs Classification and Related Factors.

	Contract Duration	Work shift Schedule	Contract Amount	Requirements of Contract Nature & Complexity of Work	Site Layout	Other Factors
1. Contractor's Supervision Team						
Engineers & Site Office Administrative Staff	H	H	H	-	H	- % of subcontracted works
Field Site Staff	H	H	H	-	H	L
Indirect Labor	H	H	M	-	H	L % of subcontracted works & number of subcontractors
2. Site Facilities & General Expenses						
Mobilization & Demobilization						
Site Offices and Warehouse	L	-	-	-	-	H Contractor's supervision team size & space availability
Site Offices Equipment	L	-	-	-	M	- Contractor's supervision team
Site Offices Furniture	L	-	-	-	-	- Contractor's supervision team
Temporary Fence	L	-	-	H	-	H
Temporary Power & Lights	L	H	-	-	H	H Build-up area
Project Signs & Boards	L	-	-	H	-	H
Temporary Gates & Access Roads	L	-	-	-	L	H
Temporary Sanitary Facilities	L	-	-	-	-	H Volume of labor force
Installation of Equipment	-	-	-	-	M	H Number & type of equipment, build-up area, building height
Other Temporary Facilities & Set Up	-	-	-	-	M	H
Demobilization	-	-	-	-	-	H Mobilization
General Requirements						
Lab Tests & Studies	-	-	H	H	H	-
Safety Requirement	M	-	L	H	H	M Volume of labor force, building height
Site Clean-up (Without Labor)	M	-	M	M	H	M Site location, building height, build up area
Running Expenses						
Temporary Utilities	H	H	H	-	M	-
Stationary & photocopying	M	-	H	H	H	- Contractor's Supervision Team, volume of paper work
General Site Office Expense	H	-	L	-	-	- Contractor's supervision team
Equipment & Tools						
Equipment Ownership & Rental Costs	H	H	H	-	H	H Building height, build-up area, footprint area
Fuel & Oil	H	H	H	-	H	- Number & type of equipment
Maintenance & Repair	H	H	H	-	H	- Number & type of equipment
Shared Tools	H	-	H	-	H	-

(Continued)

Table 1. Project Overhead Costs Classification and Related Factors (Continued)

	Contract Duration	Work shift Schedule	Contract Amount	Requirements of Contract	Nature & Complexity of Work	Site Layout	Other factors
3. Engineer & Client Facilities							
Mobilization & Demobilization							
Site Offices	-	-	-	H	-	H	Engineer's team, availability of required space
Site Offices Equipment	-	-	-	H	M	-	
Site Office Furniture	-	-	-	H	-	-	
Running Expenses							
Temporary Utilities	H	H	H	H	M	-	
Stationary & photocopying	M	-	H	H	H	-	Engineer's Team, volume of paper work
General Site Office Expense	H	-	L	H	-	-	Engineer's Team
Special Requirements							
Staff	H	H	H	H	-	-	
Engineers Overtime	H	H	-	H	H	-	
4. Soft Overhead							
Insurance							
Contractor's All Risk (CAR.)	H	H	H	H	H	H	Total value of equipment , machinery, scaffolding, & formwork, site location, building height
Workmen's Compensation Policy	H	L	H	H	H	M	Volume of labor force , total salaries of the employees
Bonds & Guarantees							
Advance Payment Bond	H	M	H	H	-	-	Bank interest rate, amount of guarantee, registration stamps fee, Progress of work
Performance Bond	H	-	H	H	-	-	Bank interest rate, amount of guarantee, registration stamps fee
Authorities Levies & Charges	H	H	H	H	L	M	Government regulations
Contract Registration Stamps	-	-	H	H	-	-	
Financing Costs (Overdraft)	H	H	H	H	L	-	Cash flow, progress of work, retention, amount of advance payment, minimum amount of interim certificate, % of material on site, subcontractors and suppliers payment terms, relation with Engineer & Owner, payment certification and processing

A proposed mechanism for estimating and controlling overhead costs is presented in Figure 1. A continuous update of the project overhead costs database, taking into consideration all similar-type completed projects, is believed to be a prerequisite to any new estimation effort. Any significant changes to the maximum, average, or minimum percentages of overhead item amount shall be carefully studied and analyzed before accepting these changes into the database. After estimating overhead cost items based on a careful analysis of the requirements for the project on hand, the amount estimated for each item shall be examined as to whether it is within the applicable maximum and minimum limits shown in the updated database. For those items which are difficult to accurately estimate with a solid logic, historical percentage estimates can be adopted from the database by examining the records on similar-type projects previously undertaken.

After setting the budget in the bidding phase, the contractor shall distribute the budget of each overhead item over the construction life span, based on the type and time of occurrence of such a cost item, in order to periodically compare it with the actual cost. To this end, the actual spread of overhead expenditures over the construction duration was carefully examined and analyzed for two of the considered projects. The indication is that the actual spread of the total overhead cost over the length of the construction duration is almost linear (see Table 3) despite the fact that the actual spreads of some overhead items are not linear, when considered individually.

Periodical comparisons between actual overhead expenditures and budgeted figures are in order if effective control is to be exercised. The reasons underlying possible variances shall be investigated and their potential impacts on the possible future costs assessed. Costs anticipated at completion shall be projected and compared with the set budgets. In addition, the percentages of anticipated costs at completion for individual overhead items to the direct costs figures can be calculated and compared with the range of relevant percentages deduced from the updated database, to judge the severity of experienced variances.

Summary and Conclusions

The study reported herein has analyzed and studied project and site overhead costs, based on historical data from six building projects. It is concluded that: (1) about 47, 41, and 12 percent of project overhead costs can be attributable to the cost of the contractor's supervision team, site facilities and general expenses, and soft overheads, respectively. It should also be noted that engineers and site office staff, running expenses, and equipment and tools seem to represent the major overhead cost centers, which together account for about 55% of the total overhead amounts encountered. Another important observation is that the percentages of total overhead amount to the direct cost amount for the studied projects were almost equal (actual amounts not revealed herein for confidentiality reasons). This, coupled with the observed almost constant ratios per overhead component to the total overhead, can facilitate the calculation of the percentage of each overhead component amount to the direct cost amount. It is also concluded that the actual expenditure of total overhead cost versus time is almost linear in spite of the fact that the actual expenditure of some overhead items over time does not follow a linear form.

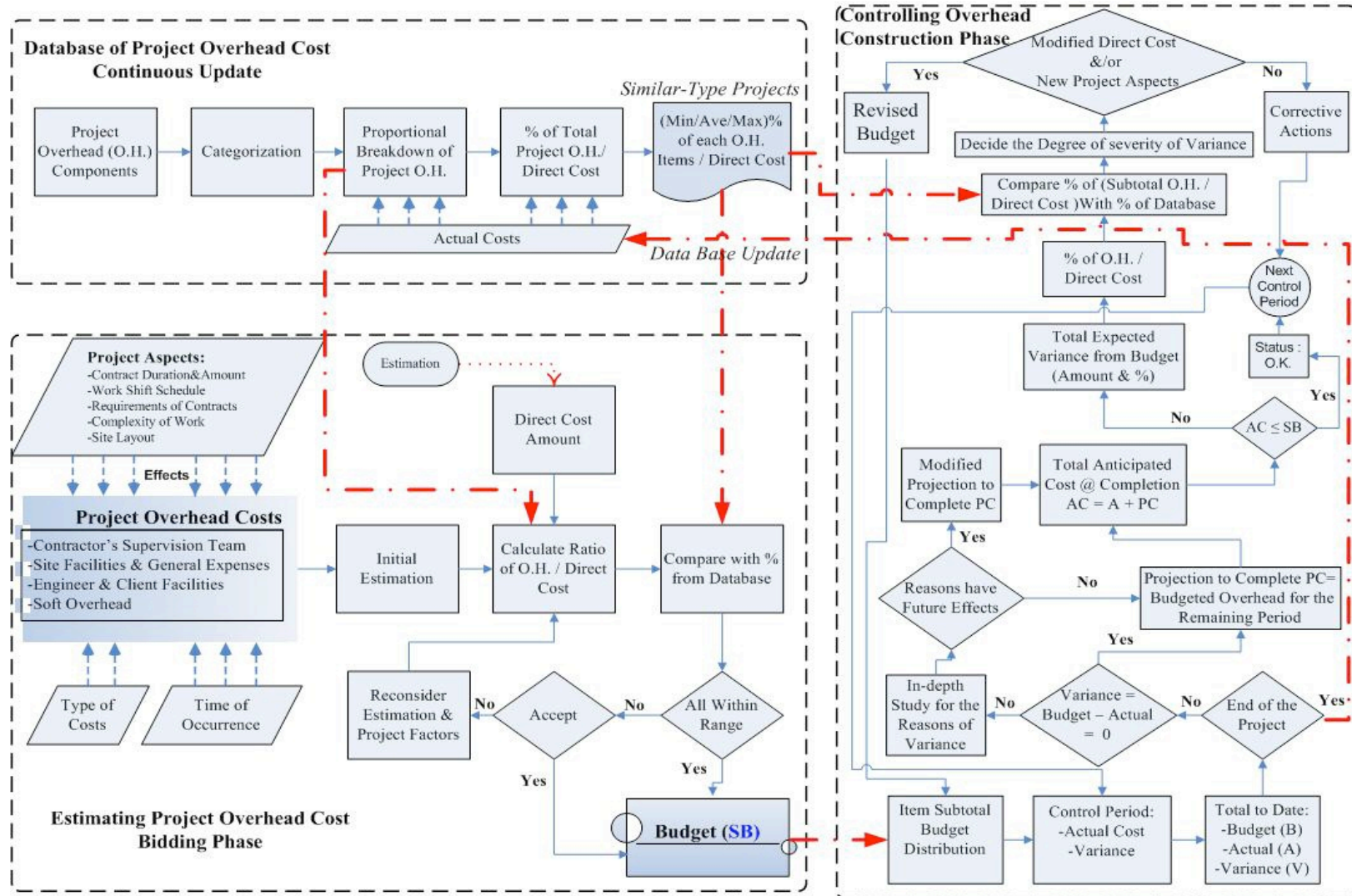


Figure 2. Process Model for Overhead Estimation and Control

Table 3. Distribution of Project Overhead Expenditures.

Percentages of Overhead Expenditure versus Time											
Percent of Time	Engineer	Staff	Unproductive Labor	Mobilization	General Requirements	Running Expenses	Equipment & Tools	Bonds & Financial	Stamp	Insurance	
10	5	10	5	60	10	5	10	15	100	55	
20	15	20	10	80	20	10	20	25	100	60	
30	30	30	15	80	30	15	30	35	100	65	
40	45	40	20	80	40	20	40	45	100	70	
50	60	50	30	80	50	25	50	60	100	75	
60	70	60	40	85	60	35	60	70	100	80	
70	80	70	55	85	70	50	70	80	100	85	
80	90	80	70	85	80	65	80	90	100	90	
90	95	90	85	90	90	80	90	95	100	95	
100	100	100	100	100	100	100	100	100	100	100	
Proportional Breakdown of Overhead	21%	12%	13%	4%	6%	16%	15%	6%	3%	4%	Total Overhead Expenditure vs. Time
10	1	1	1	2	1	1	2	1	3	2	14 %
20	3	2	1	3	1	2	3	2	3	2	23 %
30	6	4	2	3	2	2	5	2	3	3	31 %
40	9	5	3	3	2	3	6	3	3	3	40 %
50	13	6	4	3	3	4	8	4	3	3	50 %
60	15	7	5	3	4	6	9	4	3	3	59 %
70	17	8	7	3	4	8	11	5	3	3	70 %
80	19	10	9	3	5	10	12	5	3	4	80 %
90	20	11	11	4	5	13	14	6	3	4	90 %
100	21	12	13	4	6	16	15	6	3	4	100 %

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A Study of Preproject Planning and Project Success Using ANN and Regression Models

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ABSTRACT

It is long recognized by the industry practitioners that how well preproject planning is conducted has great impact on project outcome. Through industry project data collection and model analysis, this research intends to investigate the relationship between preproject planning and project success. In early stage of the project life cycle, essential project information is collected and crucial decisions are made. It is also at this stage where risks associated with the project are analyzed and the specific project execution approach is defined. To assist with the early planning process, Construction Industry Institute (CII) has developed a scope definition tool, Project Definition Rating Index (PDRI) for industrial and building industry. Since its introduction, PDRI has been widely used by the industry and researchers have been using the PDRI to collect preproject planning information from the industry. Scope definition information as well as project performance are collected and used for this research analysis. This research summarizes preproject planning data collected from 62 industrial projects and 78 building projects, representing approximately \$5 billion in total construction cost. Based on the information obtained, preproject planning was identified as having direct impact on the project success (cost and schedule performance). Two techniques were then used to develop models for predicting cost and schedule growth: statistical analysis, and artificial neural networks (ANN). The research results provide a valuable source of information for the industry practitioners that proves better planning in the early stage of the project life cycle have positive impact on the final project outcome.

Keywords: Preproject Planning, Project Success, Regression Model, ANN Model

1. INTRODUCTION

Preproject planning is “...the process of developing sufficient strategic information with which owners can address risk and decide to commit resources to maximize the chance for a successful project”(CII 1995). It is at this early planning stage that significant decisions are made by the project team. Preproject planning process constitutes a comprehensive framework for detailed project planning and includes scope definition. Project scope definition is the process by which projects are selected defined and prepared for definition. It is a key practice necessary for achieving excellent project performance (Merrow and Yarossi 1994) and is a key element in the preproject planning process. How well pre-project planning is performed will affect cost and schedule performance, operating characteristics of the facility, as well as the overall financial success of the project (Gibson and Hamilton 1994).

Inadequate or poor scope definition, which negatively correlates to the project performance, is among the most problems affecting a construction project (Gibson and Dumont 1996). The result of a poor scope definition is that final project costs can be expected to be higher because of the inevitable changes which interrupt project rhythm, cause rework, increase project time, and lower the productivity as well as the morale of the work force (O’Connor and Vickroy 1986). Success during the detailed design, construction, and start-up phases of a project highly depends on the level of effort expended during the scope definition phase as well as the integrity of project definition package (Gibson and Dumont 1996). Therefore, it is important to investigate the relationship between preproject planning and project success with real data from the industry. In order to measure the preproject planning efforts for each construction project, a scope definition tool, Project Definition Rating Index (PDRI) is incorporated in this research to evaluate the completeness of project scope definition.

In addition to preproject planning information collected using the PDRI, project performance (cost and schedule) information was also collected through the data collection process. Traditional statistical analysis method, Simple Linear Regression, and non-traditional statistical analysis method, Artificial Neural Network, were selected in this research to investigate the relationship between preproject planning and project performance using the sample project data.

2. SURVEY INSTRUMENT AND DATA COLLECTION

The data collection was accomplished through a series of retrospective case studies. A scope definition tool, Project Definition Rating Index (PDRI) is used as a survey instrument in these case studies to measure the preproject planning practices in the industry. Data from 62 industrial projects and 78 building projects, representing approximately \$5 billion in total construction cost, were collected and used to conduct an investigation of the early planning practices in the industrial and building industry.

2.1 Project Definition Rating Index

CII constituted a research team in 1994 to produce effective and easy-to-use pre-project planning tools so that owner and contractor companies would be able to better achieve business, operational, and project objectives (CII 1996). This research effort led to the development of the Project Definition Rating Index (PDRI). The PDRI for industrial projects is a weighted matrix with 70 scope definition elements (issues that need to be addressed in preproject planning) grouped into 15 categories and further grouped into three main sections. In responding to the needs of the building industry, CII developed the PDRI for Building Projects in 1999 (CII 1999).

The PDRI provides a means for an individual or team to evaluate the status of a construction project during preproject planning with a score corresponding to the project's overall level of definition. The PDRI helps the stakeholders of a project to quickly analyze the scope definition package and to predict factors that may impact project risk specifically with regard to industrial and building projects (Cho 2000).

2.2 Data Collection

In the PDRI survey questionnaires, specific questions were intended to obtain historical and "after the fact" project information. The questionnaires included questions regarding project basics (location, type, budget and schedule), operating information, and evaluation using an unweighted PDRI score sheet. Survey participants were asked to think back at a point just prior to construction document (detailed design) development when they filled out the PDRI evaluation score sheet. The total scores were then calculated based on pre-assigned element weights after the questionnaires were returned. Due to the unique nature of these two different

sectors, industrial and building projects were examined separately throughout this research investigation.

The sample projects used in this study were obtained from three different sources: previous PDRI research, CII Benchmarking and Metrics research, and institutional organizational (which prefers remaining anonymous) PDRI benchmarking research (Wang 2002). Nevertheless, it is important to note that the collected sample from these three sources is based on organization’s volunteering projects and not on a random sample of a known population.

Although the data from these three sources were collected by different researchers, the data collection methods remain consistent throughout the whole data collection period (1994 ~ 2001). That is, a questionnaire survey was used to collect the data and all the questionnaires are filled out by survey participants and followed up by researchers if necessary. In summary, information from a total of 62 industrial projects representing a total cost of approximately \$3.9 billion dollars was obtained for the research. In the meantime, 78 building projects representing approximately \$1.1 billion dollars in total budget cost were also collected. A detailed breakdown of the projects is presented in Table 1.

Table 1: Industrial and Building PDRI Data Collection

Sector (1)	Resource (2)	No. of Projects (3)	Represented Cost (Billion) (4)
Industrial	PDRI-Industrial Research (1996)	23	\$1.6
	Alignment Research (1998)	18	\$1.9
	CII BM&M Database (2001)	21	\$0.4
	Industrial Projects Total	62	\$3.9
Building	PDRI-Buildings Research (1999)	33	\$0.8
	Institutional Organization Benchmarking (2001)	45	\$0.3
	Building Projects Total	78	\$1.1

3. Data Analysis and Modeling

Specific questions in the PDRI survey questionnaires were intended to obtain historical and “after the fact” project information. The questionnaires included questions regarding project basics (location, type, budget and schedule), operating information, and preproject planning evaluation using a PDRI score sheet. Survey participants were asked to think back at a point of time just prior to construction document (detailed design) development when they filled out the PDRI evaluation score sheet. The total scores were then calculated based on pre-assigned element weights after the questionnaires were returned. Please refer to CII 1996 and CII 1999 for detailed development of PDRI element weights.

The PDRI score obtained from the survey is a good indicator of the level of preproject planning for each project. Two project performance aspects are of particular concern for this research: cost and schedule performance. Cost performance and schedule performance are measured by cost and schedule growth. In the survey, respondents were asked to provide estimated costs at the start of construction document development as well as the actual costs after construction completion. Total cost growth measures total project cost growth as a percentage of the initial predicted project cost. Cost performance was measured by project *Cost Growth* metric obtained as follow:

$$\frac{\text{Actual Total Project Cost} - \text{Initial Predicted Project Cost}}{\text{Initial Predicted Project Cost}} \quad (1)$$

The total project duration used to calculate project schedule growth was measured from the start date of construction documents development to the date of substantial completion in months. The following equation was used for computing project schedule performance, *Schedule Growth*:

$$\frac{\text{Actual Total Project Schedule} - \text{Initial Predicted Project Schedule}}{\text{Initial Predicted Project Schedule}} \quad (2)$$

Two different predictive models, regression model and ANN model, are selected for this research to investigate the relationship between the preproject planning and project performance using PDRI scores and cost/schedule growth.

Regression Model

The regression methodology models the distribution of a variable (dependent variable) with the help of one or more predictor variables (independent variable). Simple regression analysis involves only one predictor when investigating its relationship with the dependent variable. Though the simple linear regression accounts for only one predictor in modeling a dependent variable, in many situations, a linear function of X , or a suitably transformed X , will be a good first approximation of the true relationship (Montgomery and Peck 1982).

Using the PDRI score as the independent variable and cost/schedule growth for each project as the dependent variable, a simple linear regression analysis can be performed to examine the relationship between the independent variable (PDRI score) and dependent variable (cost/schedule growth). First, a scatter plot of the PDRI score and Cost Growth was constructed and a best fit line was then calculated and plotted on the scatter plot. The linear regression analysis is performed using SPSS 12.0. For industrial projects, the ANOVA results showed that Significant F equals to $9.5E-05$, which is less than 0.05 and indicates the linear relationship is statistically significant. That is, there is linear relationship between the PDRI score and Cost Growth for the surveyed industrial projects. In order to examine how the obtained linear regression equation represents the data, two sets of parameters, R and R^2 are calculated. Firstly, the linear correlation coefficient, R , measures the strength and the direction of a linear relationship between two variables. The obtained R value for this particular model is 0.475 , which indicate a relationship that as values for PDRI scores increase, the values for cost growth increase as well. Secondly, the coefficient of determination, R^2 , records the proportion of variation in the dependent variable (cost growth) explained or accounted for by variation in the independent variable (PDRI score). For this particular model, the obtained R^2 equals 0.23 which indicates that 23% of the variation in project cost growth can be explained by the variation in PDRI scores. Figure 1 shows the scatter plot and regression results for the industrial projects.

The analysis results show that the linear relation between PDRI score and project cost growth is statistically significant and PDRI score (completeness level of scope definition) can be used to explain a certain proportion of project cost growth. Similar results are observed for project schedule growth vs. PDRI score. However, it should be noted that many factors may influence the project after preproject planning and therefore, can contribute to cost overruns and schedule slippage such as poor

contract documents, unforeseen conditions, market conditions, strikes, and Acts of God, and so on. Nevertheless, the PDRI score still serves as a good indicator as to if the project is heading to the right direction in the beginning stage of the project life cycle.

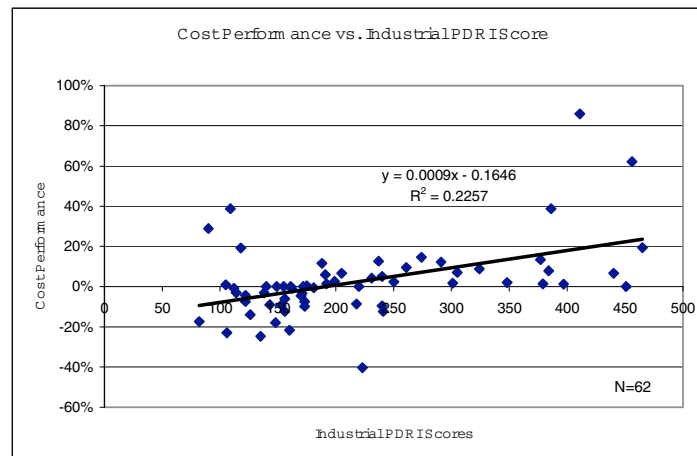


Figure 1. Simple Linear Regression: Cost Performance vs. PDRI Score

Artificial Neural Network Model

Studies have shown that ANNs have several advantages over the traditional statistical methods such as multiple regression analysis and multivariate analysis (Elhag and Boussabaine 2002). ANNs do not require that the data must follow a specific statistical distribution and does not require predetermination of the relationships between inputs and outputs. In addition, ANNs have very strong capability of self-learning and self-updating. Considering these advantages, this study has chosen neural networks for modeling project success.

The principle of Neural Networks is based on the assumption that a highly interconnected system of simple processing elements can learn complex interrelationships between independent and dependent variables (Elhag 2004). A typical neural network consists of an input layer, an output layer, and one or more hidden layers. These layers are connected by neurons to form a parallel distributed processing system. Each neuron is viewed as a processing element (PE) that receives inputs and generates outputs through an activation function. Each of the connections between the process elements has an associated weight. Figure 2 shows a typical three-layered neural network with an input layer (I), a hidden layer (H), and an output layer (O).

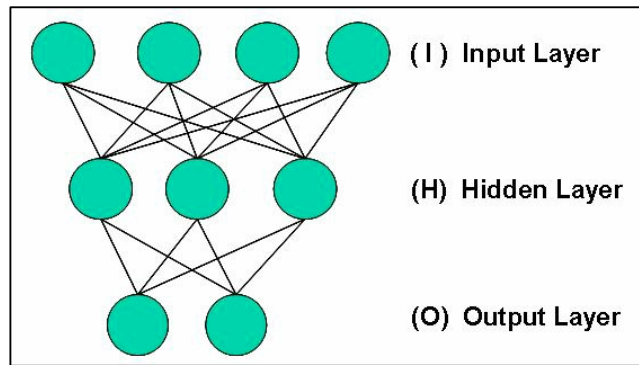


Figure 2. An example of a three-layered neural network

To develop the ANN model, a commercial software package *Neurosolution* was chosen for its ease of use (built-in with EXCEL), speed of training, and host of neural network architectures, including back-propagation with flexible user selection of training parameters. Data from 32 industrial projects are randomly chosen as the training set for the model. One project is used as cross validation to see if over-training has occurred. The model training process includes a total of three runs and each run with 1000 epochs. The mean square error for training and cross validation of this model is shown in Figure 3.

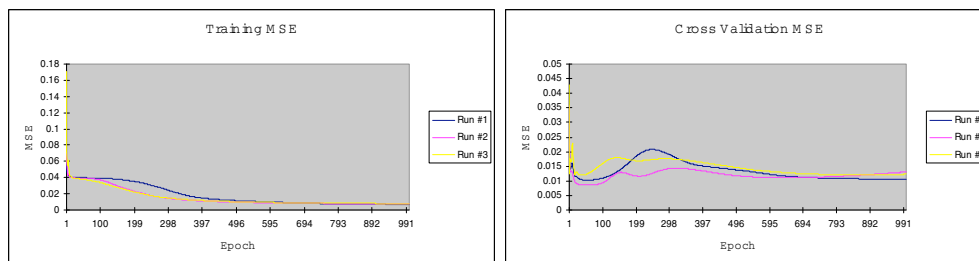


Figure 3. Mean Square Error for Training and Cross Validation

The best results (model with least mean square error) and the corresponding weights are kept for model testing. Nineteen projects are used to test the model with and Figure 4 shows the test results. The results show that the trend of the ANN outputs matches the trend of the actual cost. In addition, ANN predictions are close to the actual cost data except for some cases where actual cost growths are very high. The calculated coefficient of correlation is about 0.75, which indicates that the ANN model is better than the linear regression model.

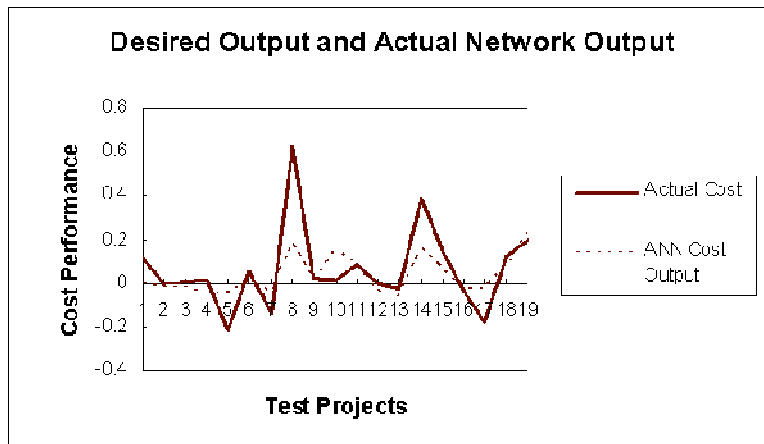


Figure 4. ANN Model Test Output

Conclusion

This paper studied the preproject planning of industrial and building construction projects and investigates its relationship with project success (cost and schedule performance). Questionnaire surveys were used to obtain information related to the status of preproject planning and project performance. The score obtained from the Project Definition Rating Index in the questionnaire was used to measure the completeness level of preproject planning for each surveyed project. This PDRI score was considered the independent variable in the model development. Also from the surveyed questionnaires, project Cost Growth and Schedule Growth were calculated as the dependent variable in the model. A scatter plot was first produced to examine the relationship between the independent variable (PDRI Score) and dependent variable (cost and schedule growth respectively). Based on the collected data, this research developed two models to predict the project performance (Cost and Schedule Growth individually) using the PDRI score. The first model was simple linear regression model and the second was artificial neural network (ANN) model. Data collected from a total of 62 industrial and 78 building projects were used for the model development. Both models show positive relationship between PDRI score and cost/schedule growth for this particular sample of projects. The results indicate that projects with better preproject planning are more likely to have a better project performance at completion.

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A Model to Predict the Impact of Excusable and Non-Excusable delay on Selected Construction Projects

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Delays are the most common and also the most costly problem on construction projects. If the construction time period is extended, higher borrowing costs, inflation and other commercial factors will increase the construction costs. And if the construction costs increase, from an economic perspective, the project may lack an acceptable and functional performance. Thus, the primary objective during the construction process is to complete the project on time and within budget, while meeting established quality requirements and other specifications. However, in some cases, delays during construction can create huge cost damages, and these damages can result in the projects' parties taking legal action against each other through a construction claim.

Historically, scheduling and finding the delays in big construction and commercial construction projects was based on discussion within an engineering department involved in the planning sector of the construction business. The reason for delay discussion is all about the effect of delays on the cost of construction. The amount of money involved in big construction is huge. And by extending the construction time, other expenses would be added to the original construction cost such as inflation, the cost of material and the interest costs of the investor's money.

With smaller construction projects like housing, given that the amount of money involved was small, few investors were serious about the postponement of the completion time of the project.

But today, the construction cost of a brand new high end house is more than in the past. In some places in the Toronto area, the cost of construction and land cost together become more than a few million dollars. This kind of cost causes investors to be far more interested in hiring a project scheduler to control the builder's time firmly in order to avoid the negative effects of the inflation cost of material and higher borrowing costs of the investor's money .

Usually, the claims for residential projects are premised on two concepts: firstly, the costs of the late practice of a project, which is usually determined at the same rate as inflation; and secondly, the profit acquired from the investment, which is counted as interest on the of amount of money stock in the project for the duration of time (delay) that the project is not ready for use. The costs resulting

from delays represent a percentage of the overall contract value, so, the cost of delay would be much greater in larger projects.

1. Objectives of Research

The goal of this study is to develop a linear and non-linear delay impact model that can be used to predict the impact of excusable and non-excusable delays on selected residential construction projects.

In order to reach the goal, several objectives were set:

- classify and categorize construction delays for selected construction projects;
- identify parties responsible for the delay;
- measure the impact of the delay on the project duration;
- introduce an impact model base on selected project's delays.

2. Scope of research

The scope of this research is limited to residential projects located in the City of Toronto, Ontario, Canada, and the period of study is between January 2003 and August 2007. Ten projects were selected for this study.

3. Background

Construction projects of all types, whether they be high-rise condominiums, home building or repairing a road, are all subject to completion dates. And yet, even with the most advanced project management techniques, construction projects constantly fail to meet deadlines, incurring delays and substantial extra costs. The excuses for these delays range from an Act of God to bad weather, material shortages, site conditions, and labour disputes, to name just a few. On top of this, analyzing the delays is further compounded by the fact that the causes of delays are often multi-faceted.

By choosing the correct method for scheduling the project prior to the beginning of construction, the risks of the costs of a delayed project are reduced. The judgment of an experienced engineer can help in choosing the right method for scheduling, and this in turn makes it possible to arrange suitable funding for each stage of the project.

Using the proper method for scheduling entails finding the right successor(s) for each task, finding the relation between the tasks, and predicting more realistic estimates of the duration of each task. Once the successor and relation between the tasks is determined, it will hardly ever change during the construction period. But, predicting the completion time is a completely different story, as the completion time depends on a number of conditions, and numerous

developments may change the time period and cause delays. Therefore, in order to more accurately predict the real time of completion, analyze the delays and their effect on the project schedule. In other word, the purpose of the analysis is to estimate the degree of time impacted due to the responsible party, and to determine the corresponding damages. In analyzing the time impact on the project completion date, many scheduling documents are prepared. To evaluate delays which are a result of a specific impact, schedule techniques such as critical path method or bar charts are normally used.

4. Construction delay

Delays have been met in the construction process since the concept of time has been introduced to the completion date of projects. There are two questions that must be answered when undertaking delay analysis. First, what is causing the delay? Secondly, who is liable for the delay?

In general, delays are cauterized by whether or not a time extension or additional compensation will be given to the contractor because of the delay.

4.1. Responsibility or liability of delay

The major part of delay analysis is finding liable parties in claim investigation. This is not an easy job for any Delay Claim consultant. There are three categories of delay related to liability; 1) Excusable compensable delays 2) Excusable non- compensable delays “Acts of God”3) Non-excusable delays. From the viewpoint of liability, one or a combination of the above situations may correspond to a delay.

4.2. Delay Cause

The most important aspect of delay analysis is to find the cause of the delay that has affected the critical path and the completion time of the project. Identifying the cause of delays can be a difficult task and is dependent on the project’s scale and the availability of construction records. The following table (table 4.1) presents the delay cause with delay source.

Table4.1: Delay Type-Delay Source

Delay Type	Delay Source
<p>Poor design</p>	<ul style="list-style-type: none"> • Defects in plan and specifications • Delay because of defects in shop drawings • Delay because of denial of construction

	<ul style="list-style-type: none"> material approval • Lack of information
Change order	<ul style="list-style-type: none"> • Order to stop work • Change orders in any construction stage
Weather condition	<ul style="list-style-type: none"> • Failure to provide heater on job site • Failure to provide cooling system on job site • Snow • Rain • Storm and blizzard
Site condition	<ul style="list-style-type: none"> • Failure to access to site • Stop order because the safety issues • Failure of equipment • Lack of supervision • Earthquakes • Fire and flood • Subcontractor performance
Late delivery	<ul style="list-style-type: none"> • Owner’s furnished materials not available • Due to war • Due to strikes • Late delivery by suppliers
Economic condition	<ul style="list-style-type: none"> • Failure to finance the project • Failure to provide the payments • Cash flow limitation • Bankruptcy
Upgrading	<ul style="list-style-type: none"> • Introduction of major changes in requirements • Upgrading work to certain manner • Upgrading the construction standard • Governmental acts and regulations

These seven delays types may be classified based on the major causes as shown in figure 4.1:

	Poor Design	Change order	Weather Condition	Site Condition	Late Delivery	Economic Condition	Upgrading
Excusable compensable							
Excusable non compensable							
Non-excusable							

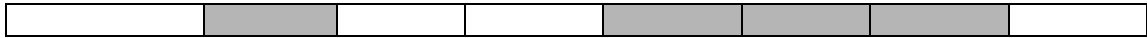


Fig 4.1: categories of delay (Liability) vs. Major Cause

If the source of delay affects only one category of delay, it is represented by a black cell. For example, the black colored cell for weather condition shows that the liability associated with this delay is always known as excusable non-compensable. If the source of delay affects more than one liability sector, it is represented by a gray cell. For example, site condition may present an either excusable non-compensable and/or a non-excusable depending upon the situation. A white cell means that the type of liable sector almost never applies to the major source of delay. For example, the white cell indicates that “excusable non-compensable” delays do not apply to delays due to economic conditions.

5. Result of delay analysis

The investigation of the selected projects in the last chapter shows that even with tasks that are repetitive, construction activities cannot be typical. Furthermore, we can find out how the delay, dependent on situation, can be different. So, the delay cannot be typical and repetitive.

Delay analysis gives two kinds of results:

- A result based on delay cause; and
- And a result based on a responsible party.

In conclusion, if we separate each cause delay by each of the projects, the following table (Table 5.1) and Figure 5.1 would present the percentage of delay causes.

Table 5.1: Amount of delay by cause

	Project	Poor Design	Change order	Weather Condition	Site Condition	Late Delivery	Economic Condition	Upgrading
		1	2	3	4	5	6	7
1	A	5	10	12	28	19	29	5
2	B	27	33	5	22	15	54	12
3	C	44	5	12	10	9	17	19
4	D	6	25	12	47	20	3	9
5	E	5	0	4	65	24	26	5
6	F	5	0	4	65	24	26	5
7	G	20	0	71	56	37	37	2
8	H	0	14	13	21	30	7	18
9	I	8	16	5	20	29	0	0
10	J	0	0	8	30	23	36	12
Total		120	103	146	364	230	235	87
Average		9.34	8.02	11.36	28.33	17.90	18.29	6.77

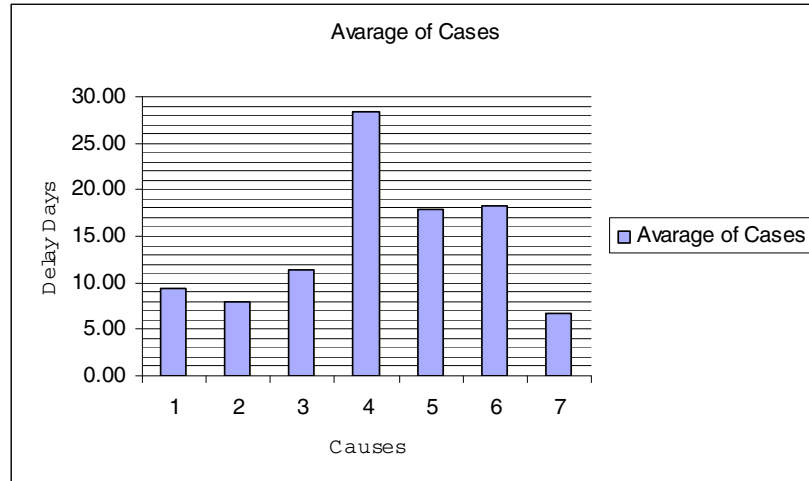


Fig5.1: percentage of delay cause

From figure 5.1 and table 5.1 the following information can be presupposed:

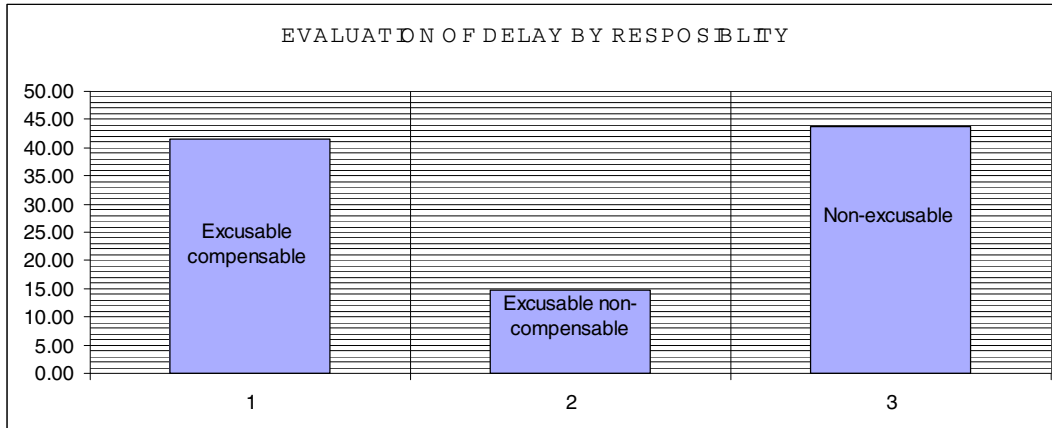
- Maximum delay cause would be happened by site condition with 28.33 percent chance,
- Second and third delay cause would be, economic condition and late delivery by 18.29 percent and 17.9 percent chance,
- And less chance for delay case would be upgrading by 6.77 percent.

It must be considered that the chance of occurrence of economic condition as a delay cause did affect by unexpectedly by the increasing metal price during 2006 and 2007. So, the result may be higher than usual for this cause.

Thus, if we conclude the percentage of each delay, responsibility is shown in the following table (Table 5.2) and Figure 5.2:

Table 5.2: Percentage of each delay responsibility

	Job Section	Excusable compensable	Percentage in project	Excusable non-compensable	Percentage in project	Non-excusable	Percentage in project	Percentage in all cases
1	Demolition & excavation & Inspection	5.00	7.14	5.00	20.00	9.00	12.16	11.24
2	Footing and Foundation	7.00	10.00	7.00	28.00	1.00	1.35	8.88
3	Back filling		0.00	1.00	4.00	1.00	1.35	1.18
4	Trusses order	1.00	1.43		0.00		0.00	0.59
5	Carpentry	5.00	7.14	3.00	12.00	7.00	9.46	8.88
6	Ext.Wall Construction	6.00	8.57		0.00	2.00	2.70	4.73
7	Roofing	1.00	1.43		0.00	1.00	1.35	1.18
8	Wiring(rough)	7.00	10.00		0.00	8.00	10.81	8.88
9	Plumbing(rough)	6.00	8.57		0.00	8.00	10.81	8.28
10	Heating(HVAC)(rough)	6.00	8.57		0.00		0.00	3.56
11	Pouring concrete of basement+ stair+ Draining	3.00	4.29	1.00	4.00	5.00	6.76	5.33
12	Insulation and vapor barrier	3.00	4.29		0.00		0.00	1.78
13	Dry wall	2.00	2.86		0.00	2.00	2.70	2.37
14	Typing		0.00	1.00	4.00	6.00	8.11	4.14
15	Flooring	1.00	1.43		0.00		0.00	0.59
16	Installing Kitchen and Vanity	4.00	5.71		0.00	5.00	6.76	5.33
17	Finish Plumbing	3.00	4.29		0.00	1.00	1.35	2.37
18	Installing interior doors + Finish carpentry	6.00	8.57		0.00	4.00	5.41	5.92
19	Finish Painting	2.00	2.86		0.00	7.00	9.46	5.33
20	Landscaping	1.00	1.43	7.00	28.00	7.00	9.46	8.88
21	Pre delivery inspection	1.00	1.43		0.00		0.00	0.59
	Total case	70.00	100.00	25.00	100.00	74.00	100.00	100.00
	Cases percentage		41.42		14.79		43.79	



5.2: Percentage of each delay responsibility

From figure 5.2 and table 5.2 the following information can be assumed:

- Maximum delay would be happened as non excusable with 43.79 percent chance,
- Second responsible party for delay with not much difference from first place would be excusable compensable by 41.42 percent,
- And less responsibility for delay would be excusable non-compensable by 14.79 percent.

It must be believed that the chance of occurrence of non excusable delay cause is almost equal to excusable compensable. In other words, the owner of project was responsible for delays as much as the contractor of the project.

Furthermore, the weather condition as an excusable non-compensable delay can be recognized by simulation from the past. By simulation of past weather conditions, the risk of this kind of delay may be reduced.

6. Conclusion of analysis and modal impact factors

In Table 6.1, the study will include the reliability-cause relation box by percentage. In this table, both percentage of reliability and delay cause have been presented in one box, and for conclusions, the total percentages will be offered in the end of the columns and rows.

In Table 6.2, there is a study focus on reliability-cause relation box by delay ratio. In this table, the same as Table 6.1, a combination of two kinds of ratio information are offered in one box. In the end, the total ratio offers are in the end of the columns.

Table 6.1: Reliability-cause relation box by percentage

	Poor Design	Change Order	Weather Condition	Site Condition	Late Delivery	Economic Condition	Upgrading	Total Percentage
Excusable compensable	0.25	9.50	0.00	2.10	15.34	12.28	6.16	45.64
Excusable non-compensable	0.00	0.00	10.38	2.50	0.68	0.29	0.00	13.85
Non-excusable	8.92	0.00	0.00	22.91	1.28	5.70	0.97	40.51
Total Percentage	9.17	9.50	10.38	27.51	17.30	18.27	7.13	100.00

Table 6.2: Reliability-cause relation box by delay ratio

	Poor Design	Change order	Weather Condition	Site Condition	Late Delivery	Economic Condition	Upgrading
Excusable compensable	1.00	1.10	1.00	1.02	1.16	1.01	1.08
Excusable non-compensable	1.00	1.00	1.11	1.03	1.01	1.00	1.00
Non-excusable	1.12	1.00	1.00	1.25	1.02	1.07	1.01
Total Ratio	1.12	1.10	1.11	1.29	1.18	1.08	1.08

Interesting results can found on Table 6.1:

1. Excusable compensable delay is covering 45.64 percent of all delays, which in Chapter Four was calculated as 41.42 percent.
2. Excusable non-compensable delay is covering 13.85 percent of all delays, which in Chapter Four was calculated as 14.79 percent.
3. Non-excusable delay is covering 40.51 percent of all delays, which in Chapter Four was calculated as 43.79 percent.
4. Poor Design as a delay cause covers 9.17 percent of all delay causes.
5. Change Order as a delay cause covers 9.5 percent of all delay causes.
6. Weather Condition as a delay cause covers 10.38 percent of all delay causes.
7. Site Condition as a delay cause covers 27.51 percent of all delay causes.
8. Late Delivery as a delay cause covers 17.30 percent of all delay causes.
9. Economic Condition as a delay cause covers 18.27 percent of all delay causes.
10. Upgrading as a delay cause covers 7.13 percent of all delay causes.
11. Site Condition as a delay cause still has the highest chance to make delays.
12. Upgrading as a delay cause has the lowest chance to make a delay
13. Economic Condition as a delay cause, because of unexpected metal price changes, has the second highest chance to make a delay.
14. Late Delivery as a delay cause is still in third place for the chance of making a delay. If the commercial effect on economic condition is eliminated, late delivery would be the second cause for delays.

If model factors effect on total time period of project, the result of effective schedule will be linear (figure 6.1). And if model factors affect on each task of project, the result of effective schedule present as non-linear (figure 6.1).

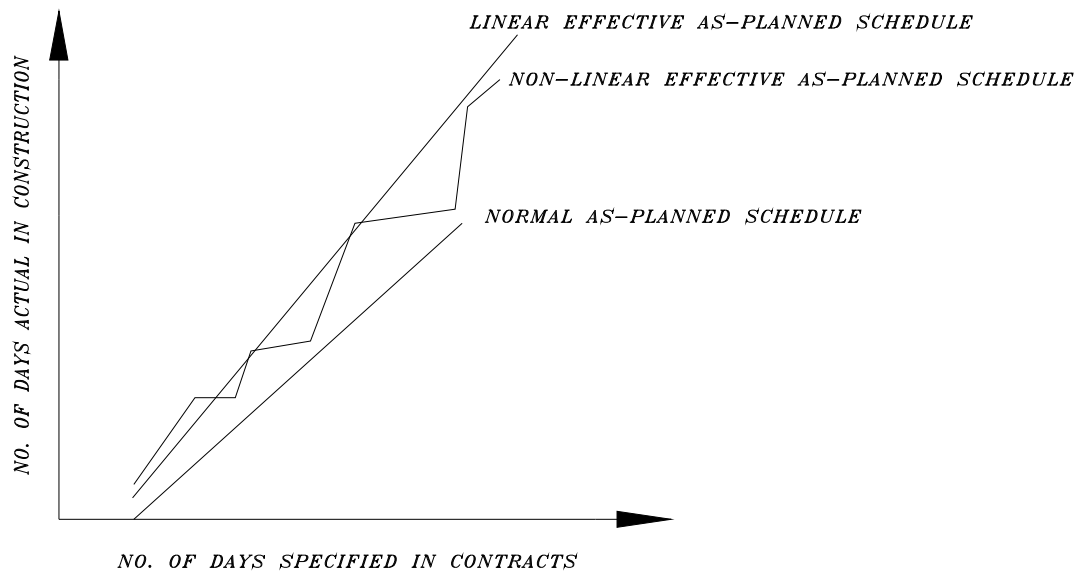


Fig 6.1: Effected as-planned schedule by modal factors

Even though the results developed on Table 5.3 can be accepted as a developing model to predict the impact of delay, the estimated cost and time depends on an engineering judgment methodology. And, using the factors needs engineering judgment. For example, the weather condition may have an effect on the mechanical part of construction which no one assumes, like when a furnace does not has access to a ventilation system because of snow. So, by changing the conditions using the model things will be different.

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Applying MD CAD Model to Streamline Information Transformation for Construction Project Planning

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Abstract: Information processing is critical to the successful development and execution of a construction project plan. Currently, Building Information Modeling (BIM) has been widely used in representing the physical and functional characteristics of design and construction. However, in order to use BIM effectively in construction project planning, more construction knowledge should be transformed and integrated. This paper proposes an activity-based modeling approach to process project information in an organized manner and proposes a Multi-Dimensional (MD) CAD model. The MD CAD model is composed of the MD CAD objects. The model can assist project planners in assessing information concerning the inter-relationships between schedule, cost, resources, and work areas. A computer implementation called MD-Construction Project Management Information System (MD-CPMIS) is presented to verify the feasibility of the proposed approach.

Keywords: MD CAD Model; Project Planning; Building Information Modeling

1. Introduction

In construction project planning, information processing is perhaps the most challenging task faced by project teams. As a construction project increases in size or complexity, the efficiency and accuracy of information processing between different project participants have a tremendous impact on the successful development and execution of a construction project plan. The planning process requires the transformation of data into decisions and actions. Information required for the execution of a project needs to be extracted from design and construction data. This information is then processed to formulate project knowledge necessary for making appropriate decisions and actions.

In practice, the implementation of information processing by most project teams is substantially manually. The project team needs to study the 2D design drawings and contract documents, and investigate the site in order to further develop plans on budget, schedule, and so on. Furthermore, these project plans are generally created

independently of one another. This respective approach becomes a heavy burden to the project team due to the tremendous amount of information that must be manually pieced together to integrate a comprehensive plan (Waly and Thabet 2002). For example, it is still very difficult to integrate information from budget and schedule plan. Thus, it is hard to take account of both cost and schedule impact during the decision-making process.

Along with current development of computer technology, various research efforts attempt to develop innovative ways of processing project information. Embarking on the advancement in data standards, CAD systems and information systems, many researchers have tried to streamline the information processing tasks by modeling and standardizing the information required for design, construction and operation of constructed facilities. The Building Information Modeling (BIM), with its data-rich digital representation, is a new approach to cataloging the physical and functional characteristics of design and construction (GSA 2006; NIBS 2007). However, to use BIM effectively in construction projects, the information processing between different participants in the construction process has to be more efficient (buildingSMART Norway 2008). In addition, more construction knowledge and project information should be implemented in BIM to assist project planners to effectively process construction documents into project plans.

This paper starts with analyzing the discrete processes undertaken within building construction in order to present the information of construction activities and specifications in the BIM model. These essential characteristics of the work items and building components are defined in the property sets of the proposed MD CAD objects (Feng and Chen 2005). These graphic objects are then utilized to create the MD CAD model, which will be further exported as a project database. In order to transform these project data into the information for project planning, this research employs an activity-based modeling approach as a mechanism of information transformation. In the transformation process, the scope hierarchy structure is availed to define the activity items in a consistent formation. This organized manner assists in integrating project information into a standard format for cost estimating, project scheduling and project control. Since the MD CAD objects are created based on work items, which contain the information related to the required resources including labor, machine, and material. These objects facilitate the process of quantity takeoff. In addition, the proposed MD CAD model system can assist project planners in assessing information concerning the inter-relationships between schedule, cost, resources and work areas.

In the first section of this paper, the development of Building Information Modeling (BIM) is elaborated. Next, the procedures of developing a MD CAD model system are described. Finally, a computer implementation MD-Construction Project Management Information System (MD-CPMIS) is presented to verify the feasibility of the proposed approach.

2. Building information modeling

CAD systems have been widely used in the AEC industry and have evolved over the years. However, the CAD graphic documents often exclude information needed for effective project planning. The information that is sufficient for project designs is often insufficient to meet the requirements of project planning. In addition, information

fragmentation within project documents has always been an obstacle to information sharing and exchange among different participants and different phases.

Building Information Modeling (BIM) is known as an innovative approach to modeling building information in a 3D object-oriented model. The purpose of BIM is to visualize the physical and functional characteristics of design and construction, so that the design intent and program can be easily understood and automatically evaluated via its digital representation. Currently, BIM has been recognized to be essential in AEC industry for its capabilities of managing, sharing and exchanging information among project participants throughout the project lifecycle. As a data-rich shared resource, BIM can lighten the load of re-gathering or re-formatting information, by classifying and standardizing design and planning information, which usually fragmentally exists in a variety of phases and aspects of a building project. Besides, the International Alliance for Interoperability (IAI) (IAI 2008) is one of the initiatives for collaborative working in AEC industry. IAI has developed Industry Foundation Classes (IFC) as a standardized data structure for representing information used in BIM. Currently, the IFC data model has been widely adopted in the developments of BIM applications. The model defines an integrated schema to represent the main physical and logical building objects, including their characteristics and their inter-relationships in the form of a class hierarchy.

In the development of model-based information integrating systems, however, research efforts are still focused on the integration of design-related information instead of the actual requirement of information management in the construction phase (Zaneldin et al. 2001). From a project planning perspective, the ideal world would be the one in which an architect and a project contractor are able to exchange building information model data between their applications in a seamless fashion. Although the model schema of building product is now available, standards in construction management domain still need to be defined. The IFC hierarchy covers the core project information such as building elements, the geometric and material properties of building product, project costs, schedules and organizations. However, the current IFC is still incapable of providing sufficient information required by the construction activities (Chau et al. 2005). Thus in facilitating project planning, more construction knowledge and project information should be implemented in BIM to effectively process construction documents into project plans. As a core of information integration for project planning, an ideal integrated BIM does not merely consist of physical components of a building. The information of construction and management activities linked to the relevant building components should be described as well. In addition, the relationships among the building components and activities are also crucial to an integrated BIM (Fu et al. 2006).

3. MD CAD model system

3.1 The framework of MD CAD model system

The proposed MD CAD model system will be developed as a computer system, which will focus on conducting project information and decision supports for project planning. The MD CAD model will be adopted as core project information repository, where initial project data from various design/contract documents will be retrieved and

further transformed into the information required for the execution of the project.

The diagram in Fig. 1 shows the framework of the MD CAD model system. According to this diagram, the multidisciplinary data generated by various project participants is collected in pre-construction phase. These data are then used to create the MD CAD objects an IFC-based CAD system. The MD CAD model is composed of these MD CAD objects. In order to present the information of construction activities and specifications in the MD CAD model, this paper starts with analyzing the discrete processes undertaken within building construction. These essential characteristics of the work items and building components are defined in the property sets of the MD CAD objects. After the creation of the MD CAD model, the information described in the MD CAD model will be exported to a project database. Thus, all project-required work can be identified on the basis of collected work items in the MD CAD model.

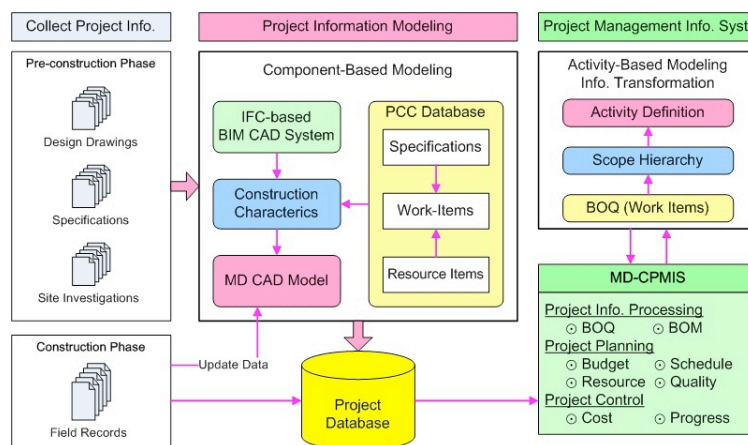


Figure 1. The framework of MD CAD model system.

In the model, even though the work items described in the MD objects can present the construction contents, they are still incapable of generating information on activity items. This limitation arises because the relationships between work items and activity are not established. Moreover, since the component-based modeling approach defines the components' structure in BIM model from the perspective of building design, the graphics objects used in BIM can only be used to show the ultimate appearance of the designed building. In order to transform the project data into the information for project planning, this research proposes an activity-based modeling approach as a mechanism of information transformation. In this transformation mechanism, a scope hierarchy structure is established to define the activity items in a consistent formation. This organized manner will assist in integrating project information into a standard format for cost estimating, project scheduling and project control.

In the MD-CPMIS system, project information such as BOQ (Bill of Quantity), BOM (Bill of Materials), and resource requirements can be produced efficiently and accurately to support future budget and schedule planning. In addition, during the construction phase, the filed records are recorded in the MD-CPMIS to assist the project team in cost and schedule control. Based on the project performance evaluations, the project plans can be modified according to the performance indexes

such as CPI (Cost Performance Index) and SPI (Schedule Performance Index).

3.2 The creation of MD CAD model

A MD CAD model is an extension of a BIM model, which incorporates multi-aspects of construction information required for project planning and management. The attributes of a MD CAD object are divided into geometric, containment hierarchy, and behavioral aspects. Figure 2 demonstrates the structure of a MD CAD object's attributes.

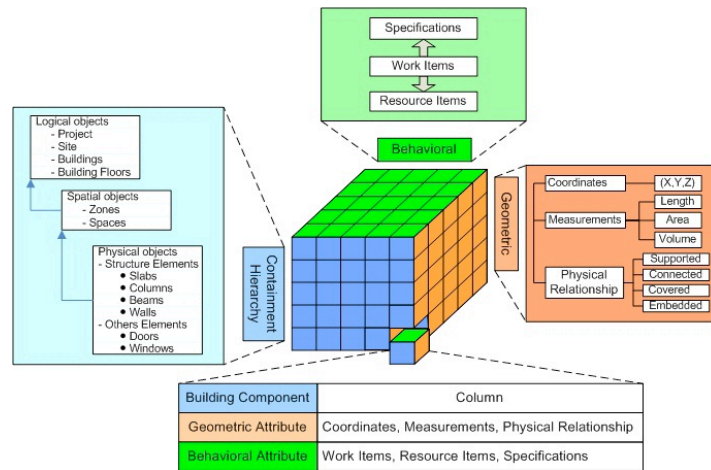


Figure 2. The structure of a MD CAD object's attributes.

- In geometric aspect, the attributes include the coordinates, measurements, and physical relationships of a building component. This information is obtained from the BIM CAD system. The measurements information will be utilized as a reliable source in quantity estimation for a project. The physical relationships between building components can be used to generate an optimal sequence of construction, this sequence can be further served as a basis on scheduling (Feng and Chen 2006).
- In the containment hierarchy, the MD CAD objects are classified into physical, spatial, and logical objects. The physical objects include all the basic building elements, such as slabs, columns, beams, walls, doors, windows, and etc. The spatial objects include spaces and zones. Generally, a space is bounded by several building elements, and a zone is consisted of spaces. Within the structure of logical objects, a building floor contains the physical and spatial objects, a building contains the building floors, a site contains the buildings, and a project contains the sites. This containment hierarchy defines the model structure of the MD CAD model.
- In behavioral aspects, the construction contents are described by the work items in an appropriate level in the containment hierarchy (as shown in Figure 3). The work items employed in this research are classified and encoded according to a hierarchical standard code. Based on the standard code of a work item, the information of the corresponding specification and resource items can be

obtained. In this research, the Public Construction Master Codes announced by the Public Construction Commission (PCC) in Taiwan (PCC 2007) are adopted as a work item database. The PCC Master Codes are developed with reference to CSI MasterFormat of the United States. By applying the master codes of work items, the information of specifications and resource items in the PCC database may be applied in the MD CAD model. Moreover, project teams can establish their own construction information database and develop knowledge management system by using the PCC's codes.

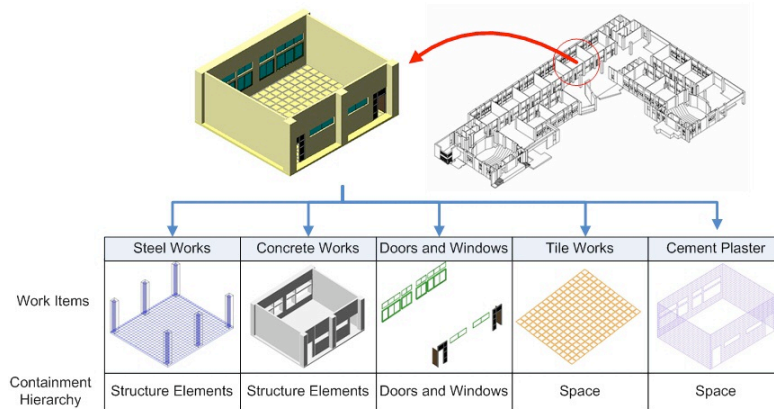


Figure 3. The analysis of construction contents.

3.3 The information transformation

A common technique used to understand and organize complex tasks is to break the project-required work into smaller pieces (divide into subparts). In construction, this technique is applied in both planning and estimating. This step is accomplished by separating construction into its incremental parts. They are referred to as construction activities in this research. To create a construction plan, all of the activities necessary to accomplish the project are first identified. Activity descriptions should be concise and unambiguous. The description communicates the scope and location of the activity. In practice, activity items are seldom listed in the contract documents, but are necessary for evaluating requirements and developing project plans. At the most detailed level, each activity item is usually related to and performed by a crew. The planner develops the activity description by defining the type of effort required to construct a work item. To complete a work item, multiple activities may be necessary. Activity descriptions should be as complete and accurate as possible to lend credibility to the project planning and aid in later review and analysis.

In order to generate the information for project planning, this research proposes an activity-based modeling approach as an information transformation mechanism (as shown in Figure 4). All project-required work could be identified on the basis of collected work items in the MD CAD model. In order to define the activity items in a consistent format, a scope hierarchy structure, which is a hierarchical breakdown of the activity scope is established for this purpose. It provides a common, ordered hierarchy framework for summarizing information and for quantitative planning and

management. Thus, the activity items could be consisted of the following levels of detail: element, space, zone, floor, building, site, and project, which are corresponding with the levels in MD CAD model structure. After the project-required work is analyzed and broken into the activity items, each activity item can then be quantified prior to project planning. As depicted in Figure 4, the activity items in 1F_structure_engineering may include: 1F_Z1_Structure Steel Works, 1F_Z1_Structure Form works, and 1F_Concrete Pour Work. According to the work item breakdown analysis, the 03210_42001 Reinforcing Steel can be further divided into resource items such as materials (M0321042031 product, steel bar, SD28, $f_y=28\text{kgf/mm}^2$), labor (L7122090002 steel bar worker), and equipment (E3814005001 crane, 50~59t). Also, the Master Codes of the work items 03210 can be used to link the specification of Reinforcing Steel.

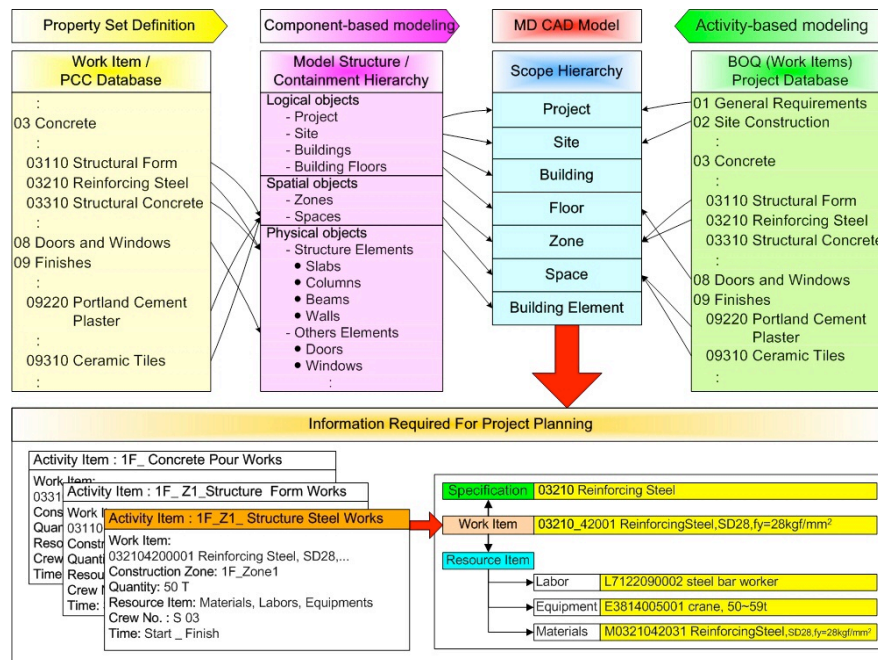


Figure 4. Activity-based modeling process.

4. Computer Implementation

This research applies the AutoCAD Architectural 2008 for the creation of the MD CAD model. In order to facilitate the process of creating project's MD CAD model, the prototype of the MD CAD objects are systematically saved in a MD Object Database. The diagram in Figure 5 demonstrates the development process of a MD CAD model. After the MD CAD model is created, the data collected by the MD CAD model will be exported to a database file, which will further be applied in the MD-CPMIS system to provide better information integration for project planning and control. There are five major sections in the MD-CPMIS system:

1. *General Information:* In this section, the general information of the project such as project name, project number, site address, construction date, and contract total cost is recorded.

2. *BOQ and BOM* (as shown in Figure 6): The quantity take-off is an important part of the cost estimate and schedule planning. In this section, based on the project database imported from the MD CAD model, project information such as BOQ (Bill of Quantity), BOM (Bill of Materials) and resource requirements can be efficiently and accurately produced. The activity items can be identified on the basis of the project-required work items. Descriptions are created to describe the scope and material requirement for each activity item. The unit cost for each activity item is developed as a direct cost with separate cost for the labor, equipment and material components. The number and detail of the listed activities will vary from job to job and will depend on the intended level of planning. This detailed review enables the planner to formulate a construction sequence and duration.
3. *Project Scheduling* (as shown in Figure 7): In project planning, the duration of an activity is estimated according to both the quantity of work to be done and the work production rate. The work production rates are determined based on the labor and equipment used to perform the task. The system supports the modification of the work production rate to allow a more flexible planning. MS Project is used for scheduling; and the outcome will then be transferred back to the system. In addition, information of the cash flow analysis is also provided in this section.
4. *Site Record*: The system is capable of generating the daily filed records for tracking the construction work. The information includes the completion of work items, resource consumption and cost.
5. *Project Performance* (as shown in Figure 8): Effective planning requires continuous monitor so that the manager can make forecast and revise plans to maintain the proper course toward the objective. In this section, the data obtained from “Project Scheduling” and “Site Record”, such as weekly scheduled completion, budgeted cost, actual completion, and actual cost, will be used in earned value analysis. The cost and schedule variances assist in evaluating and controlling project risk by measuring progress in monetary terms.

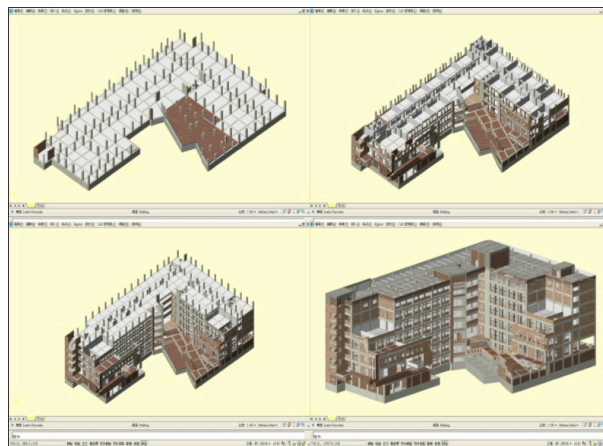


Figure 5. The development process of a MD CAD model.

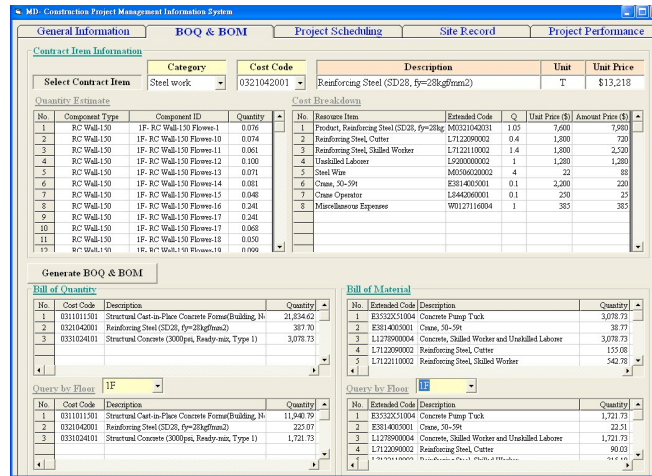


Figure 6. The “BOQ and BOM” section in MD-CPMIS.

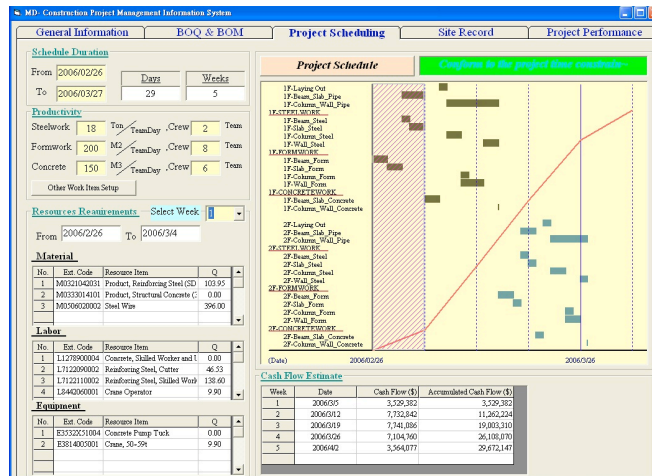


Figure 7. The “Project Scheduling” section in MD-CPMIS.

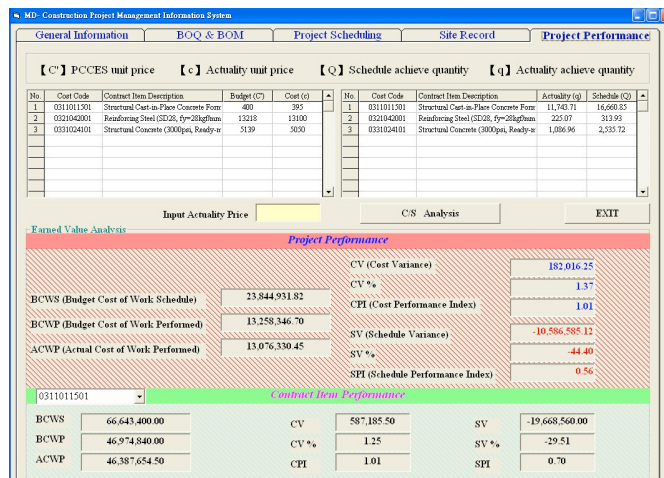


Figure 8. The “Project Performance” section in MD-CPMIS.

5. Conclusions

This paper presents a model-based system that employs MD CAD model to streamline information transformation for construction project planning. The MD CAD model consists of MD CAD objects, ensures the consistency and efficiency of information integration. In addition, extending the functionalities of BIM, this research proposes an activity-based modeling approach to define the activity items in a standard format for cost estimating, project scheduling, and project control. The MD-CPMIS system has demonstrated the efficiency and feasibility of the proposed approach in project planning and control. In the future, the results in this research will assist in developing an automating planning system.

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Automated Front-End Planning for Cost and Schedule: Variables for Theory and Implementation

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Abstract

The reliability and accuracy of ‘preliminary estimate’ and ‘preliminary schedule’ as part of front-end planning is mainly affected by the ‘specific characteristics of a construction project’, ‘project details determined at the time of planning’, ‘experience of the planners’, and ‘historical database of similar projects’. The purpose of this paper is to evaluate the variables that critically influence the viability of automated front-end planning tools. A framework of variables is first proposed in order to evaluate the comprehensiveness of the system objectives. Because there have not been many efforts to develop this type of tools, this study introduces recent real-world cases of the automated front-end planning system. Finally, findings from the entire research process are briefly summarized, and implications and lessons learned for further development are outlined as well.

Introduction

In an attempt to measure the relationship between “preproject-planning-effort and cost and schedule performance of a project”, Hamilton and Gibson (1996) stressed the significance of “well-organized and formal preproject planning” effort. Their statement is truly applicable to any project, especially in terms of improving the performance of a project in terms of cost, time, and quality.

One of the major difficulties involved during the preproject planning effort is to predict the cost and time without having drawings and details at the very beginning stage of preproject planning phase. For example, an owner may want to have an estimate and a schedule of a facility even before they decide the location, exact capacity, or other requirements. More critical fact, in this example, is that the owner may want to sign a design-build contract with this rough estimate and rough schedule. Furthermore, the owner can require what-if simulations.

Possible solutions to cope with the situation described in this example may include formulating a task force with experienced engineers, analyzing historical database, or developing information systems for repeating projects. Developing

information systems (IS) would facilitate to improve the accuracy, punctuality, and organizational learning. However, developing and maintaining information systems for a single ad hoc business function for a specific type of project would raise an issue of effectiveness in terms of utilizing corporate resources, making standard processes (in order to collect historical database), and validating the benefits.

In this context, the purpose of this paper is to evaluate the major variables that influence the viability of automated cost and schedule planning systems. These variables may serve as the most important factors for designing automated front-end planning systems in an effective way. This study focuses on the use of labor productivity as the most important parameter for delivering the lessons-learned into the future project in a structured method. 'Level of details' and 'workload' is thoroughly evaluated in employing the parameter of 'labor productivity'. This parameter will be discussed within the influencing variables.

Variables of Information Systems for Preliminary Estimate and Schedule

The information systems assessment (ISA) issues with quantitative measures have been discussed in a limited number of literatures in the construction industry. As one of the early efforts, Betts (1995) developed a five-level conceptual framework for strategic information systems (IS) from an '*industry-level*' perspective. The five levels include the national construction industry, professional institutions, construction enterprises, construction projects, and construction products. Stewart and Mohamed (2004) and Jung et al. (2004) developed two independent 'comprehensive and detailed frameworks' from the industry-level perspective by quantitatively evaluating the major factors affecting effective IT utilization in the industry.

In order to evaluate the benefits from IS on a '*company-level*' perspective, Jung and Gibson (1999) proposed a comprehensive and quantitative assessment methodology that primarily identifies the most promising areas. The variables used in their study include corporate strategy, management, computer systems, information technology, and incremental investment as listed in the Case B of Table 1. Based on the corporate level IS planning methodology developed by Jung and Gibson (1999) that encompasses corporate-wide issues across all business functions, Jung et al. (2006) proposed a '*domain-level*' knowledge management systems (KMS) planning methodology. The variables used in the KMS planning consist of frequency, explicitness, organizationality, and importance, which are more specific and technical. Another IS planning methodology (Jung et al. 2008) provides an assessment tool for evaluating safety management on a '*business function-level*'. It is noteworthy that different variables (e.g. role and responsibility) are used in order to meet the distinct requirements of safety management.

Table 1. Variables for Information Systems in Construction

Research	Perspective	Major Variables	Objectives
Case A Jung et al. (2004)	Informatization <i>(Industry Level)</i>	Infrastructure Utilization* Support	Measuring the level of informatization
Case B Jung & Gibson (1999)	Information Systems Planning <i>(Corporate Level)</i>	Corporate strategy* Management* Computer systems* Information technology* Incremental investment*	Prioritizing the most promising areas for information systems
Case C Jung et al. (2006)	KMS Planning <i>(Domain Level)</i>	Frequency Explicitness Organizationality Importance	Identifying strategic areas for organizational knowledge management
Case D Jung et al. (2008)	Safety Management <i>(Biz Function Level)</i> <i>(Corporate Specific)</i>	Role and Responsibility Specific characteristics Degree of Importance Utilization	Identifying key safety tasks those can enhance the overall performanc e
Case E Jung & Kang (2007)	Progress Management <i>(Biz Function Level)</i> <i>(Product Specific)</i>	Level of Detail* Workload* (Minimized) Accuracy* Budget weight, Duration, Method Sustainability *	Applying standard measurement packages Automatically generating initial WBS Accumulating progress knowledge
Case F This Study	Preliminary Cost Estimating <i>(Biz Function Level)</i> <i>(Project Specific)</i>	Level of Detail Workload (Optimized) Accuracy Labor productivity Adjustment factors Sustainability	Automating preliminary cost estimate process Generating estimate details based on historical database
Case G This Study	Preliminary Scheduling <i>(Biz Function Level)</i> <i>(Project Specific)</i>	Level of Detail Workload (Optimized) Accuracy Labor productivity Adjustment factors Sustainability	Automating preliminary schedule generation Generating duration and activity details based on historical database

* The variables with an asterisk are discussed and applied to this study (Case F and G).

A standard progress management system developed by Jung and Kang (2007) raises another issue of ISA variables (Case E in Table 1). The objectives of the system are to provide a standard measurement package, to automatically generate project WBS, and to accumulate the progress management knowledge for the same type of construction projects (e.g. office buildings). Therefore, it is '*business function-level*' (i.e. progress management) and '*product specific*' (i.e. all office buildings). In this case, the variables for corporate-level IS planning are still inherited, but 'business function-level product-specific' variables are focused in relation to 'making standards' for historical database, 'alleviating workload' to collect and maintain data, 'enhancing accuracy' by properly defining manageable packages, and 'sustaining adaptability' in order to have self-evolving mechanism (Jung and Kang 2007).

From the '*business function-level*' perspective, selecting variables for information systems for preliminary cost estimate and schedule (Case F and G in Table 1) must be in a similar way as the Case E (i.e. level of detail, workload, accuracy, and sustainability). However, due to the '*project specific*' characteristics of the application, the solutions for workload and accuracy variables are differentiated. For example, as the Case F and G were designed to be used for a specific type of repeating projects (e.g. the same type of still mill construction only), relatively heavier workload was acceptable. Furthermore, even though these systems are designed for preliminary planning efforts without having detailed data, Case F and G utilize very detailed historical labor productivity data in order to precisely convey the previous experiences into the new project.

As described above, different perspectives require different variables for developing information systems. In order to evaluate the appropriateness of the variables, real-world cases for preliminary estimating and scheduling systems are introduced. The variables with an asterisk in Table 1 are further discussed using the cases.

Case Study and the Variables

The cases (Case F and G) introduced in this study are the information systems developed to automatically generate preliminary estimate and schedule for steel mill construction. Even though these cases are in the area of industrial plant construction, the basic concepts and methods can be universally applicable to building construction projects.

Understanding some backgrounds of the cases would help comprehend the overall requirements of the systems. Currently, there are several basic methods of steel production depending on different treatment processes of raw materials. The case-company and its mother-company have a license of new steel production process, which provides the case-company with strategic advantages in continuing the same projects worldwide. As an initial effort, the case-company is planning to

build the steel mill in a foreign country in near future. They had built the same facilities only three times in Korea, of which capacities are all different. For the purpose of early planning, the case-company has to prepare a preliminary estimate and schedule. At the same time, the case-company wants to have an information system that is capable of simulating for changed scopes and also capable of accumulating historical database.

From an industry level perspective, utilizing estimating and scheduling information systems are of great importance not only for the case-company but also for any construction organizations. Based on a survey from 37 general contractors, estimating and scheduling were chosen as being the first and second important functions that could be enhanced by using IS (Jung et al. 2004, '*Utilization*' variable in Case A).

An IS planning effort (Jung & Gibson 1999, Case B) for a large general contractor shows that the sales and estimating are the most promising areas (the first and second rank). Preliminary estimating and scheduling directly involves the sales activity. Another important issue is the '*corporate strategy*' variable in Case B. For the case-company in Case F and G, the steel mill construction is a very strategic area. Corporate strategy of construction companies can be characterized in terms of 'product-specialization', 'vertical-integration', and 'market-globalization' (Nam 1993). All of these three strategic requirements were fully met in the Case F and G.

Because of the strategic significance, the '*level of detail*' for the Case F and G was allowed to have budget-item-level or activity-level data. The '*workload*' to operate and to maintain these systems is very burdensome. It is notable that the workload was minimized in the product-specific case (Case E) because all office building projects would use the same system.

Obtaining *accuracy* involves many different methods including adjustment factors and parameters. For the Case F and G, labor productivity is chosen as the parameter for conveying the previous experiences into the new projects. Regularly accumulating and analyzing the historical database can also provide a good source to update the standard contents and adjustment factors (Jung and Kang 2007). The *sustainability* issues of the systems in the Case E, F, and G mainly utilized this automated updating mechanism.

Preliminary Estimating and Scheduling Concepts

The variables discussed above were used to design the preliminary estimating and scheduling system for the case-company. Again, the strategic importance and project-specific characteristic of Case F and G enabled making complicated standards for analyzing and accumulating historical database. The basic concept of preliminary estimating and scheduling is shown in Figure 1 and Figure 2.

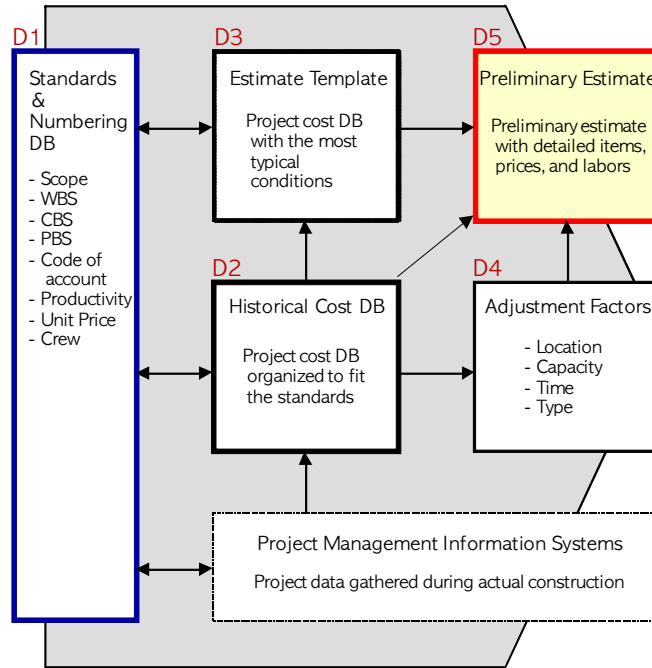


Figure 1. Preliminary Estimating System (Jung et al. 2000)

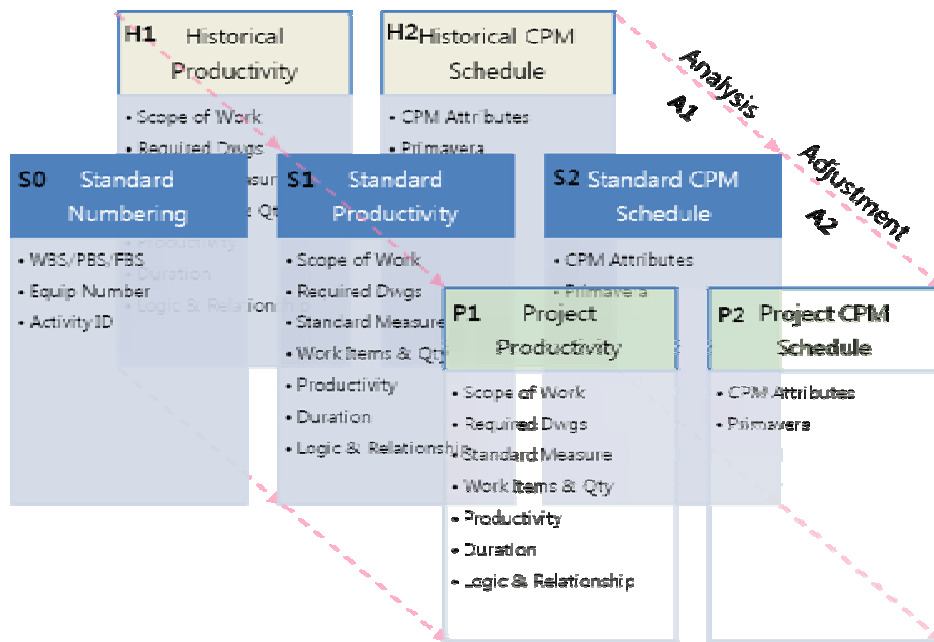


Figure 2. Preliminary Scheduling System

In order to optimize the workload and to enhance the accuracy simultaneously, a principal of separating commodity part (e.g. concrete work) and locator part (e.g. 1st floor) is thoroughly applied (Jung and Kang 2007). In the preliminary estimating (Case F), a standard numbering system and standard cost database including standard unit price, crew, labor productivity are defined as the attributes for commodity part (D1 in Figure 1) those are applicable to any work items within the same commodity group. But, the quantity is defined as an attribute for each work item (i.e. commodity with a locator, D5 in Figure 1) (Jung et al. 2000). The same principal is applied to the preliminary scheduling system (Case G). For the purpose of generating activity durations, the standard scope of work, required drawings, measure, work items, and productivity are defined in the commodity part. Only the quantity and productivity adjustment factors are defined in the activities (commodity with locators).

The other principal is the use of labor productivity as a conveying parameter. In the preliminary estimating system, the unit price for new project is in the form of a formula that automatically calculates the price by using required number of man-days rather than in the form of constant value. This feature enables instant simulations with different location, productivity, or wage inputs. The unit price is also adjusted by applying many factors (D4 in Figure 1). The same concept is also utilized in preliminary scheduling. For an easier understanding of the automated generation of activity durations, a simplified structure of P1 in Figure 2 is illustrated in the Table 2.

In Table 2, the standard characteristics of a commodity (i.e. M63 Machine Erection) are defined in several attributes (row number 3, 4, 5, 6, 8, and 9) including standard crew, man-days, and output. In order to generate activity duration, the system calculates pseudo duration (PDur in Table 2) by dividing the quantity of activity (RQty) by standard daily output (StdDailyOutput). The pseudo duration should be adjusted because even the same machine erection (M63) activities may have different productivity for a different quantity or location. In order to handle this problem, the productivity adjustment (ADJ_P) factor is developed. ADJ_P is an attribute for activities (commodity with a locator). The number of CPM activities in Case G was more than two thousands while the number of commodity was less than twenty (for construction activities only). It was possible because the commodities could be simplified into groups due to the characteristics of industrial plant construction. The duration calculated using standard productivity (PDur) is then compared with actual duration (HDur) in order to automatically produce the ADJ_P for conveying this historical productivity knowledge into new project scheduling.

The process shown in Table 2 gives the impression of being very complicated. However, from a user's perspective, it is extremely simple. If the site working conditions are very similar, all values in Table 2 will be automatically generated including quantity adjustment.

Table 2. Generating Standard Duration Based on Historical Database

No	Attribute	Code	Description	Unit	Value
1	WBS	B.3	Reactor Tower	-	-
2	PBS	211X31	Overhead Crane	-	-
3	FBS	M63	Machinery Erection	-	-
4	CrewMD	501	Total Man-Day for an M63 Crew	MD	6.00
5	CrewMix		Erector	MD	2.80
			Welder	MD	0.40
			Scaffolding Worker	MD	1.10
			Special Laborer	MD	1.50
			Engineer or Administrator	MD	0.20
6	StdNoOfCrews		Standard Number of Crews for an Activity	Crew	2
7	NoOfCrews		Planned Number of Crews for a New Project	Crew	4
8	StdMandays		Standard Number of Man-Days (StdNoOfCrews*CrewMD)	MD	12
9	StdDailyOutput		Standard Daily Output (Productivity)	Ton	2.00
10	DailyOutput		Base Daily Output for a New Project	Ton	4.00
11	RQty		Quantity of Representative Work Item	Ton	68.0
12	PDur		Duration Calculated using Productivity (RQty/DailyOutput)	Day	17.0
13	HDur		Duration Based on Historical Data	Day	15.0
14	Dur		Duration Planned for a New Project	Day	12.0
15	ADJ_P		Productivity Adjustment Factor (PDur/HDur)	Ratio	1.133
16	ADJ_E		Empirical Adjustment Factor (HDur/Dur)	Ratio	1.250
17	ADJ_T		Total Adjustment (ADJ_P * ADJ_E)	Ratio	1.417

If a user wants to increase the daily output, the only required input is the number of crews (NoOfCrews, Row 7 in Table 2). The user does not require input the NoOfCrews for all M63 activities, because the value is located in the commodity database. In other words, one input action will change all activities with M63. Changes of NoOfCrews are also allowed in a sub-group of a commodity (but, only in the P1 in Figure 2 for the database integrity).

Findings and Conclusions

The preliminary estimating and scheduling require extensive inputs from historical databases and planners' experiences. Automating this process can enhance accuracy, support final decisions, enable what-if simulations, and also maximize the organizational learning by accumulating the knowledge.

However, cost effectiveness of developing complex systems for an ad-hoc process is an issue. Therefore, identifying the variables that affect the system requirements will help successful design and implementation of information systems. It is perceived in this study that the 'level of detail', 'workload', 'accuracy', and 'sustainability' are the major variables for developing automated preliminary estimating and scheduling systems. These four variables should be supported by the variables of 'corporate strategy' and 'management'.

In particular, the variable of 'workload' can be interpreted in two ways. One is from the user perspective for performing the preliminary planning, and the other is from the system maintenance perspective for collecting and organizing historical database. This study addressed this issue by separating the commodity group as a standard data set. It is proved that the proposed method does not lower the overall accuracy; rather it improves the overall promptness and convenience for simulations and updates.

Even though the cases for preliminary estimating and scheduling in this study are in the areas of industrial plant construction, the basic concepts, methods, and variables can be universally applicable to residential, building, and civil construction projects. The proposed method was materialized as company-wide web-based applications for the case-company.

Acknowledgements

This study is part of a series of consecutive research efforts funded by POSCO Engineering and Construction Co., Ltd. Special thanks go out to Mr. Young-Seok Oh, Mr. Jinsoo Yang, Mr. Bok-Nam Lee, Mr. Younghwan Lee, and Dr. Sungkwon Woo for their support and participation throughout several painstaking workshops and interviews.

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PIECE-WISE APPROACH TO MULTI-OPTION ACTIVITY OF TIME-COST TRADEOFFS FOR FACILITY PROJECT

Kun-Jung Hsu¹

ABSTRACT

A potential facility project management involving time used of a project can always be tradeoff by additional resources input. Such a tradeoff may come from different options of the activity on the critical path of the project. Finding the optimal combination of time-cost tradeoffs include the use of critical path method and mathematical programming, which considering large numbers of permutations involved. If a CPM network has m activities, with O_1, O_2, \dots, O_m options respectively, will have $O_1 \times O_2 \times \dots \times O_m$ alternatives. Using the hull convex principle to rule out non-active options, the paper develops the piece-wise segmentation concept into a more efficient algorithm. By which the linear programming model divide an activity into piece-wise segments to approach the time-cost trade-off problem. Also, using the problem structure of the time-cost trade-offs, such a k-piece-wise approach can simplify the presentation of linear programming problem, which can be used easily but efficiency in spreadsheet. An illustrative example of such algorithm was presented, and the step by step simulation processes were showed. How the optimal project time-cost solutions were attained via different combinations of reducing time in multi-option activity was showed.

KEY WORDS

Time-cost trade-off, Critical path method, Linear programming, Optimization, Piece-wise segmentation.

INTRODUCTION

Prompted by the present emphasis on time-based competition in industry, more and more project managers face the decision of time-cost tradeoffs problem. A potential project management involving time used of a project can always be tradeoff by additional resources input. The situation of “pay more - save time” is common for project management related decision problems. The available technology of shortening the duration of each activity is often the sources of the time-cost tradeoffs problem. If an activity on the critical path can be shortened by more resources input, one can obtain another project time and its respective cost.

Applying minimum total cost principle, one can attain the time-cost trade-off curve which is the boundary of all feasible region of the time and cost combinations. Now including the mathematical programming will be more efficient when solving the time-cost

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trade-off problem. This so-called time-cost trade-off problem has been studied extensively in the literatures of management science and operation research. Whenever a project is required to be completed within certain duration, the resource assignments need to be adjusted to optimize the resource allocations, and yield the desired delivery time at a reasonable cost. Finding the optimal combination of time-cost tradeoffs include the use of critical path method and mathematical programming, which considering the numbers of permutations involved.

Using mathematical method to solve the time-cost trade-off problem has been studied extensively in the field of operation research and management science. Mathematical approaches convert CPM network and time-cost relationships of the project into constraints and objective functions. Linear programming (LP) and integer programming (IP) are the two major mathematical approaches used to solve the time-cost trade-off problems in project scheduling. By assuming linear relationship between time and cost for project activities, the LP method had been studied three decades ago (Fulkerson 1961, Kelley 1961), and well-developed later. The general philosophy of linear programming convert the project time-cost trade-off problems to minimizing the objective cost function, subject to inequality time constraints, and then solve the problem. Computerized “CPM” procedure and the application of project management system had been developed by many researchers, e.g. Bensen and Sewall (1972), Moder et al. (1983), Liu et al. (1995), and Burn et al. (1996).

The extension of possible options for activities will affect the numbers of permutations in geometric growth. These affect the efficiency of the problem solving. If a CPM network has n activities, with O_1, O_2, \dots, O_n options respectively, will have $O_1 \times O_2 \times \dots \times O_n$ alternatives. For a project manager, this is difficult to find all time-cost trade-offs by hand in practice. So if one can reduce the non-active options of each activity, the model will more efficiency. Also, because the number of ordinary constraints is the most important factor affect how long the simplex method will take. If the inequality constraints can be reduced, then the model will be more efficiency too.

In the following paragraphs, the paper reviews the relative literatures of time-cost trade-off problem first. Then the hull-convex rule of finding the effective options of the activity was introduced in section 3. The Multi-option activity with piece-wise LP model then was developed in section 4, by which it integrated k-piece-wise segments concept into the formulation of the model. An illustrative example of facility project was simulated in section 5. The conclusion remark is drawn in the final section.

FINDING THE EFFECTIVE OPTIONS OF THE ACTIVITY USING THE HULL CONVEX RULE

The basic concept of activity time-cost trade-offs can be named as the normal and crash options. The normal activity time-cost option gives the cost and time involved when the activity is performed in the normal way. By contract, the crash activity time-cost option will expend more cost to reduce the duration time.

Let's denote the normal and crash time-cost points as the coordinates (D, C_D) and (d, C_d) respectively. Supposing the options of the activity can be effective combination, so that all intermediate time-cost trade-offs also are possible and that lie on the line segment between

these two points. For the present, it will be assumed that the resources are infinitely divisible, so that all time between d and D are continuous feasible, and the time-cost relationship of the activity is given by the linear line. The CPM method of time-cost trade-off approach is to determine just which time-cost combination should be used for each activity to meet the scheduled project competition at a minimum cost.

In case of multi-option for an activity, one can use the hull convex rule to eliminate the non-active options of each activity. The following paragraphs list a few examples which showed how hull-convex rules can be processed, thus formulate the piece-wise segments of the inequality constraints. Figure 1 showed the example activities digested from the illustrative example in section. In Figure 1a, the activity-A of the project network has five pseudo-options, $A_1, A_2, A_3, A_4,$ and A_5 . In this illustration, three options A_2, A_3, A_4 can be ruled out because of these points all above line $\overline{A_1A_5}$. The activity-I in Figure 1b can be reduced to three effective options, so the time-cost relationship can be separated as two piece-wise, with different slope of line. Since the computational procedure of piece-wise time-cost tradeoff model will effective searches the critical activities to find the one that can be augmented the cheapest. For example, the time-cost trade-off process of activity-I will naturally choose the pseudo-options in the proper order since the additional cost of crashing time increase as one goes from $I_5, I_4,$ and I_1 .

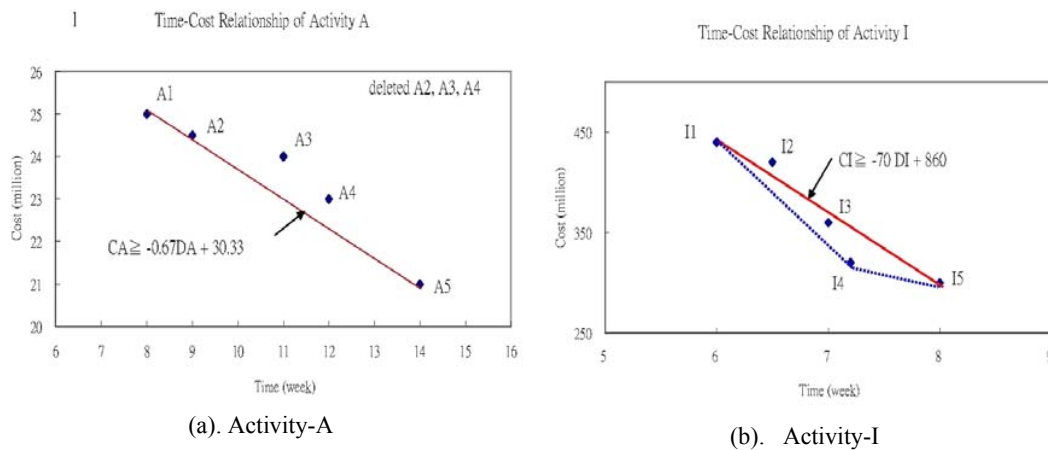


Figure 1. Examples of k-piece-wise segmentation of the activity

After recognizing the effective options of all activities, one can nest the effective inequality into Start-Finish inequality of the predecessor/successor relationship. But whenever inequalities expand, it will increase computing time of finding the solution of Simplex method. How to reduce the inequality constraints now need be reconsidered again. In the next section, the paper develops a k-piece-wise LP model to simplify the inequality constraints formulation. Thus the linear programming algorithm of time-cost trade-off problem can be done more efficiency.

PIECE-WISE APPROACH TO TIME-COST TRADEOFFS MODEL

Let C_{Di} represents the normal cost of activity i ; Δd_i represents the reduction time of activity i ; S_i represents the slope of crash activity- i , it also means the crash cost per unit time of the activity- i . Based on all *normal activity time-cost option*, the minimizing total costs principle of crash time action can be expressed as:

$$\text{Min. } C = \sum_i C_{Di} + \sum_i (S_i \Delta d_i) . \tag{equation 1}$$

The aggregation of all *normal activity costs* of the project, $\sum_i C_{Di}$ is constant, the minimizing total project costs problem then can be rewritten as minimizing the additional total crash cost, $\Delta C = \sum_i (S_i \Delta d_i)$. Now, the basic information we need to address in this question is how the minimum total reduction cost changes. Whenever we crash each possible activity, we choose Δd_i to minimize the total additional crash cost, where the total time of the critical path is T.

To illustrate the formulation of k-options activity with piece-wise approach, Figure 2 showed the network of an example facility project with ten activities. The possible options of each activity and its duration and cost were listed in Table 1 (at the end of the paper). Column (3) of Table 1 showed that except activity J with one option, the other activities all has five options; column (4), (5) showed the time and costs of each options which complete the activities. Using critical path method (CPM), one can attain that the maximum project duration is 90 weeks, and the minimum project duration is 67 weeks. The numbers of permutations for the time-cost combinations will have $5^9=1,953,125$ alternatives.

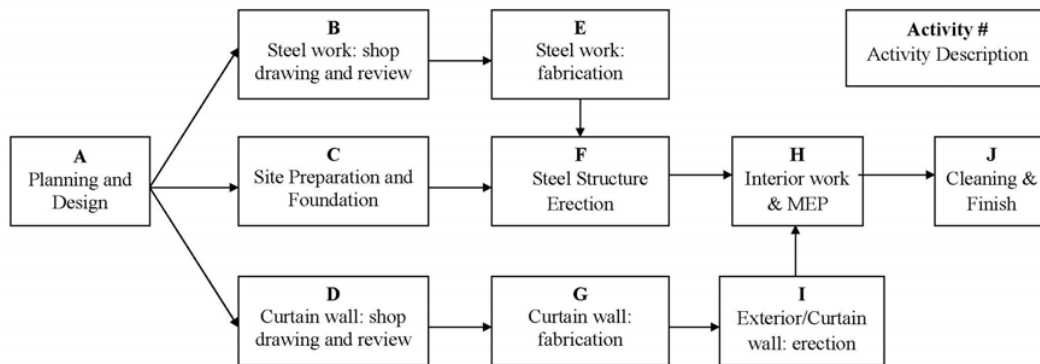


Figure 2. Illustrative example of a facility project network

Table 2. Basic input data of the illustrative example

(1) Activity #	(2) Normal Time (weeks)	(3) Crash Time (weeks)	(4) Normal Cost (million)	(5) Crash Cost (million)
A	14.0	8.0	21	25
B	5.0	3.0	15	20
C	8.0	6.0	22	30
D1	12.0	9.5	18	19
D2	9.5	8.0	19	22
E1	15.0	12.0	400	432
E2	12.0	11.4	432	448
E3	11.4	11.0	448	480
F	20.0	14.0	600	800
G	8.0	4.0	300	320
H1	35.0	32.0	500	600
H2	32.0	30.0	600	700
I1	8.0	7.2	300	320
I2	7.2	6.0	320	440
J	1.0	-	5	-

Notes: In this illustrative example, activity-D has three effective options; activity-E has four effective options; activity-H has three effective options; activity-I has three effective options.

Using hull-convex principle to check the non-active/effective options of each activity, Figure 1 showed the examples of how the hull-convex rule can be used to attain the effective options, thus formulate the piece-wise segments of the activity. The non-active options of each activity were marked in the column (6) of Table 1. For example, activity A showed that there are three non-active options, so it results two piece-wise segments.

After eliminating two non-active options, activity D has three effective options, so it can be divided as two piece-wise segments. Activity E has four effective options, thus it can be divided as three piece-wise segments; and activity H, I both has three effective options, thus it can be divided as two piece-wise segments respectively. All the effective options were showed in column (8) of Table 1. Now the total number of permutation reduce to 3,456 ($=2 \times 2 \times 3 \times 4 \times 2 \times 2 \times 3 \times 3 \times 1$) alternatives, it only about one-sixth of the original case.

The results of the hull-convex analysis of the illustrative example were summarized as Table 2, which only showed the effective options of all activities and represented in sequence of each segments for the activities. Table 2 showed the basic data of the facility project. It includes normal time, crash time, normal cost, and crash cost of all effective options respect to each piece-wise-segments for each activities.

Accordingly, objective function and constraints of all activities contribute the k-options with piece-wise linear programming approach to the time-cost trade-off problem now can be rewritten as follows:

$$\begin{aligned} \text{Min. } \Delta C = & 0.7 \Delta d_A + 2.5 \Delta d_B + 4.0 \Delta d_C + 0.4 \Delta d_{1,D} + 2.0 \Delta d_{2,D} + 10.7 \Delta d_{1,E} + 26.67 \Delta d_{2,E} \\ & + 80.0 \Delta d_{3,E} + 33.3 \Delta d_{1,H} + 50.0 \Delta d_{2,H} + 33.3 \Delta d_F + 5.0 \Delta d_G + 25.0 \Delta d_{1,I} \\ & + 100.0 \Delta d_{2,I} \end{aligned} \quad (\text{equation 2a})$$

$$\begin{aligned} \text{S. t. } \Delta d_A \leq & 6.0; \Delta d_B \leq 2.0; \Delta d_C \leq 2.0; \Delta d_{1,D} \leq 2.5; \Delta d_{2,D} \leq 1.5; \Delta d_{1,E} \leq 3.0; \\ \Delta d_{2,E} \leq & 0.6; \Delta d_{3,E} \leq 0.4; \Delta d_F \leq 6.0; \Delta d_G \leq 4.0; \Delta d_{1,H} \leq 3.0; \Delta d_{2,H} \leq 2.0; \\ \Delta d_{1,I} \leq & 0.8; \Delta d_{2,I} \leq 1.2 \end{aligned} \quad (\text{equation 2b})$$

$$y_j \geq y_i + D_i - \sum_k \Delta d_{k,i}; \text{ for all activity-}i, \text{ and each precedence } i \rightarrow j. \quad (\text{equation 2c})$$

$$y_{FINISH} \leq T; \quad (\text{equation 2d})$$

and $\Delta d_{k,i}^* \geq 0$; for all activity i .

In Eq. 2, the subscription $2,E$ in $\Delta d_{2,E}$ denotes the reduction time of effective option-2 for activity E . To simplify the formulation of the linear programming model, the predecessor/ successor relationship can formulate the job start-finish time inequality constraints. To take the project completion time into account, we add an auxiliary variable y_i which expresses the earliest start time of activity i . For any activities with predecessor (i)/successor (j) relationship, we denote $i \rightarrow j$. So all it presents as inequality constraint, $y_j \geq y_i + D_i - \Delta d_i$, for all activity time-cost trade-off relationship.

The inequality constraint showed that an activity cannot start until each of its immediate predecessors is finished. And the predecessor/successor relationship constraint, Eq. (2c) can be rewritten as follows:

- *Activity with a single immediate predecessor: B, C, D, E, G, I, J.*

$$y_B \geq y_A + D_A - \Delta d_A \quad (\text{equation 3a})$$

$$y_C \geq y_A + D_A - \Delta d_A \quad (\text{equation 3b})$$

$$y_D \geq y_A + D_A - \Delta d_A \quad (\text{equation 3c})$$

$$y_E \geq y_B + D_B - \Delta d_B \quad (\text{equation 3d})$$

$$y_G \geq y_D + D_D - \Delta d_{1,D} - \Delta d_{2,D} \quad (\text{equation 3e})$$

$$y_I \geq y_G + D_G - \Delta d_G \quad (\text{equation 3f})$$

$$y_J \geq y_H + D_{1,H} - \Delta d_{1,H} - \Delta d_{2,H} \quad (\text{equation 3g})$$

- Activity F and activity H has two immediate predecessors, the start-finish time constraints can be written as follows:

$$y_F \geq y_C + D_C - \Delta d_C \quad \text{(equation 3h)}$$

$$y_F \geq y_E + D_E - \Delta d_{1,E} - \Delta d_{2,E} - \Delta d_{3,E} \quad \text{(equation 3i)}$$

$$y_H \geq y_F + D_F - \Delta d_F \quad \text{(equation 3j)}$$

$$y_H \geq y_I + D_I - \Delta d_I \quad \text{(equation 3k)}$$

AN ILLUSTRATIVE EXAMPLE OF PIECE-WISE TIME-COST TRADEOFFS CURVE

Giving the maximum and minimum project duration and set proper interval of the range, one can use table solver to improve the efficiency of the algorithm. The optimal time-cost of each effective option for the activity respect to the project optimal cost and time reduction also can be solved simultaneously. Results of the simulation showed in Figure 3, by which the total cost across the total project duration of the illustrative facility project were showed. Using the built-in LP solver and VBA within Excel 2002, all time-cost trade-off solutions of the illustrative example were found in 3 seconds on a Pentium-V PC.

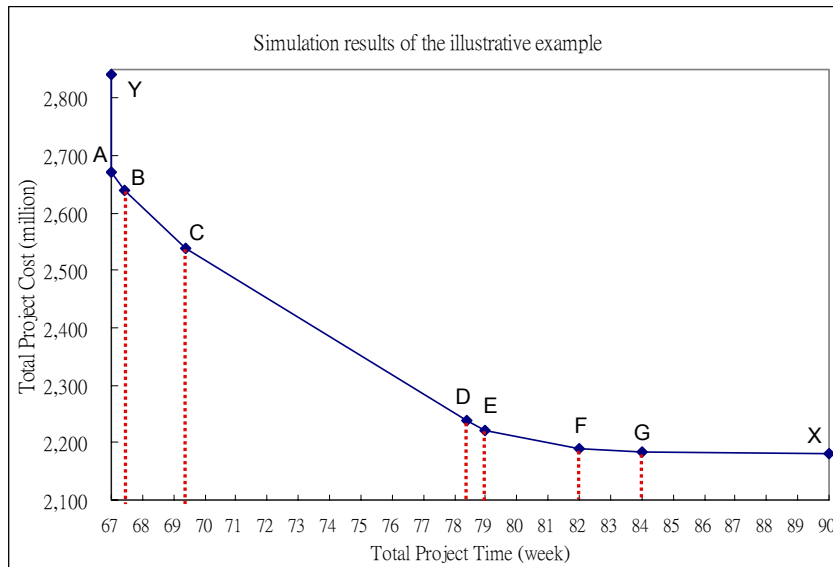


Figure 3. Simulation results of the illustrative example

Using LP to make crashing decisions can provide an efficient means of solving time-cost trade-off problem. However, the process of formulating objective and constraints function for LP is time-consuming and prone to error. Using the problem structure of the time-cost tradeoffs, the paper represents it into spreadsheets, then applies solver of MS-Excel 2002 to solve the LP problem. After representing the spreadsheet data in form of LP problem

structure, the changing cells of decision variables can be given a specific definition name to simplify the spreadsheet work.

Table 3. The decision variables and constraint parameters

(1) Activity #	(2) Max. Time Reduction (weeks); $\Delta d_{k,i}^*$	(3) Crash Cost per Week (million); $S_{k,i}$	(4) Start Time; y_i	(5) Time Reduction (weeks); $\Delta d_{k,i}$	(6) Finish Time; y_j
A	6.0	0.7	0	0	14
B	2.0	2.5	14	0	17
C	2.0	4.0	24	0	32
D1	2.5	0.4	0	0	12
D2	1.5	2.0	0	0	-
E1	3.0	10.7	17	0	32
E2	0.6	26.7	0	0	-
E3	0.4	80.0	0	0	-
F	6.0	33.3	32	0	52
G	4.0	5.0	24	0	32
H1	3.0	33.3	52	0	87
H2	2.0	50.0	0	0	-
I1	0.8	25.0	44	0	52
I2	1.2	100.0	0	0	-
J	-	-	87	0	88

Table 4. Simulation Results of optimal project time-cost for T=67, 79, 84

Activity #	T=67			T=79			T=84		
	y_i	$\Delta d_{k,i}$	y_j	y_i	$\Delta d_{k,i}$	y_j	y_i	$\Delta d_{k,i}$	y_j
A	0	6	8	0	6	8	0	6	8
B	8	2	11	8	2	11	8	0	13
C	14	0	22	8	0	16	8	0	16
D1	24	0	36	8	0	20	8	0	20
D2	0	0		0	0		0	0	
E1	11	3	22	11	3	23	13	0	28
E2	0	0.6		0	0		0	0	
E3	0	0.4		0	0		0	0	
F	22	6	36	23	0	43	28	0	48
G	36	0	44	20	0	28	20	0	28
H1	36	3	66	43	0	78	48	0	83
H2	0	2		0	0		0	0	
I1	28	0	36	35	0	43	12	0	20
I2	0	0		0	0		0	0	
J	66	0	67	78	0	79	83	0	84

For example, column (2), (3) of Table 3 provide basic parameters of maximum time reduction, $\Delta d_{k,i}^*$, and the average crash cost, $S_{k,i}$, which can be direct calculated in spreadsheet from basic data of Table 2. Column (4), (5), (6) of Table 3 present the solution of decision variables of the project in cell format, which include start time, time reduction, and finish time of each effective options of the activities based on $T=88$. Table 4 showed the LP solution of start time, time reduction, and finish time of each effective options of the activities based on $T=67, 79, 84$.

CONCLUDING REMARKS

This paper presents a piece-wise approach to multi-option activity of time-cost tradeoff Problem. The method takes advantage of hull-convex rule and piece-wise segments to structure the framework of linear programming problem. For example, if a CPM network with 9 activities, each with 5 options, then it has $5^9=1,953,125$ alternatives. After using hull-convex principle to check the non-active/effective options of each activity, the paper showed that how it can be easily reduced to 3,456 ($=2 \times 2 \times 2 \times 3 \times 4 \times 2 \times 2 \times 3 \times 3 \times 1$) alternatives.; It only about one-sixth of the original case. Using the piece-wise segmentation concept and problem structure represent in spreadsheet, one can separate the activity as piece-wise segmentation constraints. Along with the spreadsheet tool, the model provide the planner an more efficiency means of analyzing the time-cost tradeoffs problem, and finding the solutions of all project time-cost combination, and its respect reduction time of effective options of the activities. Future developments of the model include (1) to reveal the economic meaning of time-cost trade-off problem via sensitivity analysis and parametric programming. (2) Prompted by the present emphasis on time-based competition, the model can be integrated into project evaluation, so the market information and technology constraints can be treated in the model simultaneously.

ACKNOWLEDGMENTS

Support from the National Science Council of ROC through grant NSC 97-2918-I-426-001 and Fulbright Senior Research Grants 2008-2009 in this study is gratefully acknowledged.

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Table 1: Activity options for illustrative example

(1) Activity #	(2) Activity Description	(3) Option	(4) Duration (Weeks)	(5) Cost (million)	(6) Remarks	(7) Total Options	(8) Effective options
A	1. Planning and Design	CREW1+EQUIP1	8.0	25.0		5	2
A	2. Planning and Design	CREW2+EQUIP2	9.0	24.5	non-active		
A	3. Planning and Design	CREW3+EQUIP3	11.0	24.0	non-active		
A	4. Planning and Design	CREW4+EQUIP4	12.0	23.0	non-active		
A	5. Planning and Design	CREW5+EQUIP5	14.0	21.0			
B	1. Steel work: shop drawing	CREW1+EQUIP1	3.0	20.0		5	2
B	2. Steel work: shop drawing	CREW2+EQUIP2	3.5	19.5	non-active		
B	3. Steel work: shop drawing	CREW3+EQUIP3	4.0	19.0	non-active		
B	4. Steel work: shop drawing	CREW4+EQUIP4	4.5	16.5	non-active		
B	5. Steel work: shop drawing	CREW5+EQUIP5	5.0	15.0			
C	1. Site Preparation and Foundation	CREW1+EQUIP1	6.0	30.0		5	2
C	2. Site Preparation and Foundation	CREW2+EQUIP2	6.3	29.0	non-active		
C	3. Site Preparation and Foundation	CREW3+EQUIP3	7.5	26.5	non-active		
C	4. Site Preparation and Foundation	CREW4+EQUIP4	7.6	25.0	non-active		
C	5. Site Preparation and Foundation	CREW5+EQUIP5	8.0	22.0			
D	1. Curtain wall: shop drawing	METHOD1	8.0	22.0		5	3
D	2. Curtain wall: shop drawing	METHOD2	9.0	20.5	non-active		
D	3. Curtain wall: shop drawing	METHOD3	9.5	19.0			
D	4. Curtain wall: shop drawing	METHOD4	11.0	19.5	non-active		
D	5. Curtain wall: shop drawing	METHOD5	12.0	18.0			
E	1. Steel work: fabrication	METHOD1	11.0	480.0		5	4
E	2. Steel work: fabrication	METHOD2	11.4	448.0			
E	3. Steel work: fabrication	METHOD3	12.0	432.0			
E	4. Steel work: fabrication	METHOD4	14.0	415.0	non-active		
E	5. Steel work: fabrication	METHOD5	15.0	400.0			
F	1. Steel Structure Erection	CRANE1+CREW1	14.0	800.0		5	2
F	2. Steel Structure Erection	CRANE2+CREW2	16.0	750.0	non-active		
F	3. Steel Structure Erection	CRANE3+CREW3	18.0	700.0	non-active		
F	4. Steel Structure Erection	CRANE4+CREW4	19.0	650.0	non-active		
F	5. Steel Structure Erection	CRANE5+CREW5	20.0	600.0			
G	1. Curtain wall: fabrication	METHOD1	4.0	320.0		5	2
G	2. Curtain wall: fabrication	METHOD2	5.0	318.0	non-active		
G	3. Curtain wall: fabrication	METHOD3	6.0	315.0	non-active		
G	4. Curtain wall: fabrication	METHOD4	7.0	308.0	non-active		
G	5. Curtain wall: fabrication	METHOD5	8.0	300.0			
H	1. Interior work & MEP	CREW1+EQUIP1	30.0	700.0		5	3
H	2. Interior work & MEP	CREW2+EQUIP2	31.0	650.0	non-active		
H	3. Interior work & MEP	CREW3+EQUIP3	32.0	600.0			
H	4. Interior work & MEP	CREW4+EQUIP4	34.0	550.0	non-active		
H	5. Interior work & MEP	CREW5+EQUIP5	35.0	500.0			
I	1. Exterior/Curtain wall: erection	CRANE1+CREW1	6.0	440.0		5	3
I	2. Exterior/Curtain wall: erection	CRANE2+CREW2	6.5	420.0	non-active		
I	3. Exterior/Curtain wall: erection	CRANE3+CREW3	7.0	360.0	non-active		
I	4. Exterior/Curtain wall: erection	CRANE4+CREW4	7.2	320.0			
I	5. Exterior/Curtain wall: erection	CRANE5+CREW5	8.0	300.0			
J	1. Cleaning & Finish	CREW1+EQUIP1	1.0	5.0		1	1
					Total alternatives	5 ⁹ = 1,953,125	3,456

The environmental management of coastal areas in Egypt

Strategy and urban development

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Research Methodology:

Coastal areas in Egypt and all over the world suffer from the excess stress of human activities specially the touristic development of the coastal waterfront and the urban extension which negatively affect the environment.

Recently there has been an increase in the understanding of the coastal areas problems in Egypt and there have been effective decisions and moves towards the protection of the environment in general and the coastal one in particular, since it is considered to be a national heritage which must be kept well and preserved.

This research is a positive step in the resent trend of the environmental urban development of the coastal areas in Egypt.

Mainly most coasts, with its dynamic features and its direct contact with wide water surfaces, are affected with nature forces and factors which has special restrictions on the development process for these areas. An environmental balance must be taken into consideration between the different suggested activities and development, and the environmental features of the area, in order not to destroy the sensitive environmental system and at the same time protect human beings and their belongings from being in an area of negative factors and distracting effects, besides keeping the harmony between architectural features and the nature of the site.

This target will not be achieved unless we study the contents of the main elements of the coastal waterfront such as the natural elements and the new man-made elements.

Water is the first and main element, second is the nature character of the land facing water which may be either high and has gradual heights, which may reach 30 meters, as it is found in "CORAL BAY RESORT" in Sharm El Sheikh, and the suggested area in "TABA" with the fyordats.

Or the land may be flat or even less in height than the water level and in this case the water may sometimes interacts with the land resulting in water grooves, most of which are man-made as in the case of "GONA RESORT" in Hurgada. Or there may be natural reefs or barriers to protect the marine environment and this is found in most of the coastal areas between "EL KOSARE and MARSA ALAM". Or it may happen that they transfer the area to land, considering it as a sea shore, as in the villages and resorts area in "HURGADA". The final element is the new man made trend either by building, or agricultural changes in the features and contents of this region, and here we undertake this new element from different levels and point of view, such as the

kinds of building and the used building materials and even the special style or character of these kinds of buildings.

The question is: Is it necessary to use the historic symbols of the place or city or even country, as it is happening with the Old Egyptian “pharos style” used in “SHERATON SOMA BAY-SAFAGA RESORT” by the Red Sea, or the Andaloiseya Character used in “HAYATT REGENCY”-Sharm el Sheikh, or the West Arab Style used in the “IBROTEL RESORT” in Marsa Alam, and the use of the environmental design using thick self barring walls covered with vaults and domes?

Or is it better to encourage the link with the visual acceleration of the nature components and the mountains series backing the site following and matching with these sky lines and using local materials from the surrounding nature, with the ability of inventing and adopting a special and unique artistic thinking taken from the surrounding nature both form and character concept?

And in doing this we use all the available facilities to match the new touristic output with the surrounding environment keeping the harmony with it and applying the idea of sustainability to reduce consuming and pollution to protect the living creatures and the nature contents either in water or on land.

This is our aim besides the achievement of some special architectural symbols, which the architect may suggest to develop his creation power and unique character to be distinguished from the other similar projects.

The research will discuss three main elements as follows:

- 1- The main steps and major parameters for the environmental protection, especially for the coastal areas from the urban extensions with the focusing on the environmental problems and their dangerous results and influences.
- 2- The main elements which construct the water fronts and their architectural form relation with the nature of the site.
- 3- The environmental and architectural parameters of the water fronts in Egypt with the consideration of the previous elements.

Introduction:

Tourism is considered recently the most important economic feature in Egypt; tourism industry is much more developing during the last decade. It is granted that this development and investment in this field is continuing, especially in the coastal tourism and the construction of resorts and hotels which face the water fronts in Egypt and precisely in the red sea coast area, especially the sites such as Hurgada and Sharm El-Sheikh and extends south until reaching Marsa Alam and may reach, in future, till Hallayeb. It is expected that the number of hotel rooms will be five times by the year 2007.

As a result, of this increasing growth of the coastal tourism, there were some negative effects on the environment in many regions since that the sensitive present ecological systems, especially the coral reefs regions and the creatures related to it.

The rules of the law of 4 / 1994 stated that that the developing projects must perform assessment studies showing influence on environment before the construction of such projects. The study must show the negative and positive

effects which result from the construction of the project on the ecological orders of the regions, putting many restriction, in order to protect the marine and land life, but there was an ignoring for the other components of the natural environment such as the location pattern of the region, the function and the water flow, in order to protect the whole system;

The environmental consideration bases which regulates the development in the coastal regions:

Here we will discuss, in brief, the main principles and considerations which were announced, in March 1996, by the environment affairs authority with regard to the protection of the water front shore. This regulations and considerations are applied in all coastal regions in Egypt either by the Red sea side or the Mediterranean.

1/1 the main principles of the environment parameters

1st rule: “It is not allowed to perform any functions which may lead to the deterioration or damage of the natural environment”.

This rule if applied with no exceptions, their will be no touristic resorts or any other coastal constructions since they damage, in a way or another with different percentages, the natural environment when the site of the construction is prepared and a part of the green cover of the land is removed for this purpose.

2nd and 3rd rule: “It is not allowed to harm the animals or plants either on land on marines also not to cause any pollution in the land, water or air.

In fact the presence of human beings and their activities, must lead in a way or another, to the harm of some animals and natural plants in the region. Like the migration of the dears and foxes and some birds fig. (1) and the destroying of natural plants by building or pigging the soil or even there might be some changes in the topographic nature by replacing the natural plants with others as in the design of new landscape of the site.

Also there might be pollution to the natural sinks underground by desalination of salted under ground water returning after wards more salted and the usage of the drainage water and irrigation and other activities.

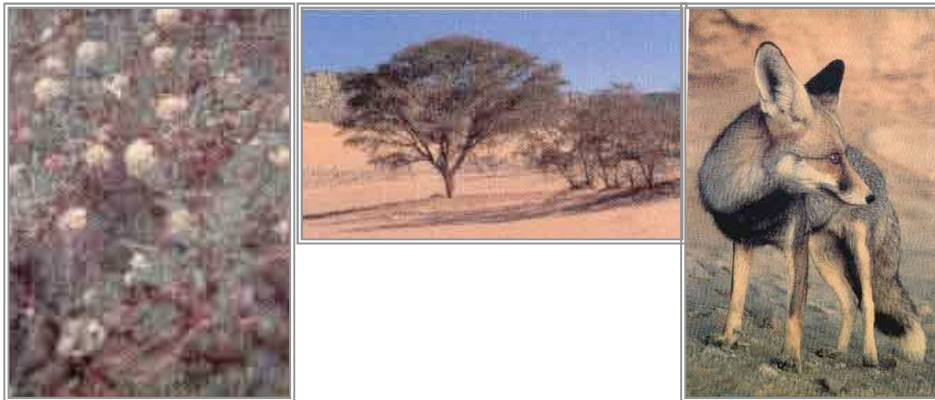


Figure 1: Animals & natural plants in the region.

4th and 5th Rule: “Encouraging the idea of the presence of natural protective regions”.

It is not enough to encourage such ideas, there must be multi regions selected for this purpose by the government, like those in Ras Mohamed in Sharm El-Sheikh which is considered as a natural protected region.

6th rule: “It is a must to evaluate the environmental influence of all the projects either new ones or the already built:

In most cases the evaluation is not accurate or even there are some exceptions according to the level or standard of the owners of the projects.

7th rule: “It is not allowed to any project that may influence the flow of the natural shore line or cause any change either outward or inward.

In this point there is the usual statement, that always leads to illegal actions which is: “unless there is an agreement from the officials and the authorities of the environmental affairs”

In fig (2): a picture taken by the satellites in the years 1984 – 1997 - 2007 for the development of the coast in Hurgada – As it shows there has been destroying of most of the coral reefs in the shallow water which is considered to be the warm water for the small fish before their transfer to the deep water, also it is noticed the change inwards of the shore line as in El-Gona resort in Hurgada.

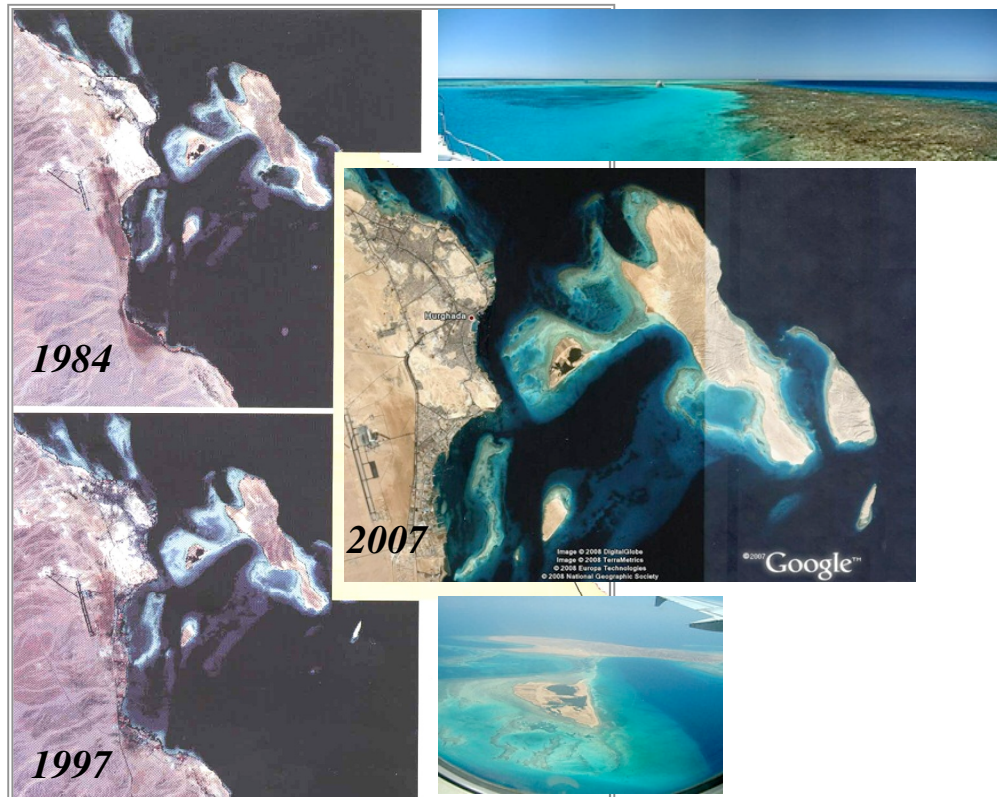


Figure 2: The development of the coast in Hurgada.

1/2 the coastal environmental problems and the risk that may result from it, through the following:

- (a) Land form and the Lithology.
- (b) The vertical and horizontal land movements.
- (c) Range of tide and the sea waves heights in addition to the regions with grooves in the land topography as a result from heavy raining and torrents which is frequent on the coast of the red sea fig. (3).



Figure 3: The coast of the Red sea.

Urban design systems and the development management to control and protect against such risks:

1. The study of risk zone mapping (RZM).
2. The study of the possibility of damage exposure, vulnerability analysis (VA).
3. Preparing the strategy of dealing with the expected problems: i.e.: torrents (precautionary strategy).
4. The study of the master plans.

The urban design planning deals with these studies in a comprehensive way in order to reach a high degree of protection from the risks, which exposing the coastal regions.

Taking an example of such risks, the torrents that considered a major risk. The study starts with the risk zone mapping (RSM) which aims to specify the level of risks in the study case region, these maps are prepared every interval of time of 20 – 50- 100 years.

It consist of contour lines separating the different levels of risk and it is considered to be the basic maps which are used afterwards for the preparation of the vulnerability analysis maps, in which they can drive a prediction of the situation during the torrents and after and the estimation of the damage and the ways to prevent such a crises to happen.

Also there are plans for the land use and the urban planning with the recommendations and parameters, the building materials and the construction systems best to be used, also preparing the ways in which to deal with the different stages of the torrents crises before, in and after it happens, that is to prepare for the vulnerability strategy in which the technical recommendations of the buildings and the necessary precaution for the infrastructure elements such as roads and sewage plumps are designed to deal with the torrents water.

Here it is worth mentioning the catastrophe of the ocean in south East Asia, where the tide caused by the earthquake lead to the death of more than 150 thousand human beings in December 2004.

2/1 the main elements of the water fronts:

It consists of natural elements as well as man-made elements. The main and most important natural element is water, and then comes the nature of the land in front of the water, which takes different shapes and natures.

2/1/1 the element of water:

water either salt or fresh, for the salt water which is present on the sea shore and its distance and level from the different buildings according to the geographic nature of the site, and this will be discussed when dealing with the nature of the land shape in the water fronts.

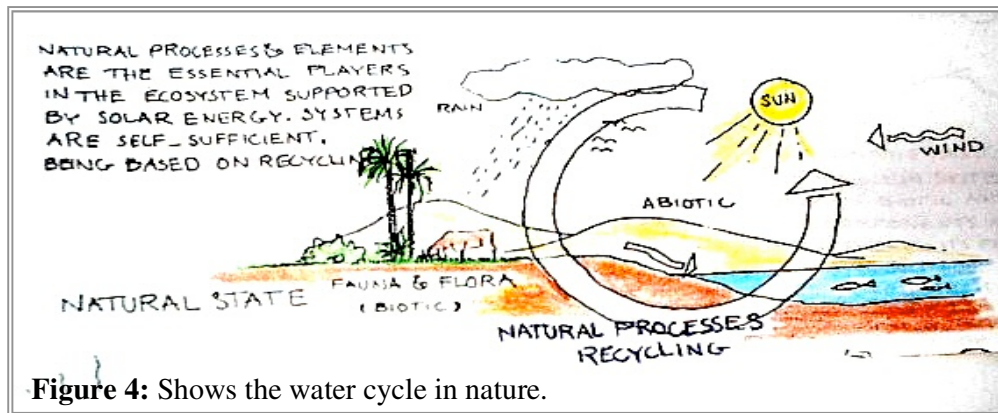
As for the sources of fresh water which is used by the plants, animals and humans for living, could be obtained from the reservoirs, water and sinks, the main natural sources are:

- 1) Rain water.
- 2) Rivers and lakes.
- 3) Under ground water

Lately there has been other source resulting from the technological systems such as:

- 1) The desalinated water.
- 2) The treated sewage water.

It is noticed that Egypt, although having a desert nature, still it does have all these sources, with different rates according to the ecology of the region.



When approaching a design in a site with environmental considerations, i.e.: desert, hot and inhumed weather, usually there is an important question that is “How are we going to support the natural local community? And the achievement of visual properties in a desert nature site without over usage of water.

To achieve this it is important to study first the sites sources and supply of water in order to be able to estimate the intensity and kind of community in this site in the future and not the opposite like what had happened in a place such as Hurgada which grow at random and developed without enough study for its water sources which lead to a problem in the fresh water supply for the city. Also there is an example in southern California in the United States, in L.A city, which developed depending on the import of fresh water from the neighbor states.

What happed was that as the needs of inhabitants were satisfied the more the urban extended and the more the water problem became greater, and the need to find a new water supply was important, they solved their problem by using water with care besides the re-use strategy of water which helped to solve the fresh water problem supply in such regions.

There is a relation between beauty and planting which is an idea came from the western culture and spread all over the world, where the European ideas of “ green gardens “ either tropical or other was the main idea of designing the landscape, but in desert regions, as in our coastal areas in the red sea, the problem with such green arrangements and their great need of fresh water, drove the designers to notice the beauty in the natural origin desert environment, which may be considered for some people that it is a poor environment, but in fact it could be a piece of art if well designed as in the project of developing the California desert and keeping it raw and natural with its own beauty. In fact there are many tourists, who come to the Red sea region in Egypt, ask for the desert view beside the sea view. Fig (5)



2/1/2 the nature of land forms in the water fronts:

- In some cases the water interacts with the land forming natural water grooves and bays, acting like a natural marine, such as that found in “Movinpeik quasar Resort” Fig. (6) and “ Coray Maras Alam” and ‘ Solay Marsa Alam “



Figure 6:
Quasar Movinpeik resort.

- Or there may be the form of land where there is a great difference in levels between the land and the sea shore, such as that found in Sharm El-Sheik in “ Coral Bay Resort “. Fig.(7)
- Or there may be a natural barrier of reefs, acting like a natural protection to the marine environment, this is found in most coastal areas between “ El Kosar” and going south till “ Marsa Alam”, Such as “ EL Fayroz Plaza Resort “ in Marsa alam “. Fig. (8)



Figure 7: Coral Bay Resort in Sharm El-Sheik.



Figure 8: Plaza Resort in Marsa alam.

- Or in some cases where humans interfere with nature by destroying the natural feature of the sea shore by destroying the shallow reefs to become a beach – like most of the Hurgada touristic resorts, or by getting water inwards forming artificial lakes, as found in “ El-Gona Sheraton” north Hurgada. Fig. (9)



Figure 9: El-Gona Sheraton in north Hurgada.

2/1/3 Protective regulations to be followed in order to achieve an environmental strategies to be able to save water and reach a good degree of management in order to prevent its dynamic effect, such as perspiration, deformation and soil supper saturation :

1. Renewing the green cover of the land, by using the local natural planting which use minimum amount of water and helps the return of environmental balance of the area.
2. Reshaping the areas where the land had been deformed and make sure to be stable in future.
3. Stopping and slowing down of the torrents in order to benefit from it by the design and environment study of the phenomena.
4. Better to leave the flow tracks natural as possible, since they balance their nature by time and the rocks and roots work on keeping the soil strong enough so as not to be deformed .

Keeping these flow tracks as a landscape feature, even without water in them, help in making them work as a drain for the rain water and the sudden torrents. Fig (10), “El Fayroz Plaza resort”

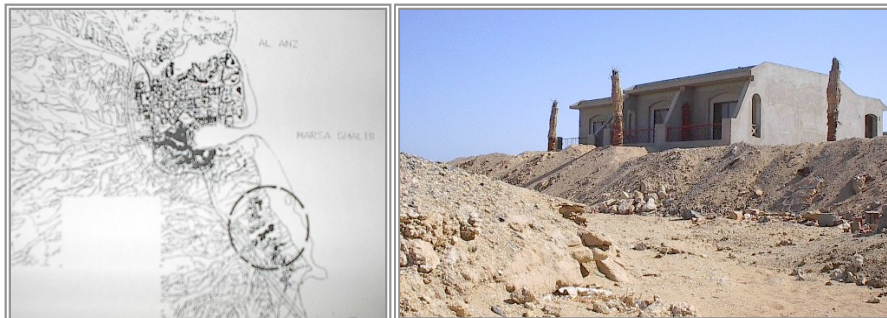


Figure 10: El Fayroz Plaza resort.

2/1/4 Coastal areas and the dunes:

- 1. It is recommended that the coastal bays and islands are kept natural without any modifications.**
- 2. The protection of the plant cover of the dunes which work as a fixing agent, with the prevention of using an mechanical equipment and using passages for the pedestrians movement , fig.(11)**



Figure 11



Figure 12

- 3- Using lifted level passages between the high levels in the land.**
- 4- The presence of a sea shore passages according to the nature formation of the site with environmental study.**
- 5- In case of the dry flow tracks, it could be used as pedestrian passages using natural rocks from the site, moving from one level to the other, till reaching the coastal shore. Also it could be used as an artificial lake using the high concentrated salted water, resulting from the desalination of water, which may reach 75% of the water extracted from wells. In this case we could benefit from these tracks visually as a landscape element as well as decreasing the economic over payment for draining the salt water in other more deeper wells with the necessary treatments which may cause damage afterwards fig.(12) “ The Gona Movinpeik Resort ”.**

2- The architectural built facades forming the coastal water fronts:

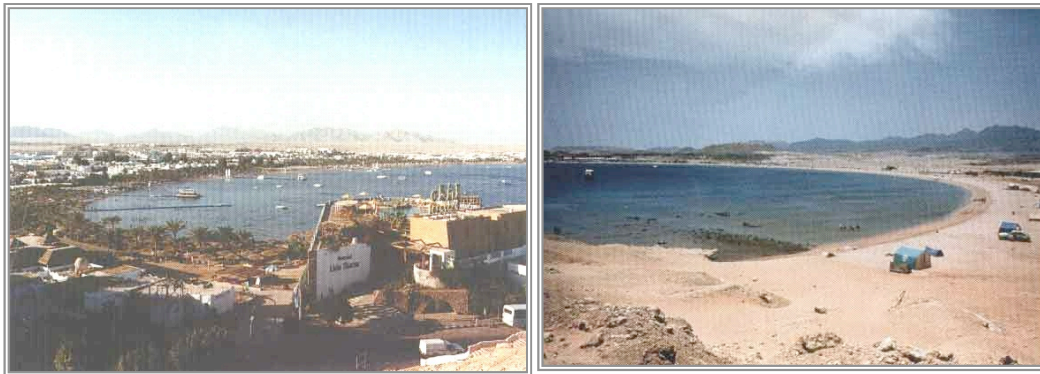
The built elevations are one of the man made components which form the water fronts.

Man had influenced the natural environment by changing its topographic nature, and using its natural resources of materials and energy sources. He also built many forms of structure with different heights, masses and forms, either using the local building materials of the site, in which it is called environmental architecture or he used other materials and design styles of another areas in which it is called emitting architecture from other cultures.

Either this or that there are architectural parameters which are used by the authorities concerned with the building specifications for the assessment of projects designed for the development of the coastal areas.

2/1 the building parameters for the development of the coastal areas:

- The human density for the touristic developed area is not more than 30 person / acre and 50 person/ acre for the small projects.
- The built density is no more than 40% of the total area of the site. The built ratio is the sum of areas of the built floors with respect to the total area of the site. See fig. (13) Before and after development in Neama Bay.



- The maximum height, of buildings which are built less them 100 m far from the edge of building at the sea side, is two floors with a maximum of 7 m high, also the rest of the buildings also where in the site is of maximum three floors with maximum 10 m height, with the exception of structures of special function, i.e. : Maezana of the mosque and the water tanks, on condition that they take the agreement of the general authority of the tourism development .
- The ratio of elevations lengths facing the sea, to the depth of the site is (1:1) to (1:3).

2/2 the study of the assessment of the environmental influence includes the following:

1. The description of the proposed project: Its properties and the aim of it, the environmental and general benefit from it and finally the summary of the predicted effects as a result of constructing this project.
2. The purpose of the project: A complete description of the project components the schedule of the installing and management systems. The site location with regard to the development region.
3. The site of the project: The survey maps with the location of the site of the project and its surroundings its link with transportation and main roads – the roads, buildings and public services near the project such as the marines for yachts and ships – the distance between the project and

the sea shore- the allowed maximum heights – the natural preserved areas near by the project.

(4) The design and planning of the project and its elements:

The study of the foot print – with the maximum areas of land and water affecting the project:

- The project drawings and planning and photos and the present natural features and the proposed ones, such as the sea shore line, the places of the coral reefs and natural plants.
- The description in general of the areas and heights and the kind of the project buildings such as: hotel, units, villas ...etc.
- The description of the different activities performed in the project.
- The areas of further extension and the preserved areas in the future in the location of the project.
- The prediction of the number of inhabitants, plans, elevations and sections and the infrastructure.
- The above parameters and regulations do not in fact put restrictions on the style or system of building in such areas, and as a result we find that, The architects use different styles and system according to their design ideas or to that of the owners
- The designers may use the symbols and historic styles like that taken from the Pharonic Architecture to achieve the artistic ratios, and this is used in many resorts such s “ Sheraton Soma Bay resort” in Safaga, fig. (14) Or they may use the Andalos Styles, such as “Hayat Resort” in Sharm El-Sheik. Fig (15) or the Morocian style in Coray Bay - Marsa Alam. Fig. (16)



Figure 14: The Andalos Styles.

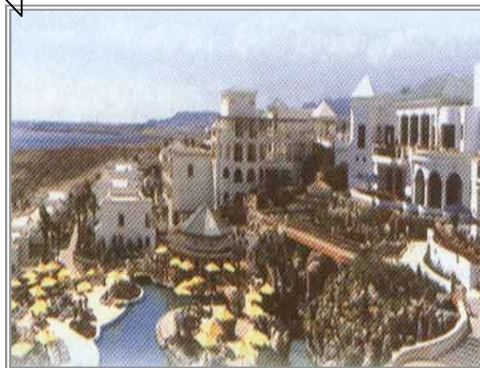


Figure 16: Hayat Resort - Sharm El-Sheik.



Figure 15: Coray Bay - Marsa Alam.

The designers also may use the local brick and rocks, extracted from the surrounding environment to be used in building and finishing of walls, and the use of domes and vaults in roofs as an environmental system to decrease the energy consuming and in order to achieve a design which goes with the natural environment and the traditional architecture, as shown in the “Movinpeik El-Kosere Resort “ Fig. (17) & fig. (18).

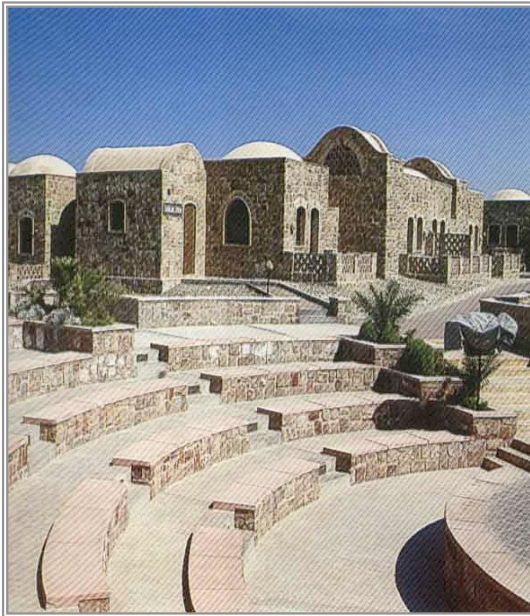


Figure 17
Movinpeik El-Kosere Resort

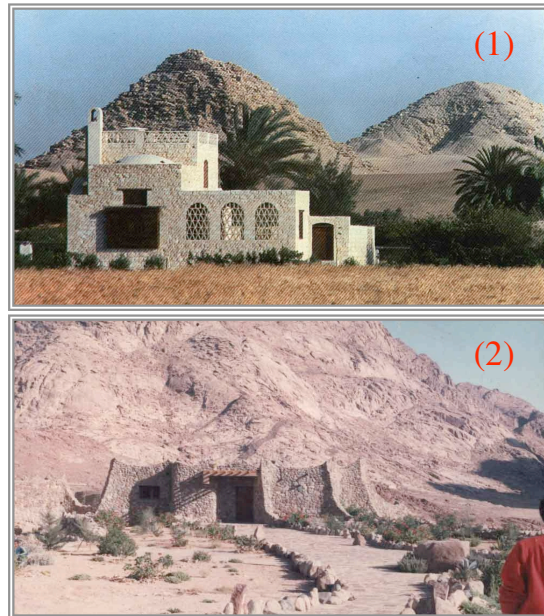


Figure 18
(1) Villa in desert Abu Seer
(2) St Catherine's Resort

There is no doubt that, the more the designer uses the environmental elements in his planning and design or the use of materials and elements which matches with the local environment in a way that goes with the nature recycling and the ecological order and the use of the renewable energy sources and the decrease of the harmful out-puts which go to the sinks causing its pollution, as mentioned before, the more this designer will be able to reach his aim and achieve the protection of environment as much as possible. In order to decrease the damage and increase the positive value of such projects, leading to an economic touristic development with environmental respect and value and this is what we aim to achieve.

Results and recommendations:

- (1) The parameters and restrictions to protect the environmental system for coastal areas must be complete with no exceptions what ever the importance of the project built in it.
- (2) The reconsideration or the environmental rules which control the development in the coastal regions, since the study which was prepared and

approved in September 1998 and up till now had proved that there are many weak points which cause trouble, and that this agenda must be followed by a sub-agenda to face the problems which leads to the destruction of environment.

- (3) The need to extend the natural protective areas and its surrounding regions.
- (4) The natural sea shore coastal line must be preserved as it is with no interference, using the satellites pictures periodically to preview any change either natural or man-made.
- (5) It is important to keep the nature of the land topography in the sites where touristic projects are built especially those with natural grooves resulting from the heavy rain- putting plans and programs to face the risk of torrents before it happens.
- (6) The more we use the water element, which is the main element in the water front character, in a natural way with its flow and combining with the land natural structure, the more will be the architectural and form value of the built project, taking into consideration not to destroy the sinks underground and the use and benefit from the desalinated water, since the use of the treated water, either with desalination or biological treatment is so promising to be the source of fulfilling the great demand of fresh water for these areas.
- (7) The original natural environment of the area, even if it lacks the green cover, still has its own beauty and its special desert environment with its limited kind of plants and rocks and natural colors, which is something most of the tourists are interested in, since they may not have environment in their own countries.
- (8) The more, the designer benefit from the natural structure of the site using its local materials and energy sources, the more he will be able to save the natural environment with its homogenous effect and influence between the built projects and its surroundings, which in turn provide an additional esthetic, cultural and economic values to the project and the site itself.
- (9) Our aim is to achieve the harmony with the natural environmental orders when designing touristic projects in the coastal area to protect the natural environment of the water fronts while developing these areas.

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Solar Thermal Electric Panel (STEP): Performance Analysis

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ABSTRACT

With increased energy costs, driven by a higher demand and dwindling supplies, the search for alternative and renewable energies to compensate or even replace the current energy production technology is very necessary. A properly sized and installed solar energy collection system can be a practical alternative for acquiring all or some of your energy needs.

The main goal of this research was to conduct an experimental analysis of an integrated photovoltaic and thermal system. The design and development was initialized by a group of students and advisors from both the University of Missouri-Rolla and Crowder College with the intent to use the hybrid system as part of the solar houses in the upcoming solar decathlons. Previous research studies on hybrid roof systems have shown increased performance however the differences in the systems studied vary in their setups and use of materials. In the case of this study a series of copper tubes were integrated into a metal seam roof with an amorphous silicon panel encased in low iron glass. This experiment encompassed almost 160 square feet of hybrid Solar Thermal Electric Panel (STEP) system panels and performance data acquired was used for input to computer simulation software to optimize the system for application to the UMR/RTI solar house that is entered into the 2005 DoE's Solar Decathlon.

Based on experimental tests overall efficiency of the STEP system is 50% while a separate thermal and electric system is estimated to be 26% for the same roof area. An assumption for the thermal systems is that they are of similar makeup and their efficiency is based on an ambient input temperature. The glazed versus unglazed analysis yielded a glazed panel reducing the PV collection by 23% and increasing the thermal collection by 200%. In conclusion this paper will discuss additional performance based analysis on the STEP system thermal and electric outcomes.

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INTRODUCTION

With increased energy costs, driven by a higher demand and dwindling supplies, the search for alternative and renewable energies to compensate or even replace the current energy production technology is very necessary. A properly sized and installed solar energy collection system can be a practical alternative for acquiring all or some of your energy needs.

The combination of photovoltaic and solar thermal (PVT) system has been proven to provide greater efficiency than their stand-alone counterparts. Previous studies on hybrid assemblies have found favorable results supporting the combination of both systems (Zondag, 2002). Drawing the heat of the back of the photovoltaic panel can increase the panels output as well as the panels lifetime output. While the concept is not new research continues on developing more effective ways that will ultimately result in better performance at a lower cost.

This experiment encompassed almost 150 square feet of the hybrid STEP system panels and performance data acquired will be used for input to computer simulation software to optimize the system for application to the UMR/RTI solar house that is entered into the 2005 DoE's Solar Decathlon. The obvious benefit of implementing a STEP type system to the planet would be the reduction in energy consumption and reducing the impact on the environment resulting with the improvement in human living conditions.

Explorations of conditions such as temperature gain due to stagnation of the thermal system and how the system performs with and without glazing as it relates to this systems design will be discussed further in this paper.

EXPERIMENTAL SETUP

The solar thermal electric panel (STEP) system is a standing seam copper roof with three channels for copper piping. The PV panel, a Uni-Solar PVL series panel, is self-adhesive and is applied to the base of the standing seam roof over the copper pipes that were inserted into the channels of the standing seam copper roof pan. There are extra bends at the top of the pan to facilitate the glazing of the collector. The glazing is needed to trap the sun's radiation thus amplifying the heat, via the greenhouse effect, inside the collector. Figure 1 shows the cutaway view of the STEP collector.

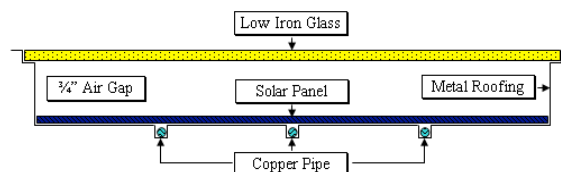


Figure 1: STEP System - Cutaway View

The roof section tested consists of six STEP panels. The overall dimensions of the collection area are 16.67 feet tall by 8.91 feet wide for an area of 148.61 square feet. The angle measured from the ground is 61 degrees. This angle is optimal for winter

solstice for the latitude in Rolla, Missouri, where the testing took place. The results presented here are for the collector angle of 61 degrees in late March. Figure 2 shows a picture of the test roof.



Figure 2: Test Roof

Using a 40 gallon solar hot water tank with food-grade antifreeze with a rust inhibitor combined with distilled water. The fluid is pumped using a 1.5 horsepower pump from the bottom right corner and flows out the top left corner. The equal geometric flow pattern lends itself to uniform flow and ensures that there will be a solid column of water in the thermal collection piping.

A type-T thermocouple was used to measure the inlet and outlet temperature. When the fluid has passed through the collector area it passes through the outlet thermocouple before entering the hot water tank via the top hot water supply pipe as indicated in Figure 3.

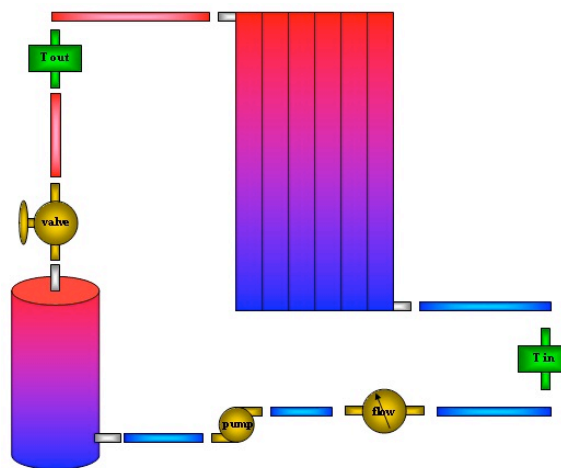


Figure 3: Fluid Flow System

The photovoltaic (PV) panel voltage was measured across the capacitor bank while panel amperage was obtained by measuring the voltage drop across a low resistance

resistor, 0.03 Ohm, in series to the flow of electricity. Knowing the amperage and voltage the maximum power can be obtained.

A silicon PV based pyranometer was used to obtain the available radiation numbers used in this report. The pyranometer was mounted on the roof surface to see the same angle as the STEP panels, 61°.

The data was collected in ten second intervals. The data logger collects all of the temperature and voltage data from the components mentioned previously. The ambient temperature was taken via a digital temperature gage. A diagram of the measurement system can be seen in Figure 4.

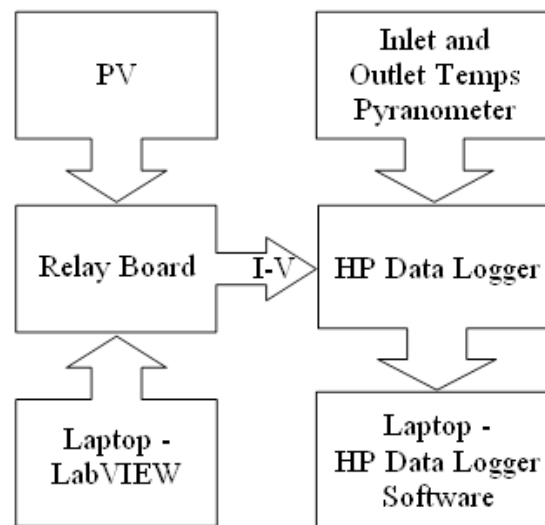


Figure 4: The Measurement System

The PV power output was obtained by finding the maximum power point on the I-V curve. This technique was used because finding a consistent load and the cost of inverting equipment would have been more expensive and would add more unknowns into the experiment. Forty-eight data points are taken at each capacitor bank charging cycle to obtain the maximum power the PV is outputting. Figure 5 shows the I-V curve over a ten second charging interval. Figure 6 shows the voltage and amperage plotted versus time.

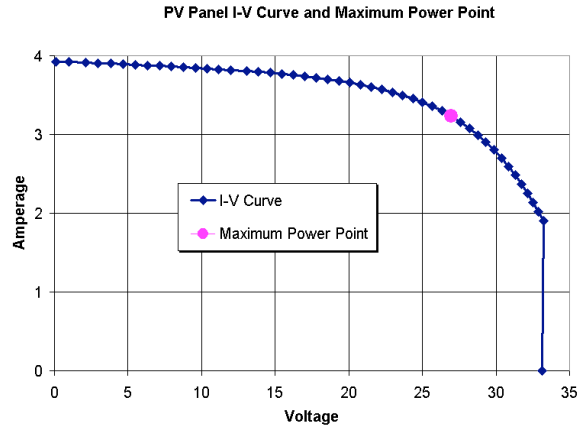


Figure 5: The I-V Curve

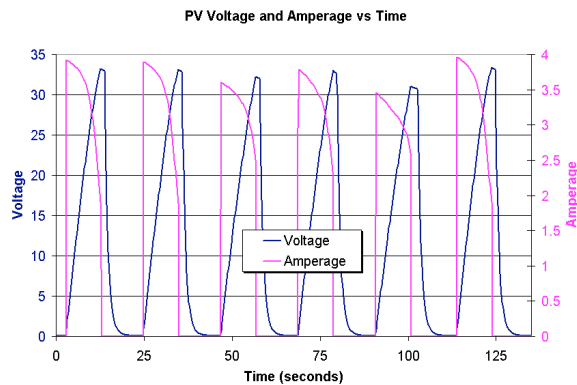


Figure 6: A Sample Set of PV Data

RESULTS

Over the four hour testing period, two hours before solar noon and two hours past solar noon, the PV array collected about 2.4 kilowatt-hours (kWh). Figure 7 shows the energy gain during its sampling interval compared to the available radiation striking the collector surface.

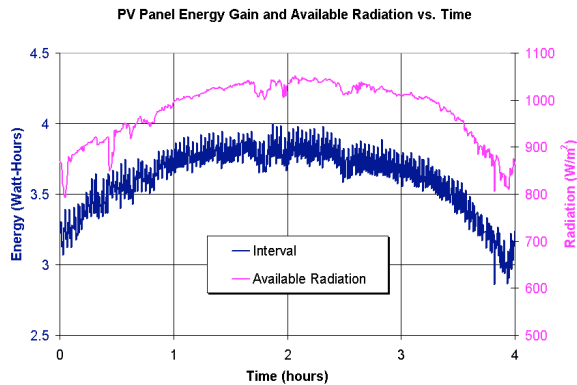


Figure 7: Energy Gain Interval

As expected there is a direct correlation to energy gain and available radiation. Figure 8 shows the same energy gain interval curve with a comparison to the cumulative energy gain throughout the four hour sampling period. As stated before, at hour four the accumulation is 2.4 kWh.

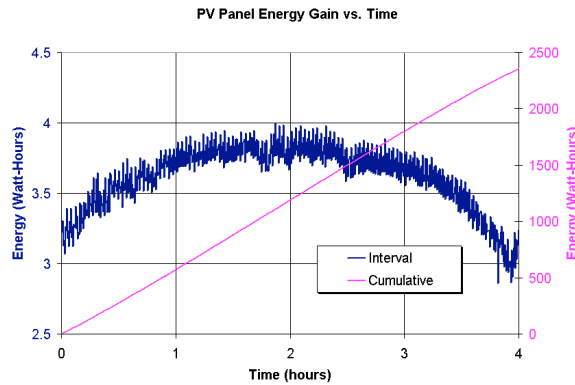


Figure 8: Interval and Accumulation

The efficiency of the solar panels in the STEP system averaged around five percent. Figure 9 is a good representation of PV panel performance on an average sunny day; the daily temperature of all seven test days averaged to 70°F. The two areas being the area only the solar panels encompass and the area of the whole collector as it sits on the roof. Figure 10 is the plot of the PV performance data with the glass removed from the collector.

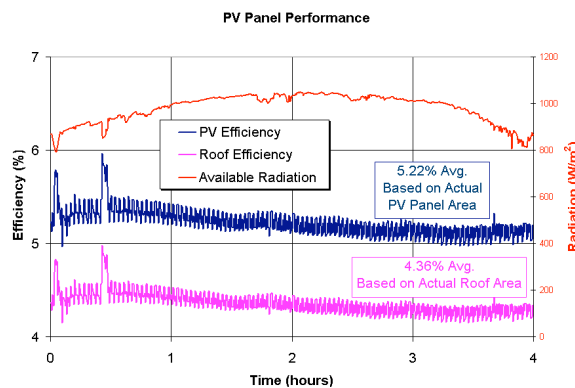


Figure 9: Glazed PV Behavior

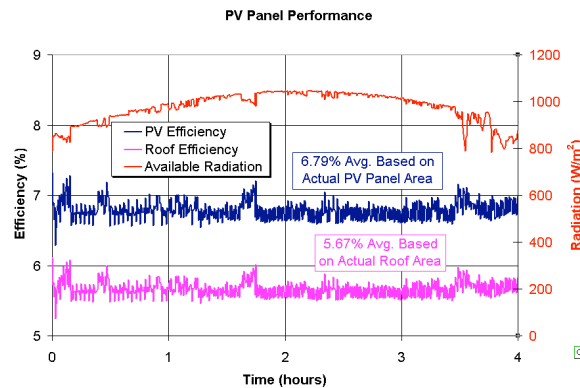


Figure 10: Unglazed PV Behavior

The efficiency of the photovoltaic panels decreases by 23% going from 6.79% to 5.22% a difference of 1.57% due to glazing. This was noted in previous research study at the Energy Research Lab in the Netherlands resulting in the same conclusion (Zondag, 2001). Similar research found the efficiency of the PV array declined with increase in temperature of the module. In an attempt to model this effect through the experimental setup the fluid pump was not run. The glazing is on the collector so the greenhouse effect is still in play. With no fluid flow the PV panels would see a higher temperature. Figure 11 shows the performance of the PV panels with the effects of stagnation. When comparing figures 11 and 9 the effects of stagnation the output by 0.15% due to temperature gain. This, however, does not take into account the effects of extended durations of stagnation.

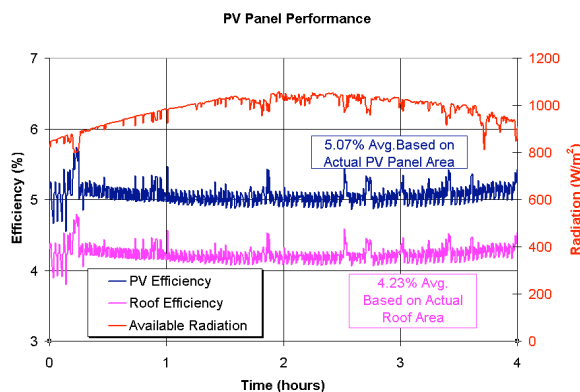


Figure 11: PV Performance with Stagnation

DISCUSSION AND CONCLUSIONS

The overall efficiency of the STEP system is 50% while a separate thermal and separate electric system is estimated to be 26% for the same roof area. The glazed versus unglazed analysis yielded a glazed panel reducing the PV collection by 23% and increasing the thermal collection by 200%.

It has been shown in previous work (Staebler, 1980) that an a-Si panel can benefit from being subjected a pattern of heating a cooling of the panels. This annealing

process can retain a higher efficiency through lowering the degradation of the a-Si panels. Also, cooling a solar panel whether glazed or not, by extracting heat away from it will increase the electrical energy gain of the system (Duffie, 1991).

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Defining Relationships between Vendors and Professional Services in Out-Sourced Civil Engineering Squadrons

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Abstract

In response to several failed facility maintenance contracts, Air Force Space Command initiated a Tiger Team aimed at improving the basic contract structure. The Tiger Team reviewed all out sourced Civil Engineering squadrons in the command, determining that the relationship between professional engineering and architectural services, the vendor, and the client (USAF) had a strong connection to how the contract was perceived. Typically, Air Force bases were strongly satisfied when they either had in-house engineering services or maintained a traditional agent relationship with an architectural /engineering firm. In contrast, the relationships were typically hostile in those contracts where the Air Force severed the traditional agent relationship by allowing the vendor to provide all or most engineering services.

This paper will discuss the agent role of architects and engineer services, the Federal Acquisition Rules which apply, the impact of not maintaining these relationships, and, finally, how AFSPC restructured on Air Force Base's maintenance contract to reflect the traditional professional services relationships.

1. Introduction

Starting with the Total Quality Management initiatives in the 1990's, the US Air Force began innovating with its contract methodologies with concepts like "Performance Based Contracting". While highly effective in addressing the historically hostile relationship between vendors and clients, it failed to address the role between agents and clients. In short, these concepts were applied to Engineering and Architectural (A&E) contracts as the Air Force coped with 30% plus down-sizing. One cost-saving measure was to out-source entire Civil Engineer Squadrons, including agent roles, using performance-based methods. This approach flew in the face of Federal Acquisitions Regulations (FAR) Part 36, which states the Federal Government use "best value" selection to hire architects and engineers, meaning we hire agents based on knowledge and skill rather than performance of skill. This failure to delineate between agent and vendor has created potential conflicts of interest as vendors fight to increase profit while representing the interests of the Air Force.

Before the 1980's, the US Air Force used Federal Acquisition Regulations (FAR) Part 36 to acquire both construction and Architect and Engineer (A&E) services. This FAR section laid out relationships construction vendors and the Air Force client and a fundamentally different relationship between the Air Force and A&E agents. This approach treated vendors as having a 'predatory' relationship where each party attempted to maximize either the amount of service or the amount of profit gained from the contractual relationship. By contrast, the A&E contracts attempted to define a relationship where the agent represented the interests of the US Air Force and focused on the concept of "best value" versus the "best cost" approach of the vendor relationship. The concept of "best value" recognized the importance of experience, education, and professional skill in getting the best solution to the construction and planning needs of the US Air Force.

Starting in the 1980's, innovative thinkers began questioning both the cost and frustration of the traditional approach to vendor relationships with the US Air Force. These innovators recognized that a predatory, year-long contractual relationship with vendors increased cost and hindered the government in gaining the best cost by inflating the risks a contractor assumes in a short-term, fixed priced contract. Additionally, these annual contracts represented a huge government cost and time commitment to award. Congress responded by providing new laws allowing the government contracting community to award contracts, which technically having a one year term, allowed the government to easily award following year contracts ranging from three to seven years. This allowed the contractor to reduce cost by spreading risk over the potential multi-year life of the contract. The government saved because the award of the out years saves significant manpower versus an annual award. With the reward of winning another contract year gave the government the ability to assess the performance of the contract and to further reward the contractor with bonuses for meeting the performance criteria set forth in the contract.

For the most part, these acquisition reforms have allowed the Air Force to maintain its high standards while reducing its military manpower by over 30 percent since 1980. One obvious area to reduce non-war fighting manpower was to out-source significant portions of the Air Force's facilities development and maintenance functions, referred to as Civil Engineering (CE). The Air Force decided to outsource much of the day-to-day CE personnel such as architects and digital control technicians, skill sets having little connection to the Air Force's war fighting needs. By out-sourcing these functions the military gained several advantages. First, by pulling from individuals working in the commercial sector, the military gains more current skills, as compared to in-house personnel who rarely get exposure to commercial practices. Second, the military benefits by escaping from having to pay for generous retirement benefits which potentially begin at the worker's twenty year point.

However, the Air Force continues to need military members with CE skills to support world-wide contingency operations. It is impossible to launch and land

aircraft in remote locations for periods extending for decades without skilled builders and maintainers. The challenge faced by the Air Force was balancing the cost of running both permanent, non-war-fighting, state-side bases which could operate using contractors to handle the vast majority of operations with the needs to man the remote contingency bases with military members skilled in operating a base's infrastructure and facilities.

The clear way ahead was to out-source entire base CE activities at certain bases and to plus up the military presence at the other bases. This approach provided several advantages to the military. At the out-sourced bases, the government was divested of paying out generous benefit packages to governmental employees, enjoyed the commercial practice skills of the vendor, and gained the flexibility of changing out vendors or modifying the contractors based on the needs of the government. The remaining military operated facilities development and maintenance bases gained from having larger numbers of military personnel, which allows for better contingency training (economies of scale) and developing the sense of teamwork needed to operate effectively in remote locations. The Air Force decided which bases to out-source were based primarily on the war-fighting role of the individual base or its owning headquarters. As a result, bases carrying out primarily logistical, procurement, and training activities saw their CE activities out-sourced to civilian vendors. Whereas, bases with bombers, fighters, and other missions directly related to defending the United States saw slight increases in military members assigned to base support.

While these were thoughtful and appropriate responses to balancing declining resources and defending the Nation, the whole-cloth out-sourcing of facility development and maintenance function muddled the construction industry standard American Institute of Architects (AIA) contract relationships by allowing the maintenance vendor to hire engineering services, rather than having the Air Force hire these skills independently. AIA contract documents clearly define the traditional roles of client, vendor, and agent.

The vendor – client relationship is fundamentally predatory. This doesn't imply hostile and antagonistic relationship, but rather that the vendor's goal is to meet the minimum requirements of the contract while maximizing profits. The client's role is to gain the greatest service or goods for the least cost. This is the relationship targeted by performance-based contracting reforms. While not fundamentally changing the predatory nature of the relationship, these reforms aimed at reducing the friction created by this relationship while staying within the acquisition rules set forth by Congress. For example, Congress only allows for single year contract terms. The performance-based approach applies this standard, but allows the Air Force to easily award additional one-year long terms for up to six additional years, if the vendor provides the pre-determined skills sets and performs to the liking of the government. Additionally, the government has the power to award incentive fees for performance above the contract minimums. This long-term relationships benefits both the contractor and the government by fostering teamwork, rewarding improvements,

reducing risk to the contractor, and allowing for simpler, proscriptive contracts (as opposed to the complex, prescriptive contracts demanded before these reforms).

The AIA contract documents define the agent relationship as that of representing the best interests of the client. In many ways, the role of architects and engineers (A&E) is closer to that of lawyer. Their roles ensure the client's facilities and plans meet all the laws of the land and help the client ensure vendors are providing them the appropriate services by applying sound professional knowledge. In many ways, the FAR recognizes the unique role A&Es play in representing the government's interests by stating in 36.601-3 "Sources for contracts for architect-engineer services shall be selected in accordance with the procedures in this subpart..." Why? Because the government hires A&E's using a best value standard which balances cost with the quality of the A&E's professional skills, skills used to ensure the government gets the best quality in its construction and facility maintenance plans and programs.

How do these roles impact the recent out sourcing of CE activities at US Air Force bases? Because the Air Force approached privatizing CE as the out sourcing of a single entity, of which the A&E services were a minor component. As a result, in many of these new contracted CE squadrons, the A&E component now represented the vendor and not the government. Since the Air Force allowed the vendor to hire A&E services as part of an encompassing contract, these engineers and architects are obligated to represent their employer, the vendor and not the Air Force. In fact, these professionals have sworn to a code of ethics which demands they represent the interests of their employer. As part of vendor contract, that interest is to maximize the profits or business position of the vendor within the laws of the land and contract requirements. To put this into easy to understand terms, if the vendor's engineer develops a recurring maintenance plan for all real property installed equipment, the engineer can turn to several sources to determine the maintenance frequencies on this equipment, for example use recommendations from: manufacturers; a maintenance trade organizations, or in-house cost-benefit analysis. The vendor's engineer must choose the frequency with the greatest return for the vendor. If the same engineer worked for the government, the maintenance frequency would be based on best meeting mission and fiscal needs. Worse, since the vendor both develops the plans and programs and carries out the activities, the potential for padding the projects is high. Couple this with limited government oversight and professional qualifications and the vendors have even greater temptations to abuse the contract with little risk of an effective challenge.

In 2006, an Air Force Space Command base requested help in rewriting what they viewed as a failed facilities maintenance contract. As a symptom of how badly the contract relationship was, the \$6 million dollar annual contract usually costing the government in excess of \$10 to \$12 million annually. These cost overruns were due to the contractor identifying projects and repairs deemed to reside outside the contract's scope. Who identified these projects? The vendor's engineers and other employees who have traditionally carried out an agent function for the Air Force.

One exceptionally egregious example was the base fire chief who worked for the vendor. He came to the government representatives complaining that the vendor was pressuring him to identify as many fire violations as possible. The reason? The vendor knew these ‘violations’ had to be repaired immediately and the cost of the repairs was outside the scope of the contract. Compounding the problem was that the government had no control over the hiring and firing of the professional staff, resulting in an engineering staff which did not even speak English and making it very difficult for the government representative to determine the reasons behind engineering decisions. From the perspective of the government representatives, since the vendor’s engineering analysis was based on ‘professional judgment,’ they had little recourse other than accepting the vendor’s position. As the Tiger Team explored these issues, they came to realize how this mixing of professional services and vendor activities created significant potential for conflicts of interest. In exploring these potential conflicts, the Tiger Team recognized that the rift wasn’t between A&E professionals and vendors, but rather along the ‘white collar’ and ‘blue collar’ fissure; managers versus tradesmen. Basically, the Tiger Team came to realize the role of managers and traditional white collar workers is to typically represent the interests of the Air Force, while the blue collar workers carried out highly skilled tasks.

3. Conclusion

As a result of the Tiger Team’s analysis, the affected base split the contract in to two distinct parts – maintenance and management. The maintenance contract focused solely on providing and scheduling tradesmen to carry out both direct scheduled and planned work orders. Besides reducing or effectively eliminating the aforementioned conflicts of interest, the splitting of the contract had several other immediate benefits. First, the former contractor realized they had a monopoly on all facilities maintenance work on the base and could use this position to manipulate the Air Force’s response. If the Air Force terminated the contract, all current and future work was potentially stopped for up to twelve months while a new large contract was let. Since the Air Force staff consisted of only 15 people, they lacked the resources to let small, temporary contracts or step in to do some of the work. By creating two separate contracts, if one of these contracts were to fail, the base now has the luxury of using the other contract to help them get by until a new contract is let. Second, the Air Force benefits by having a professional team representing its interests, agents who have the capability of helping the Air Force team assess and evaluate the performance of other contractors.

PERFORMANCE EVALUATION OF DOX PLANK

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Abstract

This paper presents a literature search on the early developments in floor and roof slabs constructed of “assembled concrete blocks” made from precast, lightweight hollow core blocks, commonly referred to today as Dox Plank. This survey includes period design guides, reference standards, United States patents, technical manuals, product catalogs, plank manufacturers, design assumptions, design theory, and the manufacture of planks. The findings will aid the architect/engineer in conducting a performance evaluation by identifying the original design parameters, such as the allowable unit stresses, area of embedded steel, material properties, etc., as well as by identifying serviceability concerns. Assembled concrete block type planks are formed by assembling a row of hollow core, lightweight concrete blocks into a prestressed plank. The most common of the assembled concrete block systems was the “Dox Plank,” initially known as “Doc’s Blocks.” It was named for its inventor Bernhard A. “Doc” Vander Heyden, who patented improvements in the planks and the methods of constructing them in 1954 (United States Patent No. 2,696,729), and subsequent improvements in 1965 (United States Patent No. 3,172,932). Although Dox Plank is just one variation on the assembled concrete block system, it came to represent the industry.

Introduction

Past performance of old or historic buildings in service can be an indicator of future performance. But when past performance alone is insufficient to evaluate serviceability deficiencies such as floor or roof sag, separation between adjacent planks, differential deflection between adjoining planks, separation of individual blocks within an assembled plank, water penetration through an individual plank or between adjacent planks, corrosion of embedded steel reinforcement, or cracking and spalling are evident or when superimposed concrete overlays, re-roofing, change in building use, addition of dead loads, or renovation has occurred or is being considered, further investigation and analysis may be warranted. When they are

warranted, the analysis can be challenging, if not incomplete, without the basic understanding of the original plank design, materials, and manufacture of the planks.

This paper presents a literature search on the early developments in floor and roof slabs constructed of “assembled concrete blocks” made from precast, lightweight hollow core blocks, commonly referred to today as Dox Plank. This survey includes period design guides, reference standards, United States patents, technical manuals, product catalogs, plank manufacturers, design assumptions, design theory, and the manufacture of planks. The findings will aid the architect/engineer in the performance evaluation by identifying the original design parameters, such as the allowable unit stresses, area of embedded steel, material properties, etc., as well as by identifying serviceability concerns.

Description

Assembled concrete block systems are an assembly of a row of hollow core, lightweight concrete blocks formed to make a prestressed plank. Figure 1 illustrates a typical Dox Plank.

The planks are formed by assembling a row of hollow core, lightweight concrete blocks, placed upside down on a work table, compressing the blocks together, inserting steel reinforcing bars into aligned channels in the longitudinal direction of the row of blocks, grouting the reinforcing into place, curing the assembled plank, and then relieving the externally applied compression, thereby forming a prestressed plank.

Once the plank has cured, it is then turned over and hoisted into position on the job site to form a floor or roof slab. The tongue and grooved edges of the blocks allowed the planks to be keyed with adjacent planks to provide shear transfer, thereby forming a slab. Once the planks were in place, a thin concrete overlay may or may not be cast on the supporting planks. If a topping is cast, it is intended to fill in the grooves between planks resulting in greater strength through composite action.

The assembled concrete blocks had the advantages of being factory made with greater supervision, control, and uniformity than would otherwise be available on the job site. The planks could be quickly erected and once in place serve as a working platform for other trades during construction and as a ceiling for the floor beneath the platform. The hollow cores and lightweight aggregate reduced the overall weight of the slab. The hollow cores had the added advantage of being used for utilities or as air ducts. The underside of the planks formed a flat ceiling which may or may not have been plastered.

Historical Background

Assembled concrete block systems gained popularity in the 1940's and 1950's as an economical alternative to wood and light steel joists for relatively light loads. They

were not recommended for heavy loads or moving machinery, which could cause impact and vibration.

Assembled concrete block systems emerged in an industry attempt to eliminate or minimize the cost of carpentry that was otherwise necessary for cast-in-place concrete formwork. It was an alternative that was particularly well suited for both residential and light commercial construction and especially for schools.

Numerous variations have focused upon the clamping action using threaded rods and end plates, grouting process, external tensioning, arching the row of blocks, interlocking planks, shape of blocks, and the apparatus used in the manufacturing process. Some of the variations and other improvements are described more fully in United States Patent Nos. 1,891,597 (Jagdman, 1932), 1,921,285 (Davis and Brush, 1933), 2,044,382 (Dunagan, 1936), 2,075,633 (Anderegg, 1937), 2,299,070 (Rogers and Price, 1942), 2,631,450 (Lachaise, 1953), 2,645,115 (Abeles, 1953), 2,696,729 (Vander Heyden, 1954), and 3,172,932 (Vander Heyden, 1965). Most are structurally indistinguishable. Cataloging these variations is beyond the scope of this paper.

The system that attracted the most attention was Dox Plank, initially known as "Doc's Blocks." It was named for its inventor Bernhard A. "Doc" Vander Heyden, who patented improvements in the planks and in the methods of constructing them in 1954 and subsequent improvements in 1965. He also founded the Dox Plank Manufacturers Association, which licensed manufacturers across the eastern half of the United States (Figure 2). Although Dox Plank is just one variation on the assembled concrete block system, it came to represent the industry.

In 1954, the Precast/Prestressed Concrete Institute was formed. By then, the prestressed concrete industry had already emerged. With the advent of prestressed concrete and the production of high strength steel reinforcement (especially wire strands), and the increased compressive strength of concrete, prestressed concrete improved upon the disadvantages of conventionally reinforced concrete that had used assembly hollow core block. This naturally led to its decline and eventually discontinuation.

Minimum Standard Requirements for Precast Concrete Floor and Roof Units

Around the 1920's, the American Concrete Institute (ACI) tasked its newly formed Committee 711 to undertake a critical examination of precast concrete joist, superimposed concrete floor systems, and construction methods on the state of the art with recommendations for minimum standard requirements. Committee 711's report titled, "Precast Joist Concrete Floor Systems," was submitted to ACI on July 25, 1929. The report was later published by ACI in January 1940. It pertained to three systems: 1) precast concrete joist with cast-in-place slab, 2) precast concrete joist with precast concrete slab, and 3) independent precast joist with concrete slab (i.e., precast slab that is not bonded to the joist).

The 1940 report of the ACI Committee 711 evolved into the “Proposed Minimum Standard Requirements for Precast Concrete Floor Units,” which was later published in 1944 (ACI, 1944) and revised again as proposed standards in 1946 (ACI, 1946).

In October 1946, ACI adopted the standard on “Minimum Standard Requirements for Precast Concrete Floor Units (ACI 711-46)” as a supplement to the ACI “Building Regulations for Reinforced Concrete (ACI 318-41).”

In November 1952, ACI Committee 711 first recognized the popularity of assembled concrete block systems and specifically incorporated them into its proposed revision of the standard (ACI, 1952) along with the revision to reference the updated ACI standard, “Building Regulations for Reinforced Concrete (ACI 318-51).” By that time, concrete block systems had been widely used and some types of planks and manufacturing methods had already been patented. In the case of Dox Plank, the application for the first of two patents had already been filed.

Initially, ACI Committee 711 (ACI, 1952) proposed subdividing the manufacture and assembly of concrete blocks into essentially arched and prestressed systems. It distinguished the two in stating those: 1) “With contact faces between units ground to provide uniform bearing between units and to provide a slight camber to the assembly,” and 2) “With contact faces parallel, but with a tension in the lower moment bars sufficient to align and hold the assembly together and to provide slight camber.” Subsequent, Dox Plank patents incorporated both the arching and prestressing.

In 1953, ACI adopted the proposed revisions of Committee 711 (ACI, 1953). It became the first ACI standard specifically devoted to assembled concrete blocks. It is also devoted to segmental concrete, which includes tee-beam joists, concrete joists, and I-beam joists. The Standard specifies concrete protection for reinforcement, materials, manufacture, tests, and other requirements.

In 1957, proposed revisions of ACI 711-53 incorporated reference to the ACI “Building Regulations for Reinforced Concrete (ACI 318-56).” The proposed revisions of 1957 were adopted as part of ACI 711-58. Table 1 provides the Allowable Unit Stresses in Concrete that were published in ACI 711-58. Finally, upon the completion of the ACI Standard, “Minimum Standard Requirements for Precast Concrete Floor and Roof Units (ACI 711-58),” ACI Committee 711 was discharged.

With the adoption of the “Building Code Requirements for Reinforced Concrete (ACI 318-63),” which contained both Working Stress and Ultimate Strength Design methods and also extensive changes in the treatment of shear and bond, ACI-ASCE Committee 512 on Precast Structural Concrete Design and Construction was formed to review ACI 711-58 and to recommend any necessary changes.

The result of ACI-ASCE Committee 512's review was the proposed standard, "Recommended Practice for Manufactured Concrete Floor and Roof Units," which supplemented ACI 318-63 and superseded ACI 711-58.

Most of the specifications given in ACI 711-58 are contained in the report by ACI-ASCE Committee 512 (ACI-ASCE, 1966). The major additions concerned test methods for strength design and ultimate strength design, minimum reinforcement requirements, and quality requirements and acceptance procedures.

In 1967, ACI adopted the standard, "Recommended Practice for Manufactured Reinforced Concrete Floor and Roof Units (ACI 512-67)". It later evolved into "Precast Structural Concrete in Building" (ACI-ASCE, 1974).

Dox Plank Patents

On June 19, 1944, Bernhard A. "Doc" Vander Heyden filed an application, which was a continuation of an earlier application filed on May 17, 1943, with the United States Patent Office for improvements in prestressed planks using concrete blocks and the method of manufacture. A patent (Patent No. 2,696,729) was granted on December 14, 1954.

The stated objective given in Patent No. 2,696,729 was, "To provide a strong but light and porous cementitious plank combining great strength with a high thermal and acoustical insulation factor, which together with its light weight, adapts it for making of roofs and floors, and the like." The patent identified 13 claims distinguishing it from other patents using concrete and clay tile block and their manufacture.

Although the distinctions between Patent No. 2,696,729 and other patents pertaining to planks with concrete blocks and their manufacture and also the distinction among the 13 claims made by this patent maybe subtle, most important to the objective of this paper is not the distinctions, but rather the insight it reveals into the details of the planks and their manufacture.

The provisions of Patent No. 2,696,729 state that the planks are to be supported at their ends and that the planks are manufactured by grouting reinforcing bars throughout the length of the planks, so as to distribute the compressive stress to individual blocks rather than concentrating it upon the end blocks. The reinforcing bars are not threaded nor do they extend beyond the end blocks. The provisions also state that the manufacture provides a means for grinding the face of individual blocks, aligning the channels and hollow cores within a row of blocks, placing reinforcing bars at predetermined positions in the plank, compressing the assembly of blocks, grouting the reinforcing bars while the assembly of blocks remain in compression, and transferring compressive stress in the blocks through bond with the reinforcing bars thereby forming a prestressed plank. The provisions also provide a process for adjusting a plank to correct sag or to create an arch.

On December 29, 1960, Mr. Vander Heyden filed an application for subsequent improvements in the method of manufacturing a concrete plank. The patent was granted on March 9, 1965 (Patent No. 3,172,932). It identified ten claims pertaining to prefabricated or drilled access ports for pressure grouting in special blocks provided at predetermined intervals along the length of the plank and to provide a step-by-step process of grouting.

Technical Manuals and Standard Specifications

The Dox Plank Manufacturer's Association licensed those manufacturers complying with Patent No. 2,696,729 or its patent application. Figure 2 lists the licensed Dox Plank manufacturers. Each manufacturer produced its own technical manual, which included standard specifications, design assumptions, design tables, and installation details.

The standard specifications indicate that the design was performed in accordance with ACI 711-46, ACI 711-53 or ACI 711-58 and the corresponding building regulations given in ACI 318-41, ACI 318-51, or ACI 318-56, respectively. The specifications also indicate that the concrete strength of the concrete block and topping shall be 2,500 pounds per square inch or greater and 3,000 pounds per square inch, respectively. For concrete toppings, the toppings shall be reinforced with welded wire fabric. On roof slabs and other areas where no concrete topping is required, a grout coat of 1 to 3 cement and sand mix shall be thoroughly swept into the top surface of the plank after it is placed. The bearings shall provide a minimum of 3 inches on masonry and concrete beams and 2-1/2 inches on structural steel.

Reinforced Concrete Design

The assumptions used in the load capacity design are:

1. The maximum tensile stress of reinforcing steel is 20,000 pounds per square inch
2. The maximum compressive stress of concrete blocks is 1,125 pounds per square inch (Table 1)
3. The maximum compressive stress of concrete toppings is 1,350 pounds per square inch (Table 1)
4. The concrete topping slab weighs 145 pounds per cubic inch
5. The ratio of the net area to gross area of a typical plank is 0.55

The design for bending of simple span planks under uniformly distributed load using the Working Stress Design method yields the following standard formula:

$$M_s = A_s f_s j d \quad (1)$$

$$M_c = f_c \left(1 - \frac{t}{2k d}\right) b t j d \quad (2)$$

$$V_c = v_c b' j d \quad (3)$$

where the values of j and k have been derived from a T-beam section in accordance with the ACI Reinforced Concrete Design Handbook and as illustrated in Figure 3,

$$j = 1.0 - \left[\frac{\left(k - \frac{2t}{3d}\right)}{\left(2k - \frac{t}{d}\right)} \times \frac{t}{d} \right] \quad (4)$$

$$k = \frac{\frac{n A_s}{b t} + \frac{t}{2d}}{\frac{n A_s}{b t} + 1.0} \quad (5)$$

The design summary for standard 8 x 16 plank section with topped and untopped sections is tabulated in Tables 2 and 3, respectively. Applying basis engineering principles for simple span beams, the available uniformly distributed load may be computed. Tables 4 and 5 give the available uniform load for 8 inch x 16 inch planks with various reinforcing bar sizes and span lengths.

Guidelines for Performance Evaluation

In conducting a performance evaluation of an existing old or historic building in service constructed of “assembled concrete blocks” made from precast hollow core blocks, and in particular Dox Plank, the author recommends that the architect/engineer consider the following guidelines in the performance evaluation:

1. The “prestressing” cited in the reference standards, patents, and technical manuals is a misconception and should not be confused with conventional prestressing used today. Although the reinforcing bars were tensioned prior to being placed into service, assembled concrete blocks and in particular Dox Planks, are conventionally reinforced. Incidental prestressing used in Dox Plank is insignificant to long-term performance and since prestressing could not be considered permanent, it was therefore not used as a factor in the design. The magnitude of the prestress in the reinforcing steel was about 20,000 pounds per square inch, far less than conventional prestressing of about 150,000 pounds per square inch used today.
2. The design equations for bending and shear are given in Equations 1-5 above. These equations were derived from design for bending of simple span planks under uniformly distributed load using the Working Stress Design method and T-beam section in accordance with the ACI Reinforced Concrete Design Handbook.
3. Deviations from original construction or subsequent field modifications, such as the erection of intermediate bearing wall(s) to support simple span planks, openings made in the planks to accommodate stairways or ducts, concrete overlays that increase the thickness of the topping slab, changes in applied

loads from roof top mechanical units, or re-roofing over an existing roof may change the load distribution and may decrease the available live load capacity. Often the available live load is minimal and there is little margin for additional loads.

4. The original design parameters, such as the allowable units stress, area of embedded steel reinforcement, and material properties are provided herein.
5. Notable serviceability deficiencies, such as floor or roof sag, separation between adjacent planks, differential deflection between adjoining planks, separation of blocks within an assembled plank, water penetration through an individual plank or between adjacent planks, corrosion of embedded steel reinforcement, or cracking and spalling, cracks in the concrete topping running parallel to the joint formed between side-by-side planks are indicators of performance.

Conclusions

This paper raises the awareness of the performance evaluation of assembled concrete blocks, and in particular Dox Plank, used in existing floor and roof construction. Guidelines for the performance evaluation of Dox Plank are presented based up a literature search.

This paper did not include a discussion of segmental concrete or cantilevered planks. Although the original Dox Plank design did permit planks to be used as cantilevers, only simple span construction is considered in this paper.

Notation

The following symbols and abbreviations were used in this paper:

- b = width of plank (in)
- d = effective depth of section (in)
- d_b = rebar diameter (in)
- d_c = depth of cover (in)
- f'_c = compressive strength of concrete (psi)
- f_c = extreme fiber stress of concrete (psi)
- f_s = extreme fiber stress of steel (psi)
- n = modular ratio, $E_s/E_c = 30,000/f'_c$
- t = thickness of compression block (in)
- A_s = tensile steel area (in²)
- E_c = modulus of elasticity of concrete (psi)
- E_s = modulus of elasticity of steel (psi)
- L = span length (ft)
- M_c = maximum bending moment of concrete section (lb ft)
- M_s = maximum bending moment of steel section (lb ft)

Acknowledgements

The author wishes to thank Ms. Penny Sympson, Corporate Librarian, Mr. Mark R. Chauvin, PE, Senior Associate, and Mr. Dennis Rolander, PE, Senior Associate for their assistance in research and guidance; to Mr. David S. Finley, EIT, Associate II for his assistance in numerical analysis, research, and preparation of figures; and to Mr. Keith J. Niedrist for preparation of the figures. All are with Wiss, Janney, Elstner Associates, Inc.

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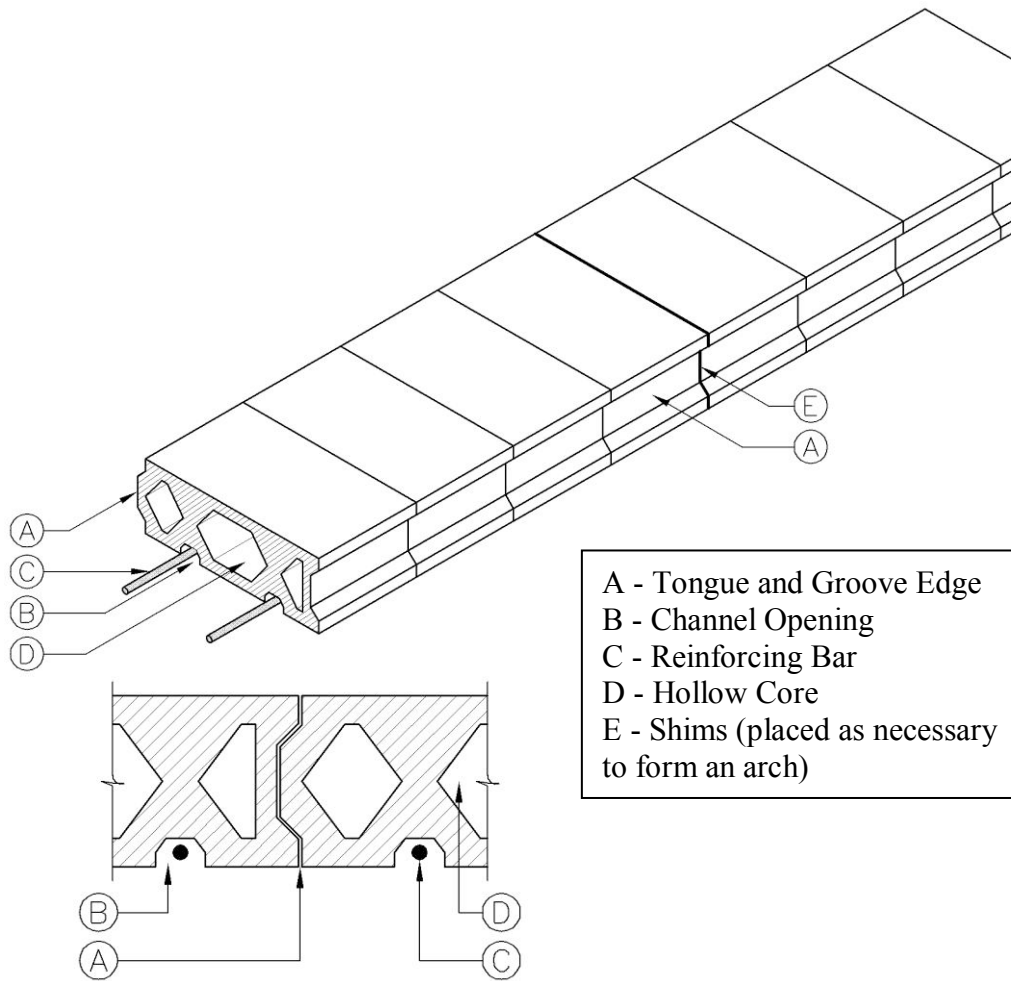


Figure 1. Typical Dox Plank



Connecticut
 Plasticrete Corp.
 Hamden, Connecticut

District of Columbia
 Nabco Plank Co.
 Washington, DC

Florida
 Maule Industries, Inc.
 Miami, Florida

Maule Industries, Inc.
 Miami Beach, Florida

Illinois
 Doxplank of Wisconsin, Inc.
 Glenview, Illinois

Maryland
 Baltimore Concrete Plank Corp.
 Baltimore, Maryland

Massachusetts
 Doxplank of Massachusetts
 Medford, Massachusetts

Michigan
 Wm. Moors Concrete Products
 Fraser, Michigan

Minnesota
 Doxplank of Minnesota
 (Dox Block System)
 St. Paul Park, Minnesota

New Jersey
 Doxplank Co., Inc.
 (Multiplex Concrete Co., Inc.)
 East Orange, New Jersey

New York
 Cossitt Concrete Products, Inc.
 New York

Doxplank of New York, Inc.
 North Chili, New York

Latta Brook Corp.
 Elmira, New York

Mid-State Concrete Plank, Inc.
 Hamilton, New York

Schaefer Brothers Co., Inc.
 Rochester, New York

Ohio
 Cleveland Builders Supply Co.
 Cleveland, Ohio

Columbia Concrete Products
 Toledo, Ohio

Pennsylvania
 Baltimore Concrete Plank Corp.
 Bala Cynwyd, Pennsylvania

Baltimore Concrete Plank Corp.
 Philadelphia, Pennsylvania

Doxplanks of Northeastern PA
 (Dox's System of Pennsylvania)
 Portage, Pennsylvania

Nelville Concrete Pipe Co.
 (Neville Cement Products Corp.)
 Pittsburgh, Pennsylvania

Tennessee
 Knox Concrete Products, Inc.
 Knoxville, Tennessee

Texas
 Valley Builders Supply Co.
 Pharr, Texas

Wisconsin
 Fond Du Lac Concrete Products
 Fond du Lac, Wisconsin

Vander Heyden, Inc.
 Milwaukee, Wisconsin

Figure 2. Licensed Dox Plank Manufacturers

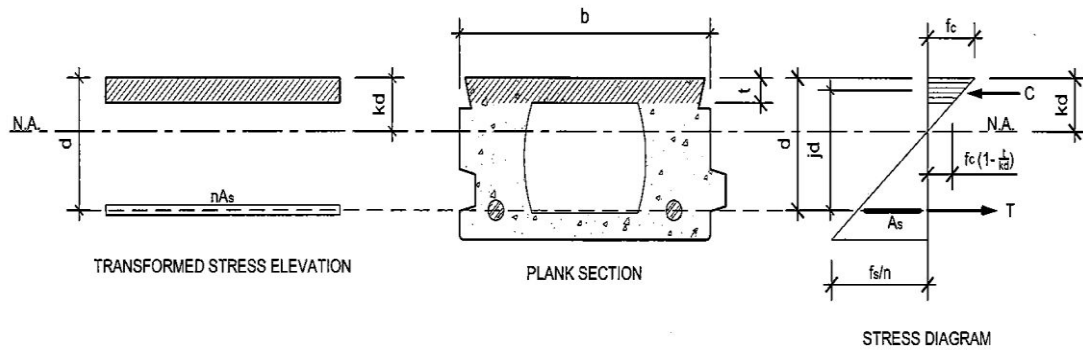


Figure 3. Stress Diagram

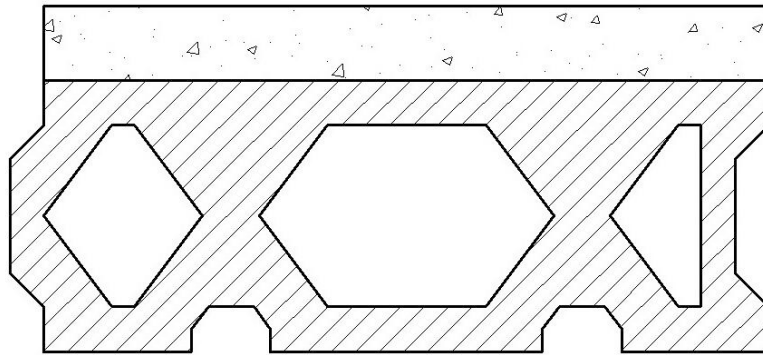
Table 1. Allowable Unit Stresses in Concrete
(Based on ACI 711-58)

Description	Allowable Unit Stresses, (psi)					
		For Any Strength of Concrete as Fixed by Test in Accordance with ACI 711 Section 302	When Strength of Concrete is Fixed by the Water-Content in Accordance with ACI 711 Section 302			
			Compressive Strength, f'_c (psi)			
			2,000	2,500	3,000	3,750
		$n = \frac{30,000}{f'_c}$	$n = 15$	$n = 12$	$n = 10$	$n = 8$
Flexure, f_c Extreme fiber stress in compression	f_c	$0.45 f'_c$	900	1,125	1,350	1,688
Shear, v Flat slabs at distance, d , from the edge of column capital or drop panel	v_c	$0.03 f'_c$, but not to exceed 75 psi	60	75	75	75

Table 2. Dox Plank Section Data - 8 x 16 Plank Section, Topped^a

Design Data:

$n = 10$
 $v_c = 75 \text{ psi}$
 $w_{DL} = 69 \text{ psf}$
 $f'_c = 3,000 \text{ psi}$
 $f_c = 1,350 \text{ psi}$
 $f_s = 20,000 \text{ psi}$
 $u = 300 \text{ psi}$
 $E = 3,000,000 \text{ psi}$
 $b' = 7.06 \text{ in}$
 $t_{topping} = 2 \text{ in}$



Std. Sect.	Tensile Steel Area	Effect. Depth			Controlling Moments		Controlling Shears			Moment of Inertia
	A_s	d	k	j	M_s	M_c	V_c	V_u	V_h	I
	(in^2)	(in)			($\frac{\text{ft} \cdot \text{lb}}{\text{ft}}$)	($\frac{\text{ft} \cdot \text{lb}}{\text{ft}}$)	($\frac{\text{lb}}{\text{ft}}$)	($\frac{\text{lb}}{\text{ft}}$)	($\frac{\text{lb}}{\text{ft}}$)	($\frac{\text{in}^4}{\text{ft}}$)
833T										
834T										
844T	0.40	8.81	0.212	0.930	4,097	10,286	3,254	5,792	4,913	170.6
845T	0.51	8.77	0.236	0.922	5,155	11,287	3,211	6,430	4,840	207.2
855T	0.62	8.75	0.258	0.917	6,218	12,067	3,187	7,088	4,810	242.0
856T	0.75	8.71	0.283	0.911	7,439	12,726	3,151	7,711	4,759	278.9
866T	0.88	8.69	0.306	0.908	8,680	13,295	3,134	8,366	4,733	314.1
867T	1.04	8.65	0.333	0.905	10,177	13,802	3,109	8,992	4,686	352.5
877T	1.20	8.63	0.357	0.902	11,676	14,186	3,091	9,629	4,674	389.1

Notes: ^a Tabular data was compiled from Dox Plank catalogs.

Table 3. Dox Plank Section Data - 8 x 16 Plank Section, Untopped^a

Design Data:

$n = 18$

$v_c = 75 \text{ psi}$

$w_{DL} = 45 \text{ psf}$

$f'_c = 2,500 \text{ psi}$

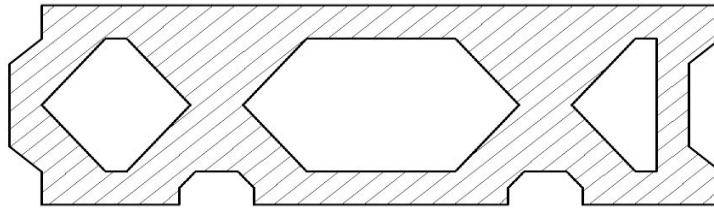
$f_c = 1,125 \text{ psi}$

$f_s = 20,000 \text{ psi}$

$u = 300 \text{ psi}$

$E = 1,666,667 \text{ psi}$

$b' = 7.06 \text{ in}$



Std. Sect.	Tensile Steel Area	Effect. Depth			Controlling Moments		Controlling Shears		Moment of Inertia
	A_s	d	k	j	M_s	M_c	V_c	V_u	I
	(in^2)	(in)			($\frac{\text{ft} \cdot \text{lb}}{\text{ft}}$)	($\frac{\text{ft} \cdot \text{lb}}{\text{ft}}$)	($\frac{\text{lb}}{\text{ft}}$)	($\frac{\text{lb}}{\text{ft}}$)	($\frac{\text{in}^4}{\text{ft}}$)
833	0.22	6.88	0.241	0.927	1,754	5,588	2,533	3,381	98.9
834	0.31	6.83	0.290	0.923	2,443	6,064	2,504	3,899	127.9
844	0.40	6.81	0.332	0.919	3,129	6,371	2,485	4,424	153.7
845	0.51	6.77	0.378	0.917	3,958	6,600	2,465	4,936	180.1
855	0.62	6.75	0.418	0.916	4,792	6,765	2,455	5,462	203.5
856	0.75	6.71	0.459	0.915	5,756	6,882	2,438	5,967	225.8
866	0.88	6.69	0.494	0.914	6,726	6,974	2,428	6,483	245.8
867	1.04	6.65	0.532	0.913	7,893	7,027	2,411	6,974	265.2
877	1.20	6.63	0.564	0.912	9,070	7,074	2,401	7,480	282.8

Notes: ^a Tabular data was compiled from Dox Plank catalogs.

Table 4. Dox Plank Uniformly Distributed Live Loads (*psf*) -
8 x 16 Plank Section, 2 inch - 3,000 psi Concrete Topping

Std. Sect.	Span (<i>ft</i>)												
	12	13	14	15	16	17	18	19	20	21	22	23	24
833T													
834T													
844T	159	125	98	77	59	44	32	21					
845T	217	175	141	114	92	74	58	45	34	24			
855T	276	225	185	152	125	103	85	69	55	44	33	25	
856T		283	235	195	163	137	115	96	80	66	54	43	34
866T			285	239	202	171	145	123	104	88	74	62	51
867T				292	249	212	182	156	134	115	99	84	72
877T					295	254	219	189	164	142	123	107	93

Table 5. Dox Plank Uniformly Distributed Live Loads (*psf*) -
8 x 16 Plank Section, Untopped

Std. Sect.	Span (<i>ft</i>)												
	12	13	14	15	16	17	18	19	20	21	22	23	24
833													
834	91	71	55	42	31	22							
844			83	66	53	41	32	24					
845				95	78	64	52	42	34	26			
855						87	73	61	50	41	34	27	
856							96	81	70	59	50	42	34
866								88	75	65	56	50	44
867									95	82	71	61	54
877										88	76	65	58

Lessons Learned for Exterior Insulation Finish Systems

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Abstract

As trained architects and engineers, we are privileged to have access to building performance failures of all types on a daily basis. Certainly, the presence and infiltration of water, in any of its many forms – such as moisture, precipitation, vapor, and humidity – are responsible for the vast majority of building performance failures. During the course of our presentation, we will use a building system failure that showcases many common deficiencies that cause damage. This case study concerns an Exterior Insulation and Finish System (EIFS) but the problems also occur in many other types of envelope systems

Introduction

Most people realize there is a design or construction flaw in a building when they first experience a flaw in the way they expect the building to perform. There are generally many deficiencies in a building, but the only problems that prove to be of concern are when they result in aesthetic or structural damage. Using the exterior skin of a building as an example, the easiest way to find a performance issue is perusing the façade and building interior, noting any evidence of water infiltration and potential points of water entry such as cracks and unsealed joints.

The case study we will use to discuss many of the common issues that arise with an EIFS building is an investigation that was completed in 1998. A series of surveys and water tests were performed on one building in a complex of several structures to determine causation of the observable and hidden damages.

The complex of discussion was built in northern California between 1994 and 1995 and consisted of approximately 34,600 square feet contained within three buildings. All three buildings were single-story wood-frame structures on shallow concrete foundations with slab-on-grade floors. The exterior wood stud walls were covered

with 3/8" plywood sheathing, glass fiber faced gypsum sheathing, and EIFS. For the purpose of this paper, we will be focusing only on the observations regarding and related to the EIFS.

The EIFS consisted of up to 4" expanded polystyrene (EPS) insulation adhered to glass fiber faced gypsum sheathing, which was mechanically fastened to the underlying wood framing. The EPS was then covered with a polymer based (PB) base coat ranging from 0.065 to 0.225 inches, depending on risk of impact (The closer to the ground or higher the risk of impact, the thicker the EPS). The window and door units were screwed into the wood framing prior to the installation of the EIFS. The joints between the EIFS and the fenestration units were sealed to the EIFS finish coat with a urethane-based sealant. A typical detail of the EIFS is shown in Figure 1.

To accomplish an accurate assessment of the building, Exponent conducted interior and exterior surveys as well as water testing and destructive openings at representative locations.

Interior and Exterior Surveys

Interior and exterior surveys of a building generally consist of inspecting as many surfaces as possible, noting anything that seems to be atypical. Common items to note are staining, cracking, saturation, organic growth, deformation, and degradation.

The main observations of the interior survey of the building revealed:

- Approximately 50 independent gypsum wallboard stains on the interior finishes in the vicinity of the window units
- Extensive water staining and saturation on the carpet in the vicinity of high traffic exterior doors

The main observations of the exterior survey of the building revealed:

- Several locations with an improper sealant joint and occasionally failure of the sealant joint between the windows and the EIFS (Figure 2)
- Cracking in the EIFS at the corners of wall openings (Figure 3)
- Vertical cracking at the base of the EIFS (Figure 3)
- EIFS sill and pilaster slopes did not meet manufacturer's recommendation
- Penetrations through EIFS were not sealed per manufacturer's recommendations (Figure 4)
- Overall, there was a high exterior grade level relative to the EIFS finish, with several locations in which the EIFS was in contact with the soils and some locations where the exterior flatwork was in direct contact with the EIFS (Figure 5)
- Drainage was directed towards the building in various locations, creating excess ponding of water rather than directing excess water away from the building

Water Testing and Destructive Openings

Two rooms were selected for water testing where water staining and mold growth was visible on the interior drywall finishes near the windows. The interior drywall finishes were removed to allow easier observation of water leakage. The EIFS and window system were spray tested under atmospheric pressure according to AAMA Standard 501.2, *Field Check of Metal Curtain Walls For Water Leakage*. Following the requirements of this standard, the EIFS and sealant joints were tested independently, with the component that was not being tested masked off with polyethylene plastic and duct tape during the testing.

Steady water infiltration was observed at the inside corner of the EIFS sill at a joint in the underlying EPS insulation (Figure 6). Minor water infiltration was observed at the window sealant jamb and head joints, where a backer rod was not installed as specified because the rough window opening was not large enough to permit the installation of both a backer rod and sealant. Significant water leakage was observed at the EIFS sill and window sill extrusion joint (Figure 7), where a lack of sealant due to insufficient gap between the two materials was observed (Figure 2).

Exterior destructive openings were cut to determine typical conditions. These openings revealed poor base coat adhesion to underlying insulation, advanced decay of the plywood sheathing substrate and wood framing, and poor or no adhesion of the EPS insulation to the Dens-Glass Gold sheathing board. In fact, the adhesive was found in some cases to have been applied to the glass fiber faced gypsum sheathing instead of to the back of the EPS (as directed by the manufacturer) and the EPS board was not pressed into the adhesive during installation, as evidenced by the ribs that were still apparent (Figure 8).

Summary of Findings

Problem Observed	Cause	Figure
Insufficient or lack of sealant at EIFS sill and window sill extrusion joint	Joint should have been designed and constructed correctly providing the ability to install sealant properly and prevent water infiltration at a susceptible area.	2, 7
Cracking at EIFS opening corners	Diagonal reinforcing mesh should have been installed to minimize cracking.	3, 6
Cracking at the base of the EIFS	The EPS boards should have been plumb with one another, and a sliver of EPS material should have been used, if necessary, to eliminate gaps.	3, 6

Problem Observed	Cause	Figure
EIFS penetrations were not sealed in accordance with manufacturer's details	Sealing of EIFS penetrations should be in accordance with manufacturer's details and include flashing where necessary to prevent water infiltration at the penetration.	4
EIFS does not have sufficient clearance from grade	Moisture from contact with grade and landscaping materials increases the likelihood of degradation of the EIFS.	5
Poor adhesion between the EPS insulation and underlying glass fiber faced gypsum sheathing	Adhesive should have been applied to the EPS insulation and the EPS then should have been pushed firmly onto the gypsum sheathing	8
No adhesion was observed at some areas between the EPS and the substrate	The EPS must be adequately adhered to the substrate to prevent premature detachment of the EIFS.	8
Leak at EIFS sill	The EIFS sill was installed too close to the window sill preventing the installation of a proper sealant joint.	-
Backer rod was not installed for sealants applied around the window	Rough openings should have been large enough to accommodate a backer rod to allow proper sealant geometry.	-
Slope of EIFS sill and parapets was too shallow	EIFS slopes should be installed with a minimum slope of 6:12 to enhance drainage.	-

Other Field Observations

In addition to the items noted in the given example, there are several other common problems associated with EIFS installation and maintenance to be aware of when using this exterior envelope system:

Problem	Why
Aligning substrate joints with EPS joints	The building system is more susceptible to cracking at joints in underlying materials.
Edge of EPS installed too close to wall openings at corners	The corners of wall openings are the most susceptible areas for cracks due to building movement and settling. Adding an additional joint in the underlying wall system at that location dramatically increases the likelihood of cracking.
Standard mesh used throughout entire project	The system itself can be easily damaged by impacts. High-impact risk areas should have a higher impact mesh to decrease the likelihood of damage caused by these expected impact.
Improper field conditions for installation	The field conditions need to be appropriate in temperature, precipitation, and wind, or the quality and performance of the product, specifically adhesion, can be affected,

Problem	Why
Treating each exterior wall face as an independent wall	The EPS boards must be interlocked at the building corners because increasing the linear footage of any underlying joint increases the likelihood of a crack forming in the base or finish coat.
Inadequate overlap of mesh	Mesh that is used as the base for the EIFS should be overlapped a minimum of 2 1/2" to decrease the likelihood of cracks forming at joints in the mesh.

Conclusion

The EIFS system examined during this investigation could have performed properly if attention to manufacturer and industry standards had been followed. The cause of the vast majority of the wall system water intrusion on this building is the lack of attention to important details such as the window-EIFS interface. Once water enters an undrained EIFS it is slow drying out. The field problems discussed in this paper are by no means exhaustive, but they are a list of many common problems encountered in the field.

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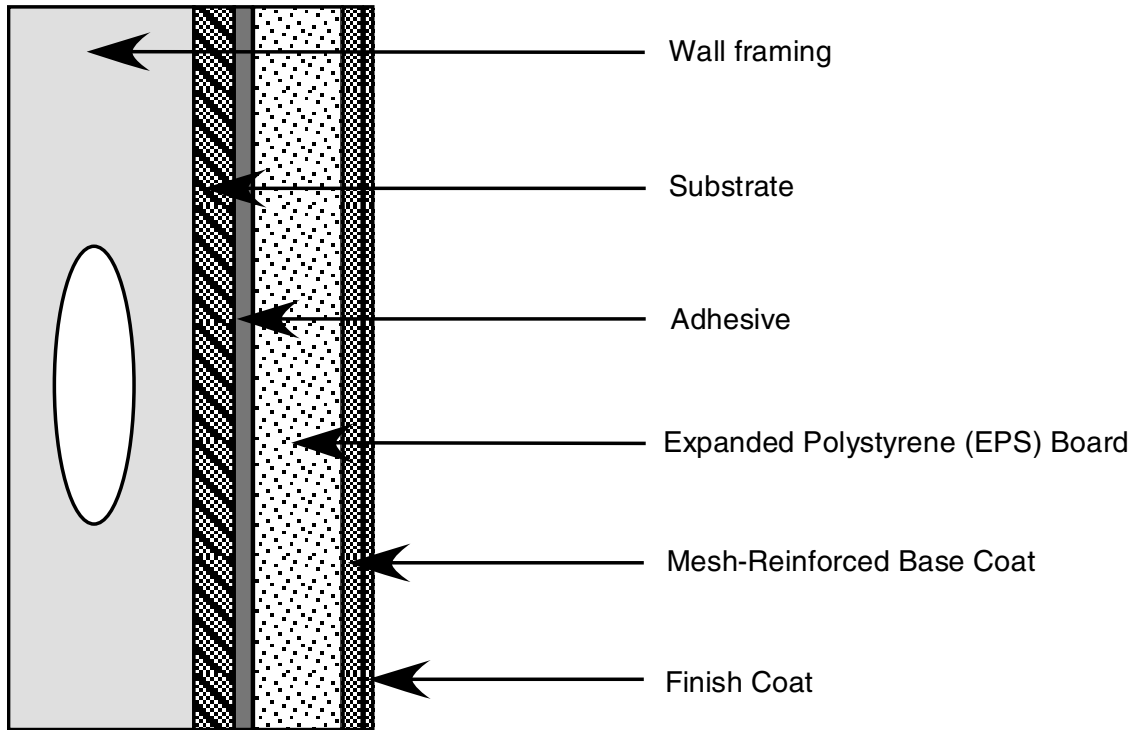


Figure 1: Basic EIFS detail

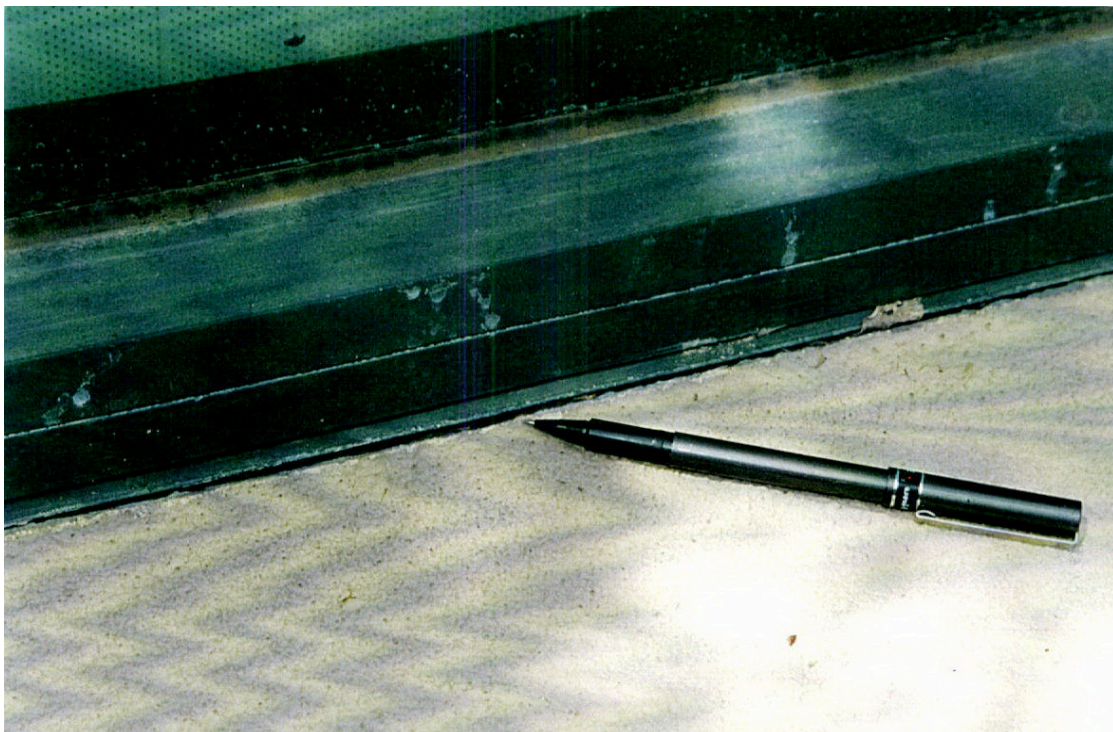


Figure 2: Lack of sealant at window sill and EIFS sill extrusion joint



Figure 3: Typical cracking visible at the corners of openings in the wall system; cracking at the base at an EPS joint

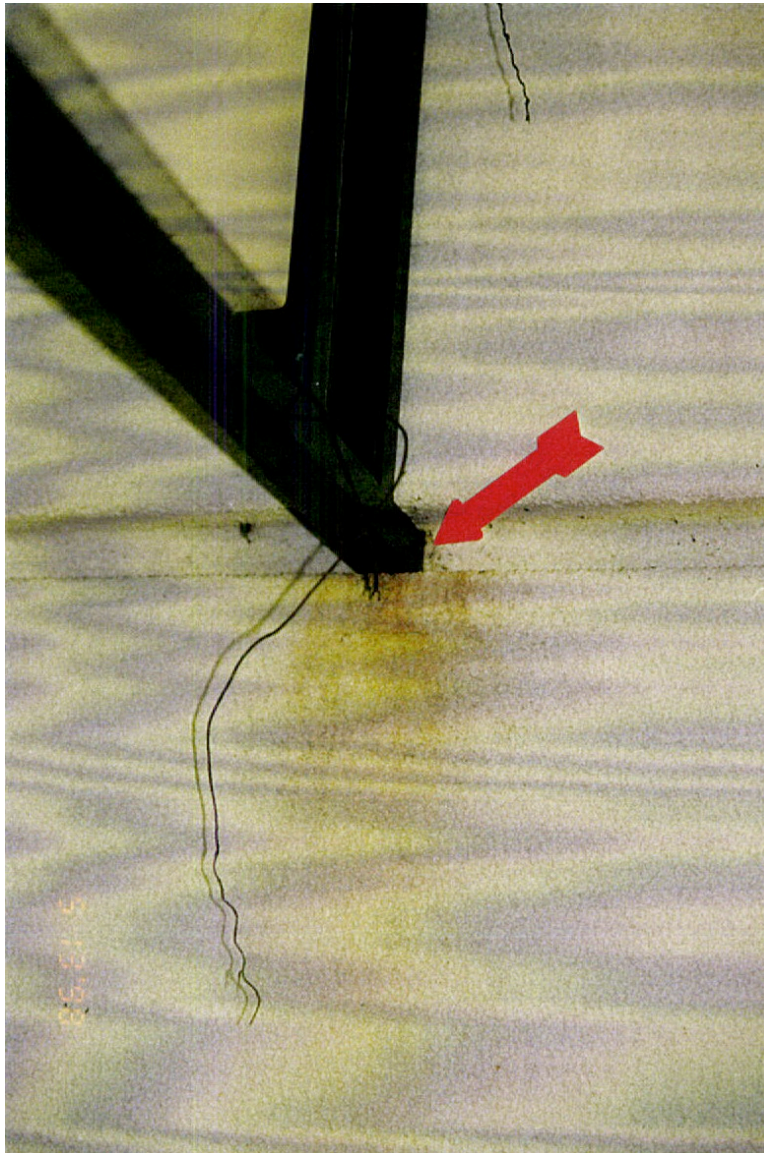


Figure 4: EIFS penetrations not sealed per manufacturer's recommendation



Figure 5: EIFS located too close to grade and more susceptible to water infiltration and premature degradation



Figure 6: Exterior destructive examination revealed an underlying cause of cracking near the base of the EIFS was a gap in the EPS boards, which should have been installed plumb, with an EPS sliver wherever necessary

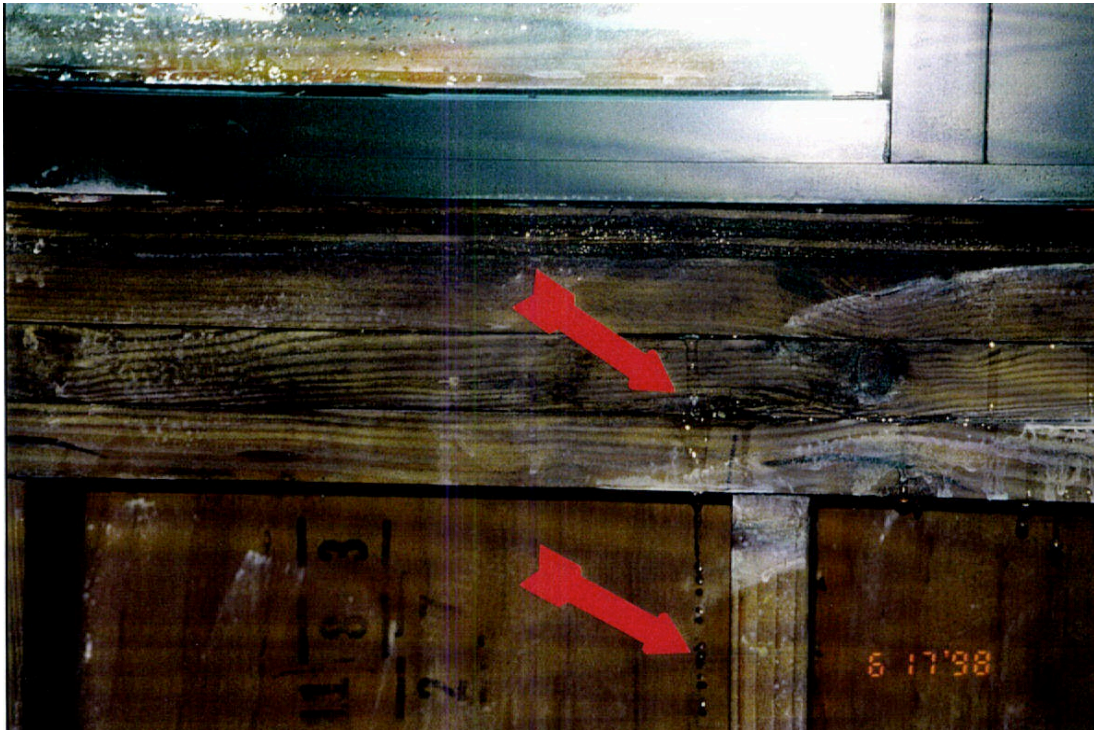


Figure 7: Water leaks observed from the interior sill when water testing the window sill and EIFS sill extrusion joint

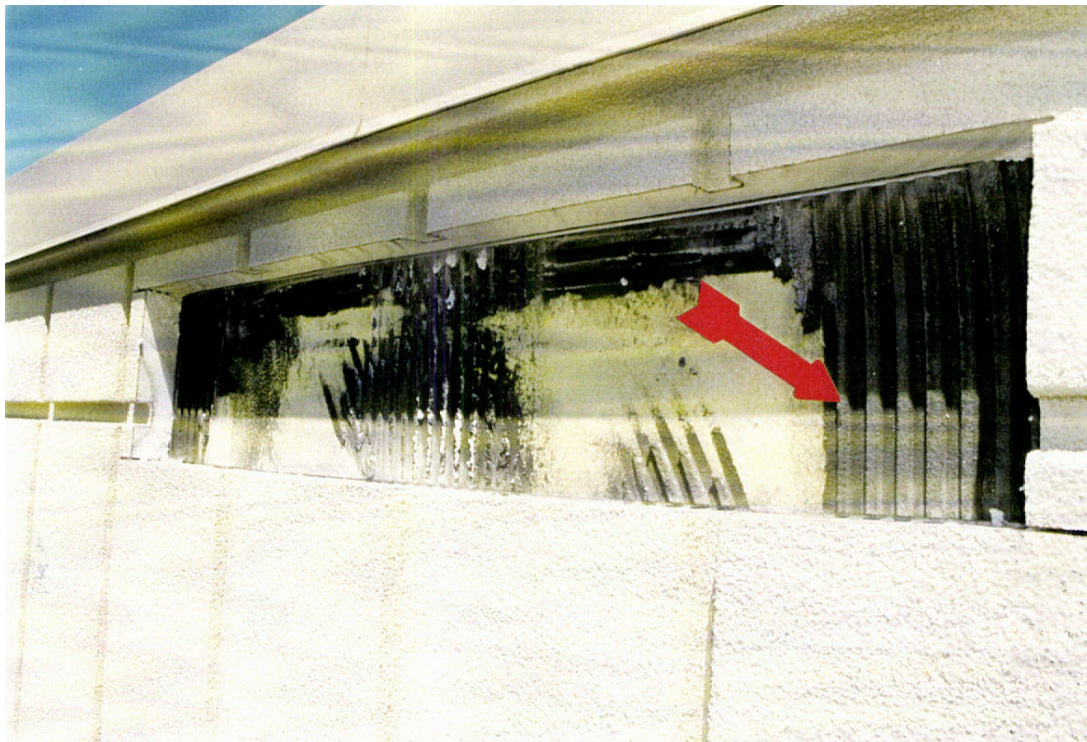


Figure 8: Adhesive material was applied to the substrate instead of the EPS and the EPS was not pushed firmly into the underlying substrate.

Window Performance Testing: What You Need To Know When Selecting Windows

Brandon S. Buchberg¹ and Michael J. Louis²

Introduction

Whether you are selecting windows for a residential house or for a large commercial project, you need to know a few things before you make your selection. Do you know what you are really getting with a window having a particular performance rating? What does the performance rating mean? What procedures and practices can a designer, or owner, implement to reduce their risk? We will attempt to answer these and other concerns related to selecting windows regarding water penetration, with limited discussion related to structural performance and air infiltration.

We will not attempt to address selection of windows having an adequate performance rating for a particular area. Selection of a performance rating for windows should be based on typical weather for a region and building code requirements. For the laymen owner, have the window manufacturer explain, convince, and prove to you that a certain performance rating is adequate for the location of your project.

Current industry standards provide methods and guidelines for testing of windows in the laboratory for certification. Industry standards also provide methods and guidelines for testing of windows in the field that reduce testing pressures; however, designers do not have to adhere to these standards and can specify more stringent requirements as part of the project specifications.

Certification Rating and Testing

The two organizations often referred to when testing windows in the United States are the American Architectural Manufacturers Association (AAMA) and ASTM International (ASTM). These two organizations overlap in some areas and their roles can often be confusing. In general, AAMA is the organization that specifies what requirements must be met to certify windows and AAMA will refer to ASTM to specify standard testing methods to measure performance characteristics of windows. Some AAMA standards may also add their own procedures or testing methods or setups in addition to the ASTM standards.

Windows are rated according to AAMA certification for structural performance, resistance to air infiltration, and resistance to water penetration, as well as other things. The certification designations and testing requirements are listed in the publication AAMA 101/WDMA I.S.2/CSA A440-05, "Standard/Specification for Windows, Doors, and Unit Skylights," and this paper will follow the procedures of

AAMA 101. The rating system allows manufacturers to produce windows that will meet needs of owners/designers for a wide variety of weather exposure conditions. From the certification ratings, designers are able to select windows for a project based on project wind loads and service use. Using this system, manufacturers have a responsibility to provide a window that meets their advertised window performance ratings. Selection of windows does not take into account severe conditions such as hurricanes, except where structural performance must be maintained for life safety. Under severe conditions, water penetration and air infiltration resistance for a window may be exceeded and leakage may occur.

Windows are tested in a laboratory setting as part of the certification program. The rating does not necessarily indicate the window's performance in service or under field conditions. In many cases, manufacturers do not have control of how the windows are handled during transport, storage on-site, or during installation. For this reason, some manufacturers will stand behind their product rating for laboratory testing, but not necessarily during field testing. For field testing, manufacturers will refer to AAMA 502, "Voluntary Specification for Field Testing of Windows and Sliding Glass Doors," which allows a 1/3 reduction in the testing pressure. We will further discuss AAMA 502 later.

Every window that is certified under the AAMA standards will have a Window and Door Manufacturers Association designation displayed on a sticker adhered to the windows. The label will have the following construction:

Product Type – Performance Class (Performance Grade) (Maximum Size of Window Tested)

For example a window having a designation of "H – LC40 36 x 78" is a window that is a **H**ung window (Product Type) of **L**ight **C**ommercial performance class having a performance grade (design pressure) of **40** psf when the maximum size of the window tested is **36 x 78**. The performance grade of a window is related to the size of the window. Often when manufacturers test windows, they will list the maximum size of the model window tested that achieves a particular design pressure. In some cases, a particular model window may be certified for more than one design pressure based on the size of the window. In general, the larger the window size of a specific model, the lower the design pressure.

Pressures for testing and design pressures can be related to a wind velocity by using the following equation (ASCE, 2003);

$$P(\text{psf}) = (0.00256)(K_z)(K_{zt})(K_d)(I)(V)^2$$

$$\text{SI: } P(\text{N/m}^2 \text{ or Pa}) = 0.0613 (K_z)(K_{zt})(K_d)(I)(V)^2$$

This equation approximates the pressure with K_z , K_{zt} , K_d , and I equal to unity. These factors should be determined to more accurately calculate the value of “P”.

The performance class and the performance grade relate to the expected performance of the window during laboratory testing. The performance classes are:

Designation	Performance Class
R	Residential
LC	Light Commercial
C	Commercial
HC	Heavy Commercial
AW	Architectural Windows

TABLE 1. Performance Class Designations

The performance class designations are suggestions for usage. The performance grade, or design pressure, is determined using ASTM E330, “Standard Test Method for Structural Performance of Exterior Windows, Doors, Skylights and Curtain Walls by Uniform Static Air Pressure Difference.” For a given design pressure, a window should be able to withstand 150% of that design pressure without glass breakage, or frame and joinery failure while experiencing a permanent deflection “in the mainframe, sash, sash member, leaf, or sill in excess of 0.4% of its span for R and LC class products, 0.3% of its span for C and HC class products, or 0.2% of its span for AW class products,” (AAMA, 2005). For example, a window having a rating of LC-40 should be able to withstand a structural overload using a pressure of 60 psf [2873 Pa] when tested in accordance with ASTM E330, ex. 40 psf [1915 Pa] x 1.50 = 60 psf [2873 Pa]. The uniform structural load is only one criterion that will determine the window’s certification rating. Each performance class has a minimum required design pressure to be classified within that performance class. The window must also meet air infiltration requirements and must meet water penetration requirements based on the design pressure.

For each performance class, a window must meet the following requirements for minimum air leakage when tested in accordance with ASTM E283, “Standard Test Method for Determining Rate of Air Leakage Through Exterior Windows, Curtain Walls, and Doors Under Specified Pressure Differences Across the Specimen”:

Performance Class	Test Pressure (psf) [Pa]	Maximum Allowable Leakage (cfm/sf) [L/(s m ²)]
R (jalousie windows)	1.6 [75]	1.2 [6.0]
R, LC, and C (except jalousie windows)	1.6 [75]	0.3 [0.3]
HC (sliding seal)	1.6 [75]	0.3 [0.3]

HC (compression seal and fixed)	6.2 [300]	0.3 [0.3]
AW (sliding seal)	6.2 [300]	0.3 [0.3]
AW (compression seal and fixed)	6.2 [300]	0.1 [0.5]

TABLE 2. Air Infiltration Requirements Based On Performance Class

The test pressures of 1.6 psf [75 Pa] and 6.2 psf [300 Pa] are equivalent to a wind velocity of 25.0 mph [40.2 km/h] and 49.2 mph [79.2 km/h], respectively.

When testing for water penetration, the two main factors to determine are the differential test pressure and the procedure for testing. When testing windows, testing is performed in accordance with ASTM E1105, and the test pressure is expressed to be 15% of the design pressure for all performance classes except AW, which is tested at 20% of the design pressure. For example, a window having a rating of LC-40 will be tested at a differential pressure of 6 psf [287 Pa], ex. 40 psf [1915 Pa] x 15% = 6 psf [287 Pa]. This pressure is equivalent to a wind velocity of 48.4 mph [77.9 km/h]. The minimum differential pressure for water penetration testing for windows is 2.9 psf [139 Pa] and no window shall be tested at a pressure less than 2.9 psf [139 Pa], regardless of design class. In addition, no window shall be tested above 12 psf [575 Pa] for U.S. applications and 15 psf [718 Pa] for Canadian applications. Testing pressures above these maximum limits can be specified if a higher pressure is required based on sound engineering calculations or actual site condition wind data; however, this requirement would be above and beyond industry standards.

Two procedures are used for water testing and the performance class will determine which procedure to use. Windows having a performance class of R, LC, and C are to be tested using ASTM E547, “Standard Test Method for Water Penetration of Exterior Windows, Skylights, Doors, and Curtain Walls by Cyclic Static Air Pressure Difference,” which is a cyclic testing procedure with four 5 min. cycles at differential air pressure with 1 min. of zero differential air pressure between each cycle. Windows having an HC or AW performance class rating are to be tested cyclically in accordance with ASTM E547 and tested in accordance with ASTM E331, “Standard Test Method for Water Penetration of Exterior Windows, Skylights, Doors and Curtain Walls by Uniform Static Air Pressure Difference,” which is a static test where the window is tested for 15 min. with a constant differential air pressure.

Designers should know that certification testing for window ratings often does not include testing with receptor frames or other external attachment hardware. Any hardware included by the manufacturer on the units in-service should be included on the window during certification testing.

Regarding windows, AAMA 101 applies to single window units. Most manufacturers do not test assemblies of grouped windows or mulled windows. AAMA 450, “Voluntary Performance Rating Method for Mulled Fenestration Assemblies,” is the applicable testing standard to test structural performance, air

infiltration, and water penetration resistance for muller windows. The testing protocol for the muller assemblies is very similar to those included in AAMA 101; however, if not specified, manufacturers and contractors will overlook this standard as it is not as well known.

Field Testing

In some cases, field testing of windows may be required as a quality assurance measure, or where the window's installed performance may be questioned. A standard that addresses field testing of single window units is AAMA 502, which provides the procedure for field testing windows regarding air and water penetration resistance.

Regarding testing procedure, windows having a performance class of R, LC, and C are to be tested in accordance with ASTM E1105, "Standard Test Method for Field Determination of Water Penetration of Installed Exterior Windows, Skylights, Doors, and Curtain Walls, by Uniform or Cyclic Static Air Pressure Difference," using a cyclically applied differential air pressure for not less than 23 min. Windows having an AW rating should be tested using a static differential air pressure for a duration of 15 min. and windows having a HC rating shall be tested using both procedures.

In 2002, AAMA 502 was issued with a modification to the field testing pressure. Window manufacturers claimed that field conditions often resulted in harsher testing conditions than the controlled setting of the laboratory. Because of this, the 2002 version of AAMA 502 includes a 1/3 reduction allowance for the water penetration test pressure, ex. 40 psf [1915 Pa] x 1.5 x 2/3 = 4 psf [192 Pa]. The adoption of this standard and procedure by AAMA allows new window installations to perform at a substantial reduction compared to the laboratory tested windows unless otherwise stated in the specifications.

In addition to differential air pressure testing in the field, it is a good idea to test the windows using zero air pressure differential in accordance with ASTM E1105. When testing with differential air pressure, gaskets and weather seals can close up creating a tighter seal to exclude water penetration and reduce air infiltration, especially in outward opening operable vents. As a result, testing under differential air pressure only, may lead to a false sense of security and give a skewed view of window performance.

Field testing should be performed with receptor frames, weep covers, and other hardware that are used in the final installation. If the field installation is different than the windows used during certification, enough windows should be field tested to obtain confidence in the performance of the installed window assembly.

Many manufacturers will specify a spray foam seal be installed between the window frame and the structural framing around the windows. The foam acts as an air seal to

prevent air infiltration around the perimeter of the window. During field testing, foam seals may effectively prevent water from leaking into the interior where it may be identified as leakage. Foam is likely to help a window pass a 15 min. or 23 min. water test, but in the long-term, foam will eventually absorb sufficient moisture to cause leakage within the wall and wetting of framing members and interior finishes. As such, the foam should not be installed prior to field testing of installed windows.

Leakage

The definition of leakage included in industry standards and as identified by window manufacturers is different than most owners' expectations. It is the job of the designer to specify a product that will meet the owner's expectations. The AAMA definition as stated in AAMA 502 (AAMA, 2002) is,

“Water penetration shall be defined as penetration of water beyond a plane parallel to the glazing (vertical plane) intersecting the innermost projection of the test specimen, not including interior trim and hardware under the specified conditions of air pressure difference across the specimen. Any such water penetration testing shall constitute failure of the water penetration resistance test. It shall also constitute failure if water penetrates through the perimeter frame of the test specimen. Water contained within drained flashing, gutters, and sills shall not be considered failure.”

Based on this definition, windows may allow water to collect on the interior surface of the window components (PHOTO 1) and not be considered a leak. Undrained troughs (PHOTO 2) can collect water that bypasses weather seals and prevent water from entering to the interior during certification testing. However, because the water does not have a means to drain from the trough or gutter, this is considered a leak. Where the trough or gutter is not drained or not well drained, the water may eventually degrade sealant at framing joints. In addition, where sealant is not properly tooled and sealed to the various framing components by the manufacturer, water will collect in the trough and leak directly to the interior or into the wall framing.

When installed in your building, the average home or building owner will undoubtedly consider water that collects on meeting rails and in undrained troughs or gutters as significant leakage. We should also point out that certification testing occurs for durations of 15 minutes and/or 23 minutes. In prolonged rain events that last much longer than 23 minutes, water can collect well beyond levels identified during certification testing, especially where water on interior window components or in troughs or gutters are not able to drain to the exterior. In these cases, water will leak to the interior by overflowing the trough or by flowing to the interior from window components.

Windows sealed with excessive applications of sealant (PHOTO 3) are very reliant on workmanship of the sealant and have reduced service lives because of their reliance on these sealants. In addition, these seals can be aesthetically displeasing.

Testing by window manufacturers also may not address any design flaws at the perimeter of the window. These perimeter flaws include:

- Discontinuities in the nailing flange – Some windows have nailing flanges that are attached to the window frame members and these flanges extend the width of the jamb, head, or sill section, but do not connect to each other. As a result, there are discontinuities in the nailing flanges at the corners. When the installer flashes the window as directed by the manufacturer, a pinhole can form where membrane strip flashing at the jambs intersects membrane flashing at the sill. Water can flow through such pinholes and onto the sill framing supporting the window. Manufacturers usually do not warn against such pinholes or specifically address this situation.
- Window Joint Durability – Window frame joinery is typically accomplished using a combination of mechanical fasteners, glues and sealants. Frame designs that allow water to pond against such glue and sealant joints allow the long-term durability of the window to be compromised, since water will soften and degrade such corner seals over time. Since manufacturers are not required to perform any form of durability testing, and certification or field testing only occurs during the early life of the window, manufacturers theoretically can design a window to meet 2/3 of the rated design pressure of the window and then apply sealant to achieve the full design pressure. As windows begin to age, their performance begins to diminish, largely due to gasket and weatherstripping deterioration, but also due to sealant and adhesive degradation. Not necessarily the expectation of the end user, such degradation begins immediately for some windows and may last longer with window designs that provide a greater degree of protection to these critical seals.
- Weather gaskets – Many windows rely on weatherstripping or bulb gaskets to seal joints between moving window parts or between each other. These weather gaskets can become disfigured during shipping, dislodged during installation, cut short during fabrication or damaged during operation. If these gaskets do not make a continuous seal to window components, water and air can enter to the interior through voids along the weather gasket seal.

Manufacturers focus on the performance of the window components and the field of the window and any flaw in the window assembly at the perimeter may not necessarily be reported during certification testing. After the window is installed, however, leakage from the perimeter installation may occur into the wall system, but it may go unnoticed until significant damage is incurred. Some window

manufacturers will not take responsibility for such design flaws and often cite perimeter problems as being the responsibility of the installation contractor or the design architect (if one is even associated with the project, since not all residential projects will even have an architect associated with it).

Warranties

Often, end users rely on manufacturer warranties to safeguard against poor performance of window products. What they may not know is that manufacturers also rely on these warranties to limit their liability. When windows are installed and found to be defective or underperforming, the manufacturer's warranty typically limits the manufacturer's responsibility to providing new windows, or at their discretion, repairing defective windows in place. The warranty does not cover labor to remove and install the new windows nor does it include providing any building components such as window trim or cladding that may require replacement as part of the removal and reinstallation process. Prior to providing new windows, the end-user must allow the manufacturer to attempt to repair the windows. This process can take months and effectively delay construction operations. In the end, the only obligation the window manufacturer has is to ship you a new window.

Safeguarding Yourself

As a designer, the first line of defense to obtaining windows that meet the project requirements are the specifications. The specifications are essentially the binding contract between the designer and the contractor. If the specifications contain ambiguous language or testing requirements, contractors (and in turn manufacturers), have the freedom to interpret such ambiguities to their own benefit. In the event the specifications are grossly unclear or ambiguous, the manufacturer will default to the industry standard which may not meet the project needs or specific design criteria. Obtaining the consultation of a qualified building envelope consultant can be an asset during design, specification, and construction administration. When writing window specifications, explicitly state the following information:

- Design pressure – This is derived from Building Codes and local weather data, but should not be left to the interpretation of the contractor and/or window manufacturer. Keep in mind that Codes are minimum guidelines; some professional judgment may be required to increase performance levels for special conditions, such as coastal applications, and for building on high bluffs or hills above surrounding obstructions.
- Testing method and applicable standard (whether cyclic or static) – Static tests are more severe while cyclic tests are designed to more closely approximate the effects of wind. It is not unheard of to require both types of tests. Also, test windows at zero differential air pressure since most rain events that the windows are likely to be exposed to in service will be at very low pressure differences.

- Testing duration – Although not well known, standard test methods for water penetration resistance allow for longer testing periods than the 15 minute and 23 minute durations outlined in the standards.
- Require field testing of installed windows – Field testing pressures should be the same as those used during certification testing with no reduction in test pressure allowed.
- Do not specify windows that will conflict with the stated design pressure – Do not state a design pressure of 80 psf and then specify a model window that has a design pressure of 40 psf.
- Include your own definition of leakage – End users and designers do not have to accept the industry standard definition of what constitutes leakage. It is accepted practice to write your own definition of leakage as long as it is reasonable. Such language may state, “During testing, water shall not penetrate the inboard plane of the glass, collect on window components, or wet any framing members or interior finishes. Water shall not collect in an undrained gutter or trough or in a manner where water ponds against window sealant joints.”
- Include language requiring the contractor to follow manufacturer details of wall assemblies specific to the project – Manufacturers installation details should provide a water management system to direct water to the exterior should the windows leak. This should include the use of flashing and air barrier tie-ins to the surrounding wall construction. The details should be verified through the shop drawing submittal process. As a minimum, list ASTM E2112, “Standard Practice for Installation of Exterior Windows, Doors and Skylights,” as a reference for window flashing details. ASTM E2112 describes how to flash windows in residential applications. With minor modifications, the guidelines of ASTM E2112 can also be applied to many commercial buildings. Flashing should extend the full width of the window and have a back-dam and end dams. All joints should be watertight and window fasteners should not penetrate the pan flashing.

Submittals will help to identify any problems with the windows while the project is still in the design phase. This process helps to prevent large problems in the field and helps to ensure that a quality system is being provided. As part of the submittal process, require the contractor to submit the following for approval:

- Certification testing results that apply to windows of equal or greater size to the windows on your project.
- If muller window assemblies are included in the project, require certification testing results for muller assemblies.
- Shop drawings of window details and pan flashings.

Quality assurance of the installed product is critical to final evaluation of the installed windows. Much of the performance will depend on the integrity of the manufacturer

and the quality of the window installer. A well engineered design followed by field testing is the best plan for a successful window installation. In addition, require the following quality control measures:

- Install a mock-up of the window installation and associated flashings before beginning general window installation. Compare the mock-up to approved shop drawings.
- Require that the manufacturer be on-site during field testing.

Summary

When selecting windows for a project, be informed about what to specify and what manufacturers have to offer. Writing a clear specification for the contractor, reviewing submittals from the contractor and manufacturer, and performing field tests for quality assurance are the keys to having windows that will perform adequately in-service. If performance requirements are not clear and quality assurance measures not implemented, a situation can arise that can delay construction, conflict with end-user expectations, or cause damage to building finishes or the building structure, possibly leading to costly litigation. These things can be avoided by performing the proper due-diligence during the design and construction phases to ensure the requirements of the project are being met.

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PHOTO 1. Water collecting on the window meeting rail.



PHOTO 2. Water collecting in an undrained trough or gutter.

PHOTO 3. Excessive sealant is applied at the window framing joint (TO BE INCLUDED).

A FRAMEWORK FOR MULTI-HAZARD DESIGN AND RETROFIT OF PASSIVELY DAMPED STRUCTURES

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Multi-hazard consideration is gaining importance within the design and retrofit process of many important structures. The primary benefits for multi-hazard approaches to the structural engineer include: (1) identification of the mitigation alternatives that address either single or multiple hazards, and (2) optimization of these hazard mitigation strategies. However, this recent popularity of a unified multi-hazard approach for design or retrofit of structures could be attributed to the high cost of the required mitigation strategies compared to retrofit of the structure for a single hazard and the limited information on life-cycle cost. Thus, a conceptual framework is required for rational comparison of hazard mitigation strategies for structures experiencing multiple environmental hazards, incorporating life-cycle cost assessment.

This paper presents a computational methodology for the optimal life-cycle cost of seismic and wind-excited structures retrofitted with passive energy dissipation devices such as metallic yielding dampers, viscous fluid dampers, and viscoelastic solid dampers. In addition to designing in agreement with the relevant codes of practice, it is important in design or retrofit of structures to consider that the performance of passive energy dissipation devices for reducing the structural responses depends on the distribution, type, size, and number of dampers in the structure. The proposed computational framework integrates a genetic algorithm (GA) based methodology to address these optimization issues of multi-hazard design within the context of nonlinear steel frame structures. Optimization objectives in this framework include minimizing both the damage to and life-cycle cost of a structure, which is vulnerable to one or two of the main natural hazards: earthquakes and strong wind loads. While considering the essential conflicts in dynamic response demands of the structures to these two natural phenomena, passively damped structural designs evolve toward configurations that satisfy prescribed constraints. Monte Carlo simulation models are generated to evaluate various retrofit strategies with different damage levels possible over the life-cycle. The life-cycle cost associated with each scenario is computed, with a discount factor, to obtain the optimum life-cycle cost for various retrofit strategies. The specifics of the computational framework are illustrated through example case studies to highlight the potential benefits of the proposed computational multi-hazard design approach.

Keywords: Multi-hazard; Optimum life-cycle cost; Genetic Algorithms; Structural optimization.

1. INTRODUCTION

The extensive damage sustained in recent natural hazards has highlighted the need to extend the focus of optimized structural design and retrofit procedures that would reliably improve performance of the structures under several hazard scenarios. Passive energy dissipation (PED) devices have been promoted as an effective solution to protect structures against hazards. In addition to designing in compliance with the relevant codes of practice, it is essential in design or retrofit of structures to consider that the performance of PED devices for reducing the structural responses depends on the distribution, type, size, and number of dampers in the structure. For this optimization problem, many researchers have suggested different control algorithms such as optimal sequential search algorithm, control theory method, single point substitution method, and steepest directions search algorithms, genetic algorithms (GA), and conventional optimization methods. In the context of passive control, Zhang and Soong (1992) and Wu, Ou and Soong (1999) proposed the maximum of the root mean square interstory drifts as the objective function and the sequential search method to determine the optimal configuration of viscoelastic dampers. Furuya et al. (1998) have examined the application of GA for damper distribution for wind-induced vibrations. Agrawal and Yang (1999) investigated three search algorithms and five types of objective functions for determining the optimal placement of PED devices in seismic/wind-excited linear buildings. These procedures are mostly restricted to linear structural and damper response. On the other hand, Singh and Moreschi (2002) and Dargush and Sant (2005) have considered nonlinear behavior for aseismic design using genetic algorithms. However, fewer attempts have been made to solve the optimum placement of PED devices with nonlinear dynamic analysis and to investigate the effects of both wind and earthquake hazards on the optimal placement and optimal type of devices. At the same time, a relatively large number of research studies have been directed towards design of buildings considering damage cost estimation methodologies called Performance-Based Design (PBD). PBD aims at finding well defined quantitative decision measurements for decision variables, in order to respond to the diverse needs of decision makers. This methodology is used by Geographic Information System (GIS) based software, such as HAZUS (Whitman et al., 1997) for estimating economic impacts of hazard loadings over a large region.

In this paper, a first attempt to extend PBD to optimized multi-hazard design or retrofit of structures with PED devices is described. In the framework, the base structure may contain a number of PED including metallic yielding dampers, viscous fluid dampers and/or viscoelastic solid dampers over a range of sizes. The proposed methodology allows decision makers to achieve a specific performance objective such as minimum expected life-cycle cost of a building. With the genetic algorithm based optimization framework, the methodology provides insight into multi-hazard performance of a passively controlled structure, as well as the efficiency of the PED devices and some key factors in order to evaluate costs and benefits of the retrofit strategy. A case study is also presented to evaluate the potential capabilities of the framework for minimizing expected life-cycle cost of a retrofitted structure.

2. PROBABILISTIC HAZARD LOSS ESTIMATION

A precise, yet practical approach to PBD is followed here. This approach, which considers hazards, structural response, resulting damage, and associated costs

using probabilistic analysis, can be formulized as (Krawinkler and Miranda, 2004):

$$\nu(DV) = \iiint |\rho(DV|DM)| \cdot |\rho(DM|EDP)| \cdot |\rho(EDP|IM)| \cdot d\nu(IM) \quad (1)$$

where, *IM* (*Intensity Measure*) is a measure of the level of the hazard, *EDP* (*Engineering Demand Parameter*) is a parameter that describes the structural response (e.g., maximum inter-story drift, absolute floor acceleration), *DM* (*Damage Measure*) is a damage state associated with specific required repair actions, *DV* (*Decision Variable*) is a parameter that governs the design decision, $\nu(DV)$ is mean annual rate of exceeding the decision variable *DV*, $\nu(IM)$ is mean annual rate of exceeding a hazard intensity (level) *IM*, $\rho(x) = [1 - P(x)]$ is probability of exceedance, where in turn *P* is the cumulative distribution function of *x*, and $\rho(X|Y) = P(X \geq x|Y = y)$ is conditional probability of exceedance. Eq. 1 has widely been accepted as a generic foundation for performance based design which deconstructs analysis into four subtasks: (1) hazard assessment; (2) structural fragility; (3) evaluation of structural loss; (4) expected life-cycle cost formulation. These subtasks are coupled via integration over all levels of intermediate variables *IM*, *EDP*, and *DM*. The formula depends on probabilistic analysis with the total probability theorem. The detailed approach, in which Eq. 1 is solved, its limitations, and its potential expansion, have been the subject of many researches. The reader is referred to Cornell and Krawinkler (2000), Krawinkler and Miranda (2004), Moehle and Deierlein (2004) for more information on such developments.

2.1. Hazard assessment

For seismic hazard analysis, representative ground motion data for the structure location are used. These motions include suites of sufficient number of ground motion records representing the hazard intensity (*IM*) of the site at different hazard levels (e.g., particular probabilities of exceedance, such as 2%, 10%, and 20% in 50 years). Note that, engineering judgment and the site hazard curve should be used to select the discrete hazard levels for which the structure will be evaluated. For wind hazard analysis, a spectral approach using the inverse fast Fourier transform is used to generate sets of time histories of the wind velocity component along the approaching flow direction. Mean wind speed (3-sec. gust wind speed) at a given location for different wind hazard levels with probabilities of exceedance of 2%, 10%, and 20% in 50 years, is estimated from the wind dataset by the Gumbel method (Holmes, 2001). Then these mean wind speed values are added to wind fluctuation data that is generated from the well known Davenport spectrum (Davenport, 1961), which has energy contents varying with altitude. The wind load is then calculated from the generated time history of the wind velocity data, which is translated to each floor level of the building with a power law model.

2.2. Structural fragility

Fragility curves are used to investigate the probability of exceeding a specific damage state for a given hazard level to represent the relationship between the probability of structural damage and intensity of each hazard (*IM*). For this study, different performance groups are assigned for the building components according to their performance similarity under hazard loading as: (1) global structural system (2) drift-sensitive nonstructural components, (3) acceleration-sensitive

nonstructural components, (4) building contents. Each of these performance groups are assumed to have a specific single fragility curve. For the global structural system, non-linear transient dynamic analysis is carried out under a number of hazardous loading ensembles that represent the region of interest. For example, the structural damage, in terms of inter-story drift and absolute floor acceleration (*EDP*), is investigated through Monte Carlo simulation of different ground motions in the horizontal direction. The probability of exceeding each damage state at each hazard level (*IM*) is found by fitting a lognormal distribution to the *EDP* at each *IM* from which the probability of exceeding each damage state is calculated and a lognormal cumulative distribution is fitted to the available data (Singhal and Kiremidjian, 1996).

For nonstructural components (NSCs), either sensitive to interstory drift or peak floor acceleration, and building contents are assumed to have a single fragility curve representing each performance group. These performance group specific fragility curves are adapted with interpolation for each defined damage state from those in the HAZUS-MH technical manual (NIBS, 2003). Note that, the fragility curves are conditional, in the sense that the failure sequences, initiating from non-structural component failure and developing into partial structural system damage and/or complete collapse of the building, are not considered.

Fragilities of each performance group consist of a sufficient number of discrete damage states that are introduced to completely describe the range of damage to the global structure, NSCs and contents. In order to estimate economical losses, the expected damage cost associated with each of these damage states are defined as a percentage of the cost of the total structure. These values, along with limits of EDPs to define damage states are assigned according to engineering judgment and rational reasoning; however available data should be used when determining the damage states and damage costs, as they vary depending on the type, age, and condition of the building.

2.3. Evaluation of structural loss

With the chosen hazard type and hazard level, engineering demand parameters, EDPs (i.e., interstory drift ratio and absolute floor acceleration) are computed by nonlinear transient dynamic analysis to evaluate the structural loss associated with the direct damage to the structure. Since nonlinear transient dynamic analysis is computationally expensive, Monte-Carlo simulation is employed to generate several realizations of structural response and damage. This technique can sample correlated EDPs from a few dynamic analyses and therefore, is an efficient procedure to estimate the entire distribution of computed EDPs in order to estimate structural, nonstructural and content damage and associated damage costs (Yang, 2006).

The present study also includes the loss associated with business interruption, which occurs when building damage disrupts commercial activity. For the estimation of business interruption, two critical parameters play a crucial role: (1) the level of income generated by the enterprise, (2) the loss of function time that the facility is not capable of conducting business. In this study, the business interruption model for commercial buildings is adapted from the model utilized in HAZUS-MH technical manual. Additionally another specific business interruption model is proposed for wind hazard. In general, earthquake-induced motions, even when they are more severe than those induced by wind, cause a totally different

human response: First, because wind storms occur much more frequently than earthquakes, and second, because the duration of motion caused by a wind storm is generally longer. Thus, wind-induced motions can usually cause occupant discomfort in low intensities and can endanger structural safety in high intensities, whereas earthquake-induced motions endanger human safety rather than human discomfort. So, it is assumed that people are not able to work during wind storms due to motion sickness symptoms and creaking noise, if the peak floor acceleration of the building exceeds a value in the range of values between $0.4\%g$ to $1.2\%g$, according to values suggested by Simiu and Scanlan (1996). A triangular probability density function with minimum and maximum values of human discomfort thresholds are assigned to represent the business interruption loss probability function of a structure under wind hazard. It should be noted that in this model no consideration is given to the frequency dependence of human perception to acceleration.

2.4. Expected life-cycle cost formulation

Regarding the cost efficient retrofit strategies; this study aims at robust design of passively damped structural systems, while minimizing life-cycle cost of the structure. The expected life-cycle cost of a structure can be defined as economic losses (i.e., monetary-equivalent losses) due to single or multiple-hazards that are expected to occur during the lifetime of a new structure or the remaining life of a retrofitted structure. Here, life-cycle cost is assumed to be the sum of the retrofit cost, C_R and five expected damage cost components: (1) expected global structure damage cost $E[C_S]$ corresponding to the damage defined by interstory drift as EDP , (2) expected drift-sensitive nonstructural component damage cost $E[C_{NSD}]$ corresponding to the damage measured by interstory drift as EDP , (3) expected acceleration-sensitive nonstructural component damage cost $E[C_{NSA}]$ corresponding to the damage defined by peak floor acceleration as EDP , (4) expected building contents damage cost $E[C_C]$ corresponding to the damage measured by peak floor acceleration as EDP , and (5) expected cost associated with the business interruption $E[C_{BI}]$ dependent on the global structure damage level, which is defined by interstory drift as EDP . The loss of human life, business inventory losses, rental income losses, and relocation expenses, which in general are also associated with the life-cycle cost of a structure, are not accounted for in the present study. Thus, the expected value of the cost $E[C]$ can then be defined as follows:

$$E[C] = C_R + E[C_S] + E[C_{NSD}] + E[C_{NSA}] + E[C_C] + E[C_{BI}] \quad (2)$$

Expected damage costs are computed using Eq. 1, which assumes that all four variables (IM , EDP , DM and DV) are continuous random variables. But, in this study, economic losses associated to each damage cost component defined in Eq. 2 are represented with discrete damage states and repair costs. Thus, Eq. 1 can be rewritten for discrete damage states for each damage cost component as

$$P\{DV_j > dv\} = \sum_{i=1}^m \int_0^\infty \int_0^\infty p(DV_j | DM) P(DM = dm_i | EDP_j = edp) | dp(EDP_j | IM) | dv(IM) \quad (3)$$

where m is the number of discrete damage states in the j^{th} each damage cost component, $p(DV_j | DM) = P\{DV_j > dv | DM = dm_i\}$ is the probability of exceedance of a decision variable DV in the j^{th} damage cost component

conditioned on knowing that the associated performance group is in the i^{th} damage state, $P(DM = dm_i | EDP_j = edp)$ is the probability that the j^{th} damage cost component will be in damage state i given that the associated performance group has been subjected to and EDP equal to edp , $p(EDP_j | IM) = P(EDP_j > edp | IM = im)$ is the probability that the EDP affecting the j^{th} damage cost component will exceed a certain value of edp given that the hazard level IM is equal to im , and $d\nu(IM) = |d\nu(IM)/dIM| dIM$ is the change in mean annual rate of exceeding a hazard level intensity IM in the range of dIM .

The expected annual cost $E[C_j]$ of the j^{th} damage cost component is computed replacing $p(DV_j | DM)$ in Eq. 3 by the expected value of the cost in the j^{th} component given that it is in the i^{th} damage state, $E[C_j | DM = dm_i]$. Thus,

$$E[C_j] = \sum_{i=1}^m \int_0^{\infty} \int_0^{\infty} E[C_j | DM = dm_i] P(DM = dm_i | EDP_j = edp) dp(EDP_j | IM) | d\nu(IM) \quad (4)$$

where C_j represents the costs associated with the j^{th} damage cost component. From the properties of expectation, the expected annual cost for the building can be computed as the sum of the expected costs of each damage cost component for the annual expected cost estimation as

$$E[C_{ANNUAL}] = \sum_{j=1}^5 E[C_j] = E[C_S] + E[C_{NSD}] + E[C_{NSA}] + E[C_C] + E[C_{BI}] \quad (5)$$

Then, the present value of expected life-cycle cost of building, $E[C_{LIFE}]$ is calculated from

$$E[C_{LIFE}] = C_R + \frac{(1 - e^{-\lambda t})}{\lambda} E[C_{ANNUAL}] \quad (6)$$

where $e^{-\lambda t}$ is the discounted factor, λ is the discount rate per year for the expected life-cycle loss of the building over the lifetime or remaining time of the building t (Rosenblueth, 1976). Although the discount rate is prone to variation, it is usually chosen in the range of 3% to 6% according to many studies.

3. STRUCTURAL MODELING

The framework uses nonlinear transient dynamic analysis with an explicit state-space approach to determine the response of the structure over a range of multiple-hazard load intensities during its lifetime. A uniaxial version of a two-surface cyclic plasticity model (Tseng and Lee 1983, Banerjee et al. 1987, Chopra and Dargush 1994) is implemented for the primary structure, which is simplified by lumped parameter models. The primary structure may contain a number of metallic yielding dampers, viscoelastic solid dampers and/or viscous fluid dampers. For metallic yielding dampers, the same two-surface cyclic plasticity model has been applied. A coupled thermo-viscoelastic Maxwell model and strictly linear Newtonian model are used for viscoelastic and viscous fluid dampers, respectively (Dargush and Soong, 1995; Constantinou et al., 1998).

4. OPTIMIZATION BY EVOLUTIONARY ALGORITHMS

This study develops an automated performance based design algorithm that can identify the optimized design or retrofit of structures under seismic or wind excitations, using genetic algorithms. Genetic algorithms, which were originally

developed by Holland (1975), are based on the evolutionary theory of natural selection and genetics of all species, commonly referred to as “survival of the fittest.” The GA starts with a set of initial structural designs that are generated from a random combination of different damper sizes and/or types. Within each generation, each structure is subjected to a specified number of seismic and/or wind load conditions and evaluated to determine the degree to which the objective is satisfied and to what extent design constraints are violated. The fitness values, along with genetic operators modeling selection, crossover, and mutation processes, define the framework of the next generation of structures.

5. CASE STUDY

As a case study, a 16-story, 58.4m high, uniform steel frame building, situated on firm ground and open terrain in Los Angeles, CA with a lifetime of 50 years, is evaluated to illustrate the application of the proposed optimization framework. The structure is classified as a professional business service building (COM4 in HAZUS-MH) with a total floor area of 6020m². The total cost of the baseline building without regard to its location, is estimated to be approximately \$740 per square meter. The baseline lumped-mass steel frame model assumed to be initially undamped and has story weight of 2500kN and lateral story stiffness of 100kN/mm, resulting in the first two periods as 3.3sec and 1.1sec, respectively. The yield forces on the inner loading surface and outer loading surface needed for previously mentioned two-surface cyclic plasticity structural model for each story of the building are set to 1100kN and 4300kN, respectively. It is assumed that 12% of the total cost of the structure is associated with structural components, 35% of the cost is associated with drift-sensitive NSCs, another 35% of the cost is associated with acceleration-sensitive NSCs, and the remaining 18% of the cost is associated with the building contents. Dampers are available in four different discrete sizes and their costs somewhat unrealistically assumed to vary from 3\$/m² to 20\$/m² depending on size. For business interruption cost estimations, proprietor’s income from the building is assumed to be 9.9\$/m²/day (HAZUS-MH). The maximum interstory drift ratio and peak floor acceleration are chosen as the engineering demand parameters (*EDP*). The seismic environment is described by three different hazard level ground motion records from the MCEER Northridge Earthquake ensemble (Filiatrault and Wanitkorkul, 2005) with probabilities of exceedance of 2%, 10%, and 20% in 50 years, respectively. Each seismic hazard level includes three horizontal ground motion records. For the wind environment, extreme wind speed data from 2002 to 2006, a total of five years of yearly maximum wind speeds collected by the National Climatic Data Center from the National Weather Service (NWS) station at the Los Angeles Downtown UCS Campus is used to estimate extreme wind speeds for Los Angeles. Accordingly, mean gust speeds are predicted by the Gumbel method as 28m/s, 32 m/s, and 35m/s for three different wind hazard levels with probabilities of exceedance of 2%, 10%, and 20% in 50 years, respectively. Then, the inverse fast Fourier transform is used to generate three sets of time histories of the along-wind velocity components for each wind hazard level. Although wind storm duration is assumed to be 2 hours, duration of 45 seconds has been used to reduce the computational time, and such duration should be sufficient to establish the stationary response properties. Parameters for damage state definitions and associated damage costs for interstory drift and acceleration thresholds used here are shown in Table 1 and Table 2, respectively (FEMA 273; FEMA 227; Lagaros

et al., 2006). It should be noted that no collapse cases are included for this study. Fragility curve parameters for NSCs and contents for the seven damage states of Table 1 and Table 2 is developed for interpolated lognormal distribution values (i.e. median and standard deviation) obtained from four damage states of HAZUS-MH. Monte-Carlo simulation is applied to the initial database of the *EDP* obtained from nonlinear transient dynamic analysis to generate 400 realizations of the seismic and/or wind loading response of the retrofitted structure. The damage cost information is analyzed to determine the expected life-cycle cost of each retrofit alternative with 5% discount rate. The primary constraints are considered to be interstory drift and absolute floor acceleration limits to keep the structural damage at a certain level. Because of the important differences between dynamic properties of seismic and wind excitations (e.g., durations, predominant frequencies), different values of constraints are proposed for seismic and wind excitations. For seismic hazard design of the building, the interstory drift is limited to 2% of the height of the story of the buildings, whereas the absolute floor acceleration is only limited to 0.8 g. On the other hand, under severe wind environments, the main purpose of installing PED devices is to reduce the discomfort of the occupants and damage to sensitive equipment and nonstructural components in buildings. Thus, for the performance of the building deformation envelope to be adequate for serviceability requirements, the interstory drift is set not to exceed 1/300 of the story height (i.e., interstory drift ratio of 0.33%) under wind-induced vibration. With these constraints, for the search of the optimal retrofit alternative, the fitness function seeks the maximum reduction in the expected life-cycle cost with the proposed distribution of passive energy dissipation devices. For this example, the normalized fitness value of each design to be maximized is calculated as follows:

$$f = \frac{E[C_{LIFE}^{UNRETROFITTED}] - E[C_{LIFE}^{RETROFITTED}]}{E[C_{LIFE}^{UNRETROFITTED}]} \quad (7)$$

where, $E[C_{LIFE}^{UNRETROFITTED}]$ and $E[C_{LIFE}^{RETROFITTED}]$ represents the life-cycle cost of baseline structure (uncontrolled structure) and retrofitted structure, respectively. From baseline analysis, $E[C_{LIFE}^{UNRETROFITTED}]$ is found to be \$200000, \$14000, and \$220000, when the structure is assumed to be subjected to only seismic, only wind, both seismic and wind loading during its lifetime, respectively.

Figure 1a shows robust design alternatives of the sixteen-story building when the building is allowed to be designed only with viscoelastic solid dampers along the height of the building for three hazardous environment scenarios: (1) seismic loading environment (the left hand design), (2) wind loading environment (middle design), (3) combined seismic and wind environments (the right hand design). Within each story, the dimension of the central blue symbol illustrates damper size. When compared to the structural design for only seismic environment, smaller and less number of viscoelastic dampers is selected for the design under only wind environment in the middle plot of Figure 1a. For the design under combined loading in the right hand plot of Figure 1a, the fitness value reduced slightly due to an increased size of the viscoelastic dampers in the lower stories compared to the left hand plot. The results of life-cycle cost analysis reported in Figure 1b, 1c, and 1d show the exceedance curves for expected damage loss before and after optimal retrofit design of the structure due to seismic, wind, and combined hazards, respectively. With the optimum retrofit configurations, the life-

cycle damage costs of the structure are reduced by 80%, 96%, and 81% for seismic hazard, wind hazard, and combined hazards, respectively. When the retrofit cost is taken into account, the fitness values (reduction in expected life-cycle cost after retrofit) are found to be 0.55, 0.18, and 0.53 for seismic hazard, wind hazard, and combined hazards, respectively. Thus, with a minor cost increase, the structure could be protected not only against seismic hazard, but also wind hazard.

Table 1 Damage state interstory drift ratio limits and corresponding costs

<i>Damage level</i>	<i>Damage state</i>	<i>Interstory drift ratio, d (%)</i>	<i>Structural damage cost (% of building cost)</i>	<i>Cost of drift sensitive NSC (% of building cost)</i>
1	None	$d < 0.2$	0.00	0.00
2	Slight	$0.2 < d < 0.4$	0.06	0.35
3	Light	$0.4 < d < 0.6$	0.60	1.40
4	Moderate	$0.6 < d < 0.8$	2.40	3.50
5	Extensive	$0.8 < d < 2.5$	5.40	17.50
6	Major	$2.5 < d < 5$	9.60	21.00
7	Destroyed	$5 < d$	12.00	35.00

Table 2 Damage state floor acceleration limits and corresponding costs

<i>Damage level</i>	<i>Damage state</i>	<i>Peak floor acceleration, a (g)</i>	<i>Cost of acceleration sensitive NSC (% of building cost)</i>	<i>Cost of acceleration Sensitive Contents (% of building cost)</i>
1	None	$a < 0.1$	0.00	0.00
2	Slight	$0.1 < a < 0.25$	0.66	0.36
3	Light	$0.25 < a < 0.5$	3.55	1.80
4	Moderate	$0.5 < a < 1$	10.53	9.00
5	Extensive	$1 < a < 1.5$	15.79	10.80
6	Major	$1.5 < a < 2$	26.32	12.60
7	Destroyed	$2 < a$	35.00	18.00

6. CONCLUSIONS

The results of case study suggest that the integrated optimization framework can improve the performance of the structural system retrofitted with passive energy dissipation devices, while it maintains low expected life-cycle cost due to seismic and/or wind hazard. Optimum values of design variables are successfully determined by a genetic algorithm, which was used in this study as the optimization technique. Thus, it is concluded that the proposed method has the advantages, not only from the viewpoint of performance under different hazard events, but also from an economic perspective. However, this study should be extended considering three important points. First, additional studies considering different structural systems and configurations must be carried out. Second, the cost data used in damage cost model is based on averages over metropolitan areas, and may not provide an accurate estimation of the cost of any specific building. Third, the results of the life-cycle cost study are sensitive to the assumptions made in developing the cost model. It should be noted that the expected life-cycle cost measure reduces the cost and probability information down to a single number and makes it hard for decision makers to distinguish the risk characteristics of alternative designs. Therefore, a life-cycle cost model with a detailed analysis of risk aversion would consider the amount of risk associated with an alternative and the risk attitudes of the decision maker. Furthermore, the decision maker is usually assumed to be more concerned with the rare events that are presented in the right

tails of the exceedance curves of costs of alternative designs. Thus, in future work, care must be exercised in defining appropriate tail distributions and in the selection of cost model parameters (e.g., discount rate, design lifetime, etc.). Additionally, approaches that incorporate uncertainty in calculating probabilities should be integrated into the framework.

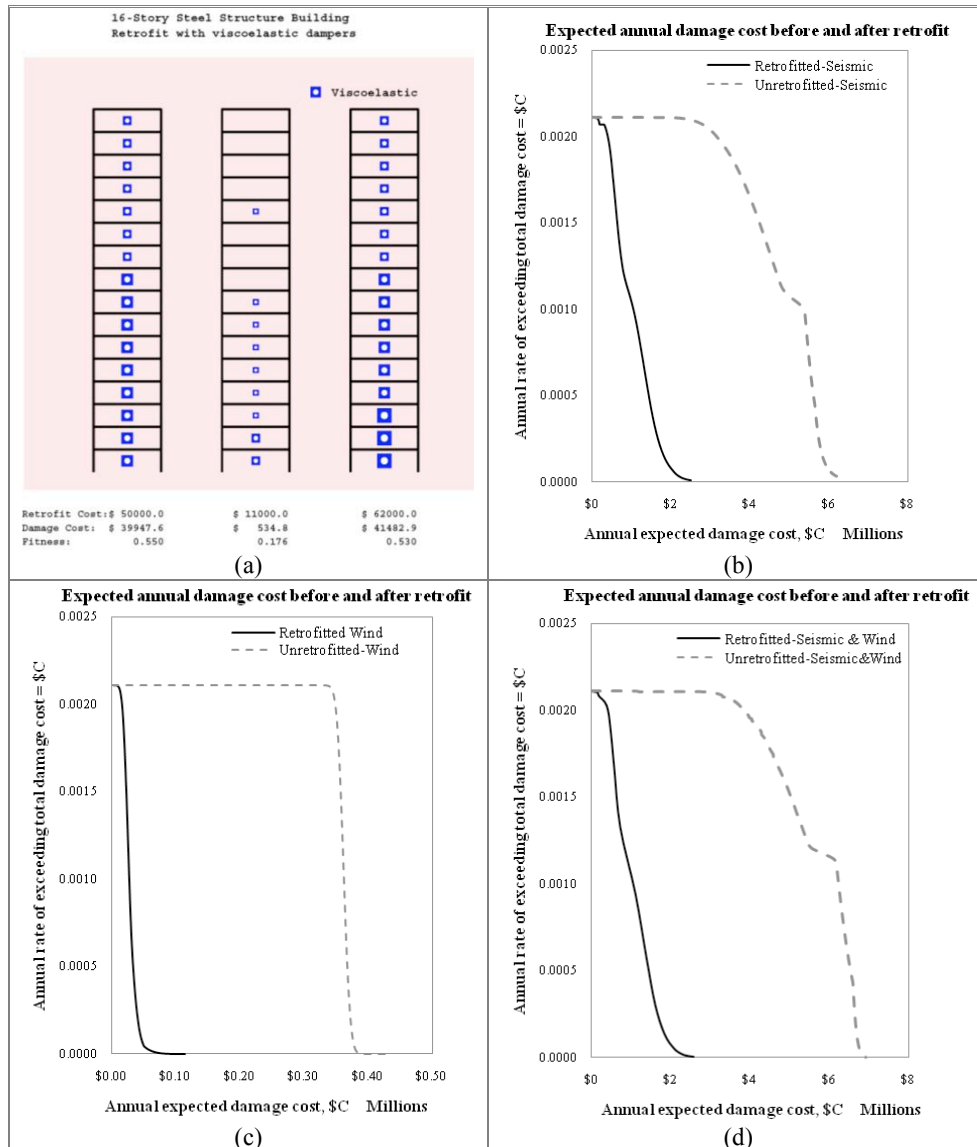


Figure 1 16-story steel frame (a) Robust designs due to different hazards (b) Expected annual damage cost for seismic hazard; (c) Expected annual damage cost for wind hazard; (d) Expected annual damage cost for combined hazards.

ACKNOWLEDGEMENTS

This work is supported mainly by the Earthquake Engineering Research Centers Program of the U.S. National Science Foundation under NSF Award Number ECC – 9701471. This financial support is appreciatively acknowledged.

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Fire Performance and Environmental Impact Evaluations of Communication Cable Installations with Unexpected Infrastructure Fire-Safety Implications

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Abstract

Changing environmental interests in the wire and cable industry (WCI) have generated studies into the eco-performance of various data and communication cabling system alternatives for computers and local area networks (LAN). To accurately assess the various alternatives, the installed electronic functionality and fire safety performance requirements of the various end-use product options must be equivalent. In buildings, high fire performance (HFP) based polyvinyl chloride (PVC) sheathed and perfluoropolymer (PFP) insulated (CMP or PVC/PFP) cables can be routed directly in concealed plenum spaces without the use of additional metal conduit fire protection. Products that do not meet specific high fire-performance criteria, such as low-smoke zero-halogen (LSZH/PE) cables or riser-rated cables (CMR or PVC/PE), often must be protected inside other structures, typically metal EMT conduit or trunking. Life cycle analysis (LCA) via ISO 14040 methodology is used to evaluate cabling alternatives using an appropriate functional unit. Copper wire is identified as a significant contributor to all impact categories, and the major contributor for human toxicity. The burdens for steel conduits dominate all other impact categories studied for systems requiring steel for fire performance code compliance. The additional fire safety and installation advantages provided by the CMP cables offset the environmental burdens associated with the initial manufacture of just the cable. Thus, CMP cables provide superior overall fire safety, environmental performance, and economic benefits versus LSZH/PE and CMR cables installed in metal conduits for U.S. plenum cable installations. During the writing of this paper, new fire safety research became available indicating that EMT die cast conduit-couplings can begin to weaken, melt and fail to fire-protect in less than 4 minutes at or less than 400°C (images to be shown). This finding is not consistent with previous beliefs that the fire performance of CMR or LSZH cable in EMT conduit was equal to the performance of CMP cable.

Keywords: Communication Cables; LAN Cabling; Life Cycle Analysis; Steel Conduit; Conduit Couplings; Environmental Burden; CMR; CMP; EMT; ISO; LCA; LSZH; PFP; PVC; HDPE; High Fire Performance Cabling (HFP-Cables); Plenum.

1. Introduction

The objective of this study is to quantify the entire environmental performance for a LAN cabling system and identify the actual key contributors by including both the copper wire and the conduit,

wireway, and support steel required in a typical plenum cable installation. The resulting study allows comparison across cable types for the specific application of plenum cable installations. This study will also show how the focus changes as a more comprehensive functional unit is defined, progressing from cables without copper (not a functional system), to uninstalled cables (not functionally equivalent due to fire performance), and finally, to installed cables in the plenum space (functionally equivalent).

A recent Design for the Environment (DfE) study on wire and cable has been conducted with the objective of providing the industry with environmental performance data on cabling alternatives to use in conjunction with economic and technical performance data when selecting cables for a given application [17]. The study focused on material selection options within cable types. Although the DfE study specifically notes that comparisons across cable types, such as CMR and CMP cabling, are not appropriate due to differences in functionality, the effect of these functional differences on environmental performance are not quantified. Furthermore, while the conductor is an integral part of a functioning communications cable, the DfE study omits the burdens associated with the copper wire under the assumption that copper use is equivalent across the cabling alternatives studied. As shown in previous research by Williams, et. al., and as will be supported by this study, the copper wire contributes significantly to the environmental burden associated with cable construction [19].

Fundamental to this study is the assumption of functional equivalence across both electrical and fire performance. The environmental life cycle analysis assumes that equivalent copper use addresses electrical performance equivalences and fire-code compliance equates fire performance equivalence. However, recent IEC research tests suggests fire performance may still not be equivalent. The results of these fire tests are discussed.

2. LCA Methodology

The goal of this life cycle analysis is to compare the environmental burdens of a CMP LAN cable installation to both CMR and LSZH/PE LAN cable installations using equivalent industry accepted codes and standards including both electrical and fire safety performance.

The life cycle analysis from which excerpts contained herein come is titled *New Fire Hazard and Environmental Burden Evaluations of Electrical Cable Installations Utilizing ISO 14040 Environmental Methodologies* [16]. This paper was presented at the 56th International Wire and cable symposium along with some supplemental data which has been incorporated into this paper. A

peer-reviewed version of this paper with additional depth of study and sensitivity analysis was still in development as of March 2008.

The evaluation is specific to installations in the United States with respect to fire code performance. Where available, life cycle data for the materials in the U.S. plenum cable installation supply chain will be used. Data from specific DuPont plant sites and suppliers are used for the PFP installation as opposed to representing industry wide PFP environmental performance.

The system boundary includes raw material extraction from the ground through installation of the cabling systems to end-of-life as depicted in Figure 1. The use-phase, the time between cable installation and removal, is deemed both equivalent and insignificant with regards to environmental burden for these low-voltage cable installations. Cable installations are typically replaced due to technological obsolescence as opposed to cable failure, so no difference in life expectancy is projected. Therefore, the use-phase is excluded from this study.

The analysis quantifies raw material use rates, energy requirements, and emissions throughout the supply chain, incorporating efficiency, distribution, and conversion losses associated with electricity generation. Allocations for co-products along the supply chain are often required and are addressed individually, via system expansion, avoidance, mass allocation, or energy allocation based upon their own merits. When possible, actual plant data is used. Otherwise literature sources consistent with the scope of this study and LCA database modules from Ecoinvent™ or others available in SimaPro™ life cycle software are used. When multiple sources are available for significant materials in the life cycle, models are critically evaluated to generate the most comprehensive representation of current manufacturing processes.

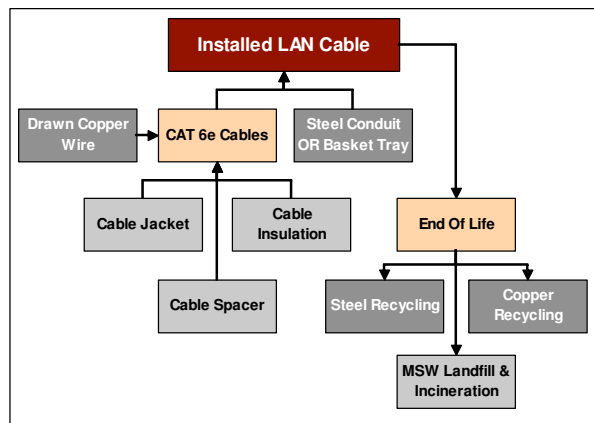


Figure 1. Cable Installation Life Cycle System Boundaries

All materials identified in any of the main cable parts, PFP insulation and spacers, PVC jacketing, copper wire, LSZH jacketing, high-density polyethylene (HDPE) insulation and spacers, galvanized steel support and conduit have been included. When literature or plant data is not available, stoichiometric derivations or similar approximations were used and detailed in this report. When data quality is poor or highly variable, sensitivity analysis is used to confirm overall accuracy. Modeling efforts for materials representing more than 1% of the mass of any component, and for materials identified as a likely contributor to

one of the impact assessments evaluated, are emphasized. This study follows the guide that inclusion at some conservative level of detail is preferable to omission.

End-of-Life (EOL) assessments account for the recycling of the copper, steel, and zinc (from galvanized steel) used in the installations. The sensitivity of recycling assumptions are evaluated and the general disposal of remaining materials is via municipal solid waste landfill or incineration. Details on the end-of-life modeling assumptions are included in Appendix A of the complete LCA report.

Impact assessment methods available in the SimaPro® life cycle software are used for evaluation of the life cycle inventories. The Cumulative Energy Demand impact assessment from Ecoinvent™ 2000 is used to calculate the primary energy demand [8]. Both non-renewable fossil and nuclear fuels are included on a lower heating value basis. Other environmental impacts are calculated via the CML 2 baseline 2000 V2.1 impact assessment available in SimaPro™ [7,8]. Updates for the global warming potential values were made to agree with IPCC TAR 2001 [18].

3. LCA Basis

3.1 Functional Unit

Per ISO 14040, product systems are to be evaluated on a functionally equivalent basis to ensure fairness in the environmental comparison. Both data transmission and fire performance must be equivalent to make meaningful life cycle comparisons. For the comparison of CMP cables with CMR and LSZH/PE cables, the additional burdens of the installation steel and conduit must be evaluated along with the burdens for the cable (with copper conductor) to meet current fire code requirements.

The study evaluates four cable installations using the terminology detailed in Table 1.

Table 1. Installation Case Basis

Installation	Jacket Material	Insulation Material	Spacer Material	Steel Installation
CMR	PVC	HDPE	HDPE	CMR Install
LSZH	LSZH	HDPE	HDPE	LSZH/PE Install
CMP	PVC	PFP	Recycled PFP	CMP Install
CMP(F)	PVC	PFP (Foamed)	Recycled PFP	CMP Install

For this study, a complete LAN cable installation for a typical 27,000 ft² office building is used, including two CAT 6e cables for data communications and telecommunications per work station, wireways, conduit, supports, power poles, etc. utilizing TIA 568A and 569A design guidelines and industry best practices by Innovative Engineering Services (IES) and Masselli Cabling Design [12]. Table 2 identifies the bill of materials developed for both a CMR or LSZH/PE installation and a CMP installation.

The general steel support layout assumes two main headers through the building using either square duct wireways (CMR & LSZH/PE installs) or basket tray (CMP Installs) at 50% fill ratios. Branches from the header use either galvanized electrical metallic tubing (EMT) at 40% fill ratios (CMR & LSZH/PE) or j-hooks (CMP)

based on fire-performance requirements. The categories for ‘Steel’ in the different cabling installations shown in the results section of this paper refer to the steel itemized in Table 2 for each application. Credits associated with steel recycle plus burdens of transportation throughout the life cycle are incorporated in the ‘Steel’ values. A life cycle process for each item at each size in Table 2 was developed individually in the life cycle model to account for the proper thickness and mass of zinc for galvanizing when required. Of note, an alternative support system for CMP cable would replace basket tray with j-hooks for support in the main headers. While this option is not evaluated in this study, it would result in a substantial reduction in steel use for the CMP system.

Table 2. Steel Requirements for CAT 6 LAN Cabling Installations in a Typical 27,000 ft2 Office Building

CMR & LSZH Installation Steel	Size	Qty	Mass (kg)
Wireway & Fittings	6" & 8"	365'	1218.1
Conduit & Fittings	2.5", 2", 1"	890'	986.0
Power Poles & Fittings		110	252.5
Threaded Rod	1/4"	640'	34.8
Brackets & mounting nuts	N/A	320	202.8
Mass of Steel			2694.2
Cabling Requirements	Size	Qty	Mass (kg)
CAT 6 CMR or LSZH/PE Cables	22.5 AWG	21.95 km (72,000 ft)	921.2
CMP Installation Steel	Size	Qty	Mass (kg)
Basket Tray & Fittings	18" x 2"	420'	440.7
Threaded Rod	1/4"	425'	23.1
Beam Clamps		250	28.3
"J" Hooks	CAT 21	160	11.2
Power Poles		110	244.5
Mass of Steel			747.8
Cabling Requirements	Size	Qty	Mass (kg)
CAT 6 CMP Cables PVC / PFP (Solid)	22.5 AWG	21.95 km (72,000 ft)	946.6
CAT 6 CMP Cables PVC / PFP (Foamed)	22.5 AWG	21.95 km (72,000 ft)	886.2

A CAT 6 cable consists of a copper conductor, insulation, jacket, and spacer. Results in this study will be broken down into these four categories, each including the wire drawing, extrusion, compounding, foaming, and/or transportation required to process the material into an installed cable. The burdens identified for ‘Copper’ incorporate the credits given through the recycling of the copper wire at the end-of-life. A separate term for municipal solid waste, “MSW”, is used to account for all incineration and landfill burdens from all materials, including steel. The burdens from the "MSW" category are typically minimal compared to those from the overall installation.

A CMP cable uses PFP insulation without any PFP recycling. For CAT 6e cables, current practice does not incorporate recycled PFP. The jacket for a CMP cable is modeled as lead-free PVC polymer compounded with a combination of flame retardants, plasticizers, stabilizers, and fillers depending on the manufacturer. For simplicity, a formula publicly available from Kemgard® and

specifically for plenum grade cable is used as shown in Table 3 [15]. PFP cable spacer is modeled entirely as recycled PFP [14]. Electrical performance is not one of the criteria for the spacer, but fire performance is still required.

Table 3. PVC and LSZH Jacket Formulations

Material	Plenum-Grade PVC Jacket	LSZH Jacket
	wt%	wt%
Polyvinyl Chloride (PVC)	43.5%	
Ethylvinyl Acetate (EVA)		60%
Aluminum Trihydrate (ATH)	30.4%	30%
Phthalate Plasticizer	8.7%	7%
Ca/Zn Stabilizer	3.0%	3%
Kemgard HPSS	4.4%	
Brominated Flame Retardant	8.7%	
Antimony Oxide	1.3%	

LSZH formulation is from proprietary data and is estimated as a typical formulation. PVC non-lead formulation per http://www.kemgard.com/technical_articles/HPSSbrochureflexPVC.pdf

The relative mass of the main components in the cable also vary considerably with manufacturer and type of CAT 6 cable. A review of cable specification sheets from major cable suppliers suggests most CAT 6e cables use 22 to 23 AWG wire. For this study, 22.5 AWG is assumed for all cable types. Further breakdown is detailed in Table 4.

Table 4. Cable Component Breakdown

Cable Component	PVC/PE	LSZH/PE	PVC / PFP (Solid)	PVC / PFP (Foamed)
	%	%	%	%
Conductor	50%	50%	48%	52%
Jacket	33%	33%	23%	25%
Insulation	15%	15%	21%	16%
Spacer	3%	3%	7%	8%
Mass (kg/km)	42.0	42.0	43.1	40.4

Overall mass is typical of CAT 6 cables per and in agreement with DfE study but with decreased copper use to match 22.5 AWG. LSZH/PE cable assumed equiv. in cable breakdown to CMR cable

LSZH and CMR cables are currently not commonly used for plenum applications due to fire-safety code requirements. This study will assume a typical breakdown by cable component similar to CMR-rated cable for both cable types. While both HDPE and low-density polyethylene (LDPE) are used as insulation material, HDPE is used in this study. Only minor differences are seen between the two from a life cycle impact perspective.

Formulations of PVC jacketing in CMR cabling are quite diverse, typically proprietary, and may include stabilizers, plasticizers, and fillers. Since the CMR cable market for use in conduit for U.S. plenum installations is limited, a formulation similar to CMP cable jackets is assumed for this study based on the expected need for enhanced fire performance. The low-smoke zero-halogen jacket is a compounded polymer, including ethyl-vinyl-acetate polymer, flame retardants, plasticizers, and stabilizers. A typical composition as shown in Table 2 is expected.

No recycle of PVC, LSZH, or PE is modeled as current practice does not support this evaluation, particularly for the compounded jacketing. Less than 0.1% of PVC is currently recycled within the

United States [4]. Recycle of PFP is assumed for the spacer only as detailed in section 3.2.2.

3.2 Life Cycle Models

The life cycle models used in the evaluation of the various cabling materials, including perfluoropolymer (PFP), recycled PFP, HDPE, PVC, Low-smoke zero-halogen (LSZH) jackets, and various plasticizers and flame retardants, and others are all detailed in the paper by Krieger, et.al. [16]. In addition to identifying the data source, the selection process is discussed, including discussions of the data's relevance and/or data quality. Expects of the details for modeling of the steel and copper wire are included here.

3.2.1 Steel Components. Steel is manufactured using two main processes, the basic oxygen furnace (BOF) or the electric arc furnace (EAF). The EAF process is predominantly used in the production of engineering steel and wire rod, as opposed to the cold-formed sections required to produce wireways and conduit [20]. While basket trays (and wire rod) are more likely produced from EAF steel, this study assumes only BOF steel is used as a base case. The amount of recycled steel is significantly different as EAF steel is supplied via 95-100% scrap steel while the BOF market is supplied with 12-30% scrap steel.

Steel recycle is modeled using methodology presented in the International Iron and Steel (IISI) life cycle report [13]. A burden for recycled (scrap) steel is calculated accounting for the production for primary and secondary manufacturing processes, scrap steel yield rates to secondary steel, and the steel recycle rate..

Within SimaPro™, models for chromium steel, high-alloy steel, low-alloyed and un-alloyed steels were available from Ecoinvent™, ETH-Zurich, BUWAL, and Franklin [2,6,9,10,11]. An LCI from the steel industry provided by IISI was purchased and included in the analysis [28]. Details on the selection process and model development are available in a separate study by Barr, S. et.al [3]. The finalized model uses the “Blast Furnace Route for Hot Rolled Coil” model from the IISI LCI as the primary data [20].

From the Ecoinvent™ model for un-alloyed BOF steel, the PAH emissions provided the major impact in the CML 2 2000 human toxicity impact assessment [2]. As the Ecoinvent™ model is thorough with respect to the energy supply chain, the PAH and other emissions omitted in the IISI LCI that contributed more than 1% of the impact from the coal supply chain in each category of the Ecoinvent™ model were added to the IISI LCI data. Ecoinvent™ models are used for the hot-rolling and cold-rolling steps [2].

3.2.2 Copper Wire. The copper wire process is modeled in three sections. Mined copper concentrate at beneficiation is converted to primary copper, which is drawn into wire and eventually recycled as feedstock to secondary copper. Only primary copper has the purity requirements for wire and cable applications.

Literature data for primary copper have substantial variations, particularly with regard to the CML 2 2000 human toxicity impact assessment. Within SimaPro™, three different Ecoinvent™ models for different regions (North America - RNA, Europe – RER, and the global market – GLO) were evaluated [2]. While the RNA model had somewhat higher energy and global warming impacts, the human toxicity impacts were a factor of twenty less than the RER model and four times less than the GLO model. In addition, a model from the ETH-Zurich database was available within

SimaPro™ and included in the evaluation [11]. While similar to the RNA model in energy and GHG emissions, the impacts are higher in other categories except human toxicity. The ETH-Zurich model has the lowest human toxicity impact (roughly one-eighth of the RNA model).

For wire drawing, only one secondary data set was available via Ecoinvent™ [2]. The impacts from this step are roughly one-third of those for primary copper for GHG emissions and energy use, but less than 7% of the toxicity impacts.

A recycle model was developed for each region, with the scrap copper displacing the pure copper content in copper concentrate, at beneficiation (by region) [1]. Since the ETH-Zurich data is a cradle-to-gate evaluation for primary copper, an avoidance flow equivalent to half the burdens for the primary copper model was used. This is consistent with the avoidance of RNA copper concentrate in the RNA primary copper system.

4. LCA Results

On an equivalent functional unit basis, accounting for both electrical-performance and fire-performance per code requirements, CMP and CMP-foamed (CMP(F)) LAN cable installations have lower environmental burdens than LSZH and CMR installations for primary non-renewable energy, human toxicity, air acidification, photo-chemical oxidation, and eutrophication per the Ecoinvent™ Cumulative Energy Demand and CML 2 baseline 2000 impact assessments [7,8]. Greenhouse gas emissions per IPCC are slightly higher for CMP installations, but CMP(F) installations are also lower than LSZH and CMR installations.

4.1 Non-renewable Primary Energy

As shown in the breakdown of energy demand by installation component in Figure 2, non-renewable primary energy requirements are dominated by the use of copper wire and steel. The impact of copper accounts for between 40% and 58% of the cable burden, excluding the steel. Reduction in one gauge size of copper would yield reduced energy use rates equivalent to 14-29% of the entire burden of plastic (i.e. jacket + insulation + spacer). A substantial mass of steel is used in conduit, wireways, power poles, and supports required in the LSZH or CMR cable installations. The energy associated with steel is more than double the energy for the plastics (jacket + insulation + spacer) in LSZH or CMR cables.

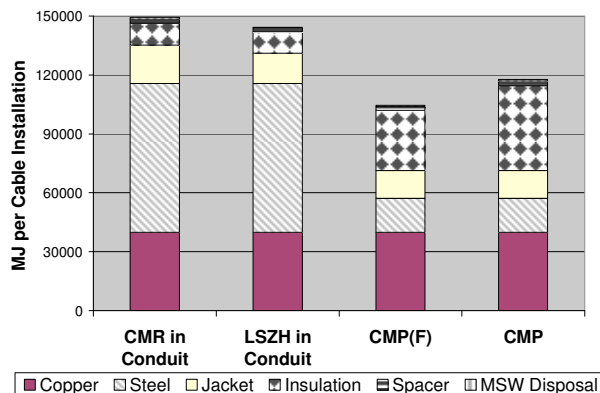


Figure 2. Non-Renewable Primary Energy by Installation Component

The CMP installation is more balanced across the different cable parts, using only one quarter the amount of steel and consuming 18-30% less energy compared to the LSZH and CMR installations, respectively. Although the energy use for the PFP insulation is higher than PE insulation, the fire performance of the PFP avoids the higher steel use and results in a net decrease of over 26,600 MJ per installation or 1210 MJ per installed km of cable (39750 MJ or 1810 MJ/km for CMP(F) installations).

4.2 Greenhouse Gas emissions

Greenhouse gas emissions (GHG) are closer for CMR, CMR(F), LSZH, and CMP installations due to the higher energy use rate for PFP insulation compared to HDPE insulation and the hydro-fluorocarbon emissions in the PFP supply chain as seen in Figure 3. CMP(F) installation is favored due to the differences in steel requirements and reduced PFP use rate for foamed insulation.

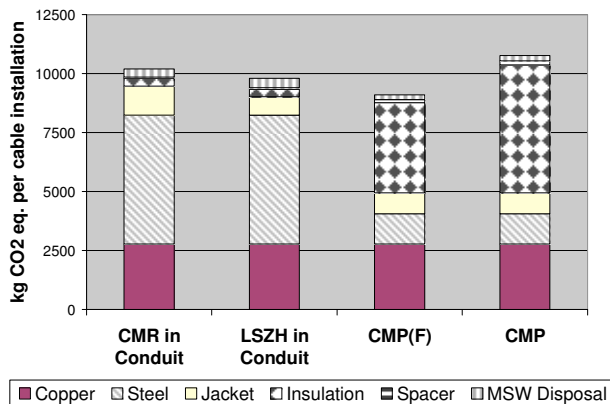


Figure 3. Human Toxicity from Industrial Emissions by Installation Component

Steel accounts for close to 60% of the LSZH and CMR installations contributions to global warming. On a cable only basis (i.e. excluding steel), copper accounts for approximately 55% of the greenhouse burdens associated with the LSZH or CMR cables and approximately 40% of the CMP cables.

The fire performance of PFP insulation allows for less steel use in the CMP installations which compensates for the additional GHG emissions from higher energy use and fluorocarbon emissions in the PFP insulation supply chain. For a CMP installation, a net increase of GHG emissions when compared to LSZH or PVC installations of 570-965 kg CO₂ eq. per installation (6-9%) results. However, a CMP(F) installation provides a net decrease of 700-1090-kg CO₂ eq. per installation, or 31-49 kg-CO₂ eq. per installed km of cable.

Carbon dioxide from energy use represents about 90% of the GHG burden for the CMR and LSZH installations but only 70% for the CMP installation.

4.3 Human Toxicity from Industrial Emissions

In life cycle analysis, social, regional, and economic differences can influence the interpretations of life cycle results across multiple impact assessments. This is also true within the human toxicity impact category due to the varied toxicological impacts of various chemicals. Human toxicity from industrial emissions can vary due to emission quantity, can have acute and/or chronic effects, and may

have impact globally, regionally, or only locally, as chemicals are released to the air, water, and/or soil. For this LCA study, the CML 2 2000 impact assessment method is used to quantify the toxicological effects from the industrial emissions from installed cabling systems [7]. This method normalizes the impacts of various industrial emissions as equivalent emissions of 1,4-dichlorobenzene.

For cabling systems, human toxicity is dominated by the copper wire life cycle. Despite a 95% recycle rate, copper results in 69-83% of the installation burdens depending on case and 87-93% of the cable burden (i.e. excluding steel). Steel is also shown to have a significant impact on human toxicity, particularly for the CMR and LSZH installations at 25% of the installation burden. The lower mass use rate of steel allowed by the fire performance of PFP insulation for the CMP cases reduces the steel burden to 7%. The plastics in the cable installations account for 5-11% of the human toxicity burden. The burden of the plastics seems proportionally higher in the PFP cases due to the reduced steel use rate. Human toxicity impacts from Municipal Solid Waste (MSW) landfill and incineration emissions account for less than 4% of the overall burden. Figure 4 shows the human toxicity impact from industrial emissions for each of the four cabling installations, broken down by installation component.

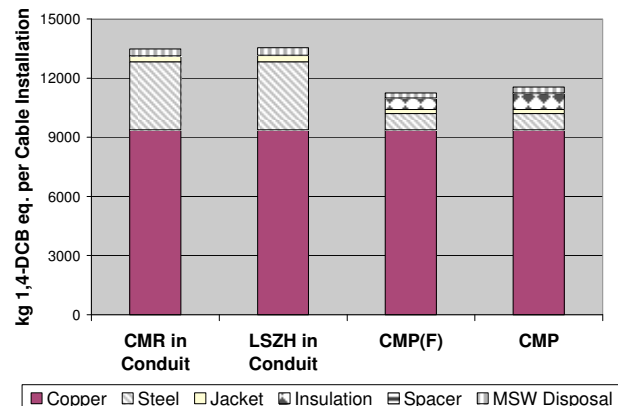


Figure 4. Human Toxicity from Industrial Emissions by Installation Component

CMP and CMP(F) cable installations are shown to have a lower human toxicity potential than either the LSZH or CMR cabling systems, but the main observations are the importance of copper on all four systems and the reduced impact from steel for the CMP cases due to a lower steel use rate allowed because of the fire performance of the PFP insulation.

4.4 Other Impact Categories

Similar trends continue for air acidification, photo-oxidation, and eutrophication potential as detailed in figure 5. For photochemical oxidation and air acidification potential, the CMR installation has the highest impact at 10.2 kg C₂H₄ eq. and 277 kg SO₂ eq, respectively. The LSZH installation has the highest eutrophication potential at 14.7 kg PO₄ eq. Emissions associated with copper and steel dominate all of these categories at a combined 62% - 96% of the impact depending on installation and impact category.

Ozone depletion is not included in this analysis due to the lack of emissions data for brominated flame retardants. Brominated hydrocarbons are identified as some of the most potent ozone depleting substances with impacts more than seven times higher

than the main ozone depleting substance found in the PFP supply chain.

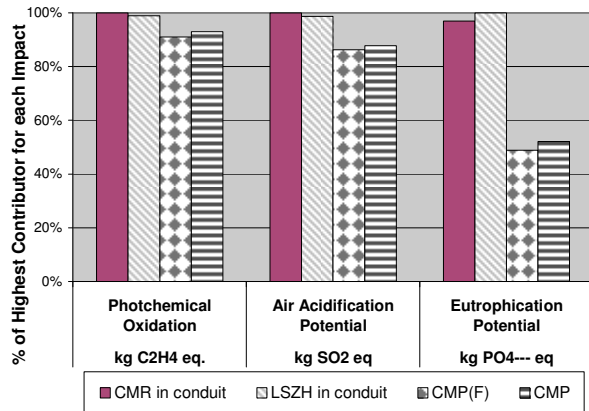


Figure 5. Additional Impact Assessments by Cable Installation Type

as a percentage of the cable type with the maximum impact in each impact category studied. With the exception of eutrophication, the CMP cable has the highest burdens when the copper is ignored.

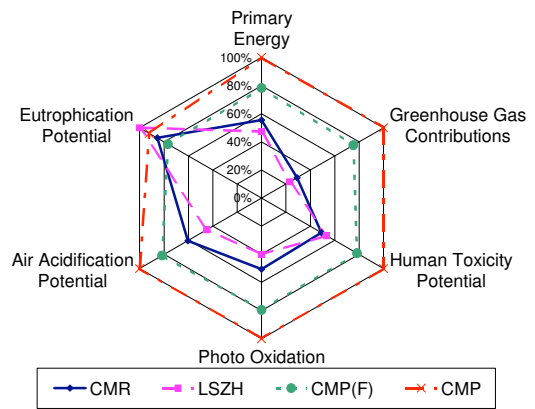


Figure 6. Cables without Copper – Relative Environmental Impacts by Cable Type

4.5 Sensitivity & Scenario Analysis

Cables and cable installations are not created equal. The mass of the different cable components is subject to substantial variation depending on CAT 6 cable type and manufacturer. Further, the building layout and building technical requirements could change the ratio of cable to steel. Sensitivity across these scenarios can be done by mass ratio with the data already presented and is left to the reader as the permutations are numerous.

4.5.1. Sensitivity to Copper Life Cycle Model. The environmental impact from copper calls for a sensitivity analysis of potential copper life cycle models. As discussed earlier, regional data from Ecoinvent™ are available [2]. In addition, a model from ETH-Zurich [11], and a revised module from Ecoinvent™ for North America using emissions from the IPCC BAT for copper production are analyzed [5]. Table 5 shows the environmental impacts from copper across all impact categories on a 1-km basis of 22.5 AWG copper with a 95% recycle rate.

Table 5. Copper Model Sensitivity – Cradle-to-Grave for 1-km of 22.5 AWG Copper

Impact Category	Units	RNA	RNA w/ IPCC	RER	GLO	ETH
Greenhouse Gases	kg CO2 eq	9075	9389	6522	7627	9283
Human toxicity	kg 1,4-DB eq	30796	24317	130969	674225	5336
Photochem. oxidation	kg C2H4	26.6	3.8	9.3	32.6	7.4
Acidification	kg SO2 eq	688	119	239	832	187
Eutrophication	kg PO4--- eq	10.8	10.8	4.1	8.2	3.8
Non renewable Energy	MJ-Eq	130165	130165	93117	111555	146476

All regional data is from Ecoinvent™. RNA = North America, RER = Europe, GLO = Global. A case for IPCC data in place of emissions in the RNA model is labeled RNA-IPPC. ETH-Zurich data is European based and included as another data source, but is at a lower data quality due to lower confidence in the recycle model

4.5.2 Functional Unit Impacts The following Figures 6 through 8 showcase the importance of identifying appropriate functional units in LCA studies. Omitting items that are similar across all alternatives and relatively unimportant is common practice in life cycle studies. However, omitting similar items with substantial impacts can lead to misidentifying the key contributors.

For the jacket insulation, and spacers (i.e. a cable without the copper conductor), Figure 6 shows the relative impacts for each cable type

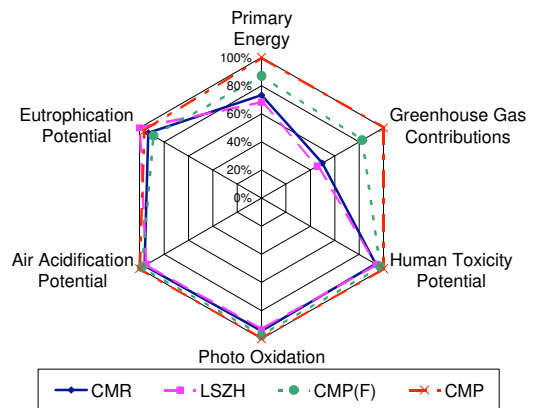


Figure 7. Cables with Copper (Uninstalled) – Relative Environmental Impacts by Cable Type

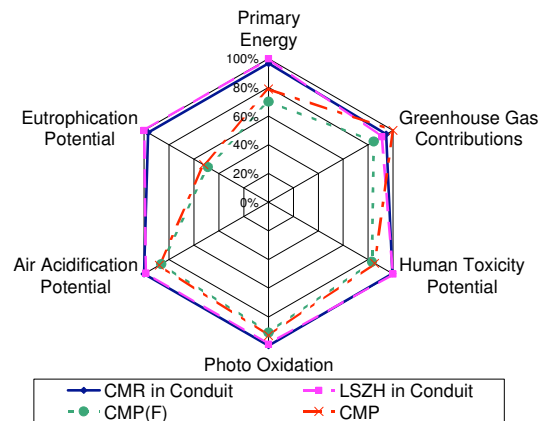


Figure 8. Installed Cables with Copper – Relative Environmental Impacts by Cable Type D1.

For complete cables (with the copper conductor), Figure 7 shows how the environmental impacts from copper dominate the overall impacts of the cable as human toxicity, photochemical oxidation, air acidification and eutrophication are all essentially the same for each cable type. The CMP cables remain higher in energy use and GHG impacts but the differential has become smaller.

Once the steel required for fire-safety performance and support is added to complete the cable installations, all of the environmental impact categories evaluated in this study favor the CMP(F) installation as shown in Figure 8. The CMP installation has consistently lower burdens than CMR and LSZH installations, except for greenhouse gas emissions. CMP or CMP(F) cable becomes the environmentally-favored cable when a holistic and truly functionally consistent basis is applied.

A significant portion of the copper burden is unavoidable as elimination of the copper eliminates the actual function of the cable. However, Table 6 shows the environmental impact reductions as a percentage of the base case for each cable installation type if 23.5 AWG copper conductors could be used instead of 22.5 AWG.

Table 6. Potential Environmental Impact Reductions for Cable Installations with Reduced Copper Use

Impact Category	Units	CMR Install	LSZH/PR Install	CMP(F) Install	CMP Install
Greenhouse Gases	kg CO2 eq	6%	6%	6%	5%
human toxicity	kg 1,4-DB eq	15%	15%	18%	17%
photochem. oxidation	kg C2H4	17%	17%	18%	18%
acidification	kg SO2 eq	16%	16%	18%	18%
eutrophication	kg PO4--- eq	5%	5%	10%	9%
Non renewable. all	MJ-Eq	6%	6%	8%	7%

Reductions in burden due to copper gauge reduction to 23.5 AWG as compared to 22.5 AWG in the base case

A more detailed demonstration of the problem caused by omitting the copper conductor in a wire and cable life cycle analysis is presented in Table 7. On a cable only basis and for virtually every impact assessment in CML 2 2000 and cumulative energy demand categories for both CMR and CMP cables, the copper conductor has the highest environmental burden.

5. Life Cycle Recommendations & Limitations

Primary data sets from multiple copper wire and steel conduit/wireway manufacturers would be preferred to the secondary data used here. Using the IISI LCA data for the steel is satisfactory for the steel manufacturing process, but the modeling of downstream processing from the hot-rolled steel, including the galvanizing process is desirable from actual suppliers. Multiple facilities would need to be evaluated to address the substantial variation in emissions resulting, presumably, from different feedstocks for copper production.

The variability in cable composition among cable manufacturers also leads to limitations in this study. A lack of data for plenum cable applications for both CMR and LSZH cabling systems required simplifying assumptions on cable materials. However, due to the impacts from copper and steel, differences in material use rates within the cables will only become substantial issues if fire-performance or functionality is changed.

Table 7. Potential Environmental Impact Reductions for Cable Installations with Reduced Copper Use

Impact Category	Unit	CMR Cable w/ Copper	CMR Cable No Copper	% From Copper
Abiotic depletion	kg Sb eq	34.1	14.9	56%
Greenhouse gas emissions	kg CO2 eq	4652	1870	60%
Ozone layer depletion	kg CFC-11 eq	1.9E-04	8.2E-05	57%
Human toxicity	kg 1,4-DB eq	10003	610	94%
Fresh water aquatic ecotox.	kg 1,4-DB eq	8003	745	91%
Marine aquatic ecotoxicity	kg 1,4-DB eq	5912643	1218324	79%
Terrestrial ecotoxicity	kg 1,4-DB eq	44.9	7.9	82%
Photochemical oxidation	kg C2H4 eq.	8.6	0.5	94%
Air Acidification	kg SO2 eq	223.7	13.7	94%
Eutrophication	kg PO4--- eq	5.7	2.4	58%
Energy, Non renewable-Total	MJ-Eq	73531	33591	54%
Energy, Non renewable-fossil	MJ-Eq	63076	29491	53%
Energy, Non-renewable-nuclear	MJ-Eq	10477	4122	61%
Energy, Renewable-biomass	MJ-Eq	892	59	93%
Energy, Renewable-wind, solar, geothermal	MJ-Eq	35	12	66%
Energy, Renewable-water	MJ-Eq	3840	443	88%

Impact Category	Unit	CMP Cable w/ Copper	CMP Cable No Copper	% From Copper
Abiotic depletion	kg Sb eq	40.9	21.8	47%
Greenhouse gas emissions	kg CO2 eq	7801	5019	36%
Ozone layer depletion	kg CFC-11 eq	2.7E-02	2.7E-02	0%
Human toxicity	kg 1,4-DB eq	10422	1030	90%
Fresh water aquatic ecotox.	kg 1,4-DB eq	7965	708	91%
Marine aquatic ecotoxicity	kg 1,4-DB eq	7493077	2798758	63%
Terrestrial ecotoxicity	kg 1,4-DB eq	43.7	6.7	85%
Photochemical oxidation	kg C2H4 eq.	8.9	0.8	91%
Air Acidification	kg SO2 eq	228.5	18.5	92%
Eutrophication	kg PO4--- eq	5.6	2.3	59%
Energy, Non renewable-Total	MJ-Eq	87315	47375	46%
Energy, Non renewable-fossil	MJ-Eq	77474	43889	43%
Energy, Non-renewable-nuclear	MJ-Eq	9857	3502	64%
Energy, Renewable-biomass	MJ-Eq	995	162	84%
Energy, Renewable-wind, solar, geothermal	MJ-Eq	49	26	48%
Energy, Renewable-water	MJ-Eq	3753	356	91%

Analysis assumes 1-km of cable with and without copper on a cradle-to-grave basis. Impact assessments are from SimaPro™ LCA software incorporating both the CML 2 (2000) v 2.1 and the Cumulative Energy Demand V1.1 (adjusted for all LHV) methods. Comparison of CMR and CMP data is not appropriate due to functional differences associated with fire performance, flame spread, and smoke generation. Ozone depletion data is incomplete due to lack of information available on brominated flame retardants.

6. Fire Safety Tests & EMT Coupling Failure

Many US jurisdictions require non-plenum communication cable to be placed in Electrical Metallic Tubing (EMT) conduit systems. Recent real-scale fire research has shown that some EMT conduit systems have failed to protect the enclosed non-plenum cable. A series of IEC 60331-21 cable fire research tests have been conducted to better understand and document these failures.

In these experiments, two 18" lengths of metal conduit are joined by a die-cast metal coupling (Figure 9). Non-plenum cable is fed into the conduit to meet the commonly used 40% fill rule. A thermocouple is positioned inside the conduit, close to the coupling. The coupling and conduit are then exposed to a 10" IEC gas ribbon burner. Initial results from these IEC tests suggest that, even at relatively low temperatures, systems using die-cast metal couplings are likely to offer less than 4 minutes of fire protection.

The images below were extracted from the video created during the IEC 60331-21 cable fire research tests. The test time durations are shown in each frame and thermocouple temperature is shown in certain images.



Figure 9 – Die-Cast Coupling attached to two 18" lengths of metal conduit (Start of Test).



Figure 11 – Flames from the non-plenum cable are visible.

In the IEC 60331-21 cable fire research test, after 2 minutes 35 seconds, at approximately 294 °C, the die-cast coupling shows blistering, weakening, melting, and rotational failure (Figure 10).

After nearly three and one half minutes, flames have increased and the misshaping of the die-cast coupling has become more evident (Figure 12).

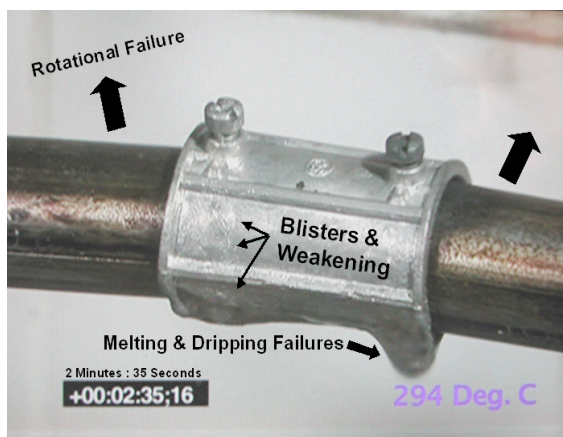


Figure 10 – Melting, Dripping and Rotational Failures of Die-Cast Coupling



Figure 12 – Flames have increased and coupling now appears to be misshaped.

After just over 4 minutes test duration, at approximately 404°C, the EMT joint has melted, rotated, and failed. The flames that are visible emanate from the now unprotected non-plenum cable burning inside the metal conduit and not from the testing apparatus (Figure 13).



Figure 13 - EMT joint failure occurs at 404°C.



Figure 14 – At 425°C. – (Apparatus used for IEC 60331-21 cable fire research tests shown).

As temperatures rise to approximately 530°C, the complete failure of the die-cast coupling is evident at 6 minutes, 17 seconds into the test (Figure 15).



Figure 15 - Complete failure of Die-Cast Coupling at 530°C, with non-plenum cabling flaming.

At nearly 20 minutes, the complete failure of the die-cast coupling can be seen (Figure 16).



Figure 16 - Complete failure of Die-Cast Coupling

This IEC fire test shows the EMT die-cast metal conduit-couplings first blister and distort at approximately 300°C. The couplings then melt, drip and break open in about 4 minutes, critically failing to protect the low fire performance, non-plenum, cables from burning inside the conduit.

Because temperatures in small and large diffusion flames can easily exceed 1,000°C, die-cast couplings are subject to failure regardless of the size and intensity of any fire. Figure 17 supports this claim by depicting a microstructure of a flame and its regional temperature profile.

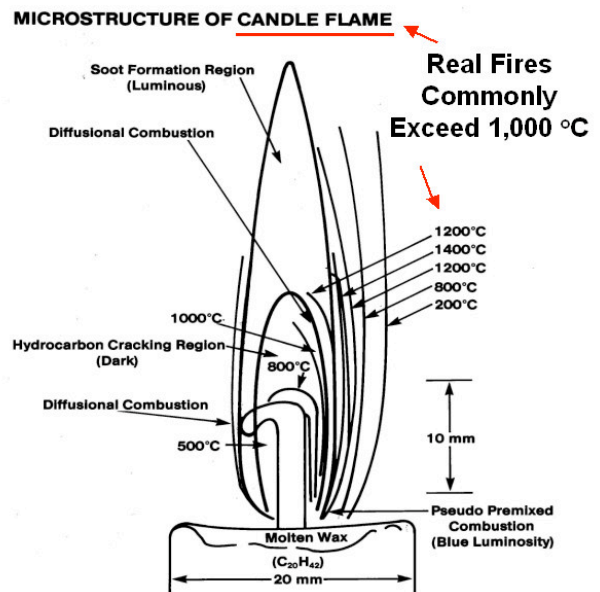


Figure 17 - Source: Pure & Appl. Chem. Vol. 62, No. 5, pp. 839-849, R. Fristrom

Photographs taken during these IEC 60331-21 cable fire research tests resemble AHJ fire-incident photographs taken during actual fire investigations, suggesting that this type of failure is not confined to the laboratory (Figure 18).



Figure 18 – AHJ Fire-Incident Image showing Joint Separation, Visible Wires and No Couplings Visible.

7. Conclusions

A holistic study for the comparison of environmental impacts resulting from the life cycle of CAT 6 LAN cabling systems in U.S. plenum space installations was performed. Manufacturing differences cannot be compared directly on a life cycle basis until they are put on a functionally equivalent basis. The superior fire-performance of PFP insulation allows the direct installation of cable without the use of steel conduit or trunking. The inclusion of all key contributors is required to properly analyze possible interactions to reduce the overall environmental burden.

The life cycle data presented in this study do suggest materials selection can make a difference in the environmental performance of the cable. However, the basis for the selection of environmentally-preferred cable materials is that they reduce either the copper use through superior electrical performance or reduce the steel use through superior fire-performance.

However, recent fire safety experiments cast doubt on assumed functional equivalence for fire performance based on code compliance for cabling requiring conduit or trunking. The tests have shown that non-plenum cable, when protected by metal conduit and die-cast couplings, cannot survive temperatures of approximately 400°C for more than 4 minutes without catastrophic failure. Since die-cast conduit-couplings have been used broadly for many years, these new R&D findings have serious infrastructure fire-safety implications.

Therefore, as modeled, the CMP cabling (i.e. plenum rated per NFPA 262 fire tests), installed without metal conduits, has been shown to be environmentally superior. Such CMP cables may also maintain superior fire performance when compared to systems using EMT metal conduit with die cast couplings to try to fire-protect CMR, low-smoke zero-halogen (LSZH), and other types of much more combustible non-plenum (rated) cables.

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INNOVATIVE METHOD FOR INVESTIGATING THE FACILITY DAMAGE ASSESSMENT PROCESS

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ABSTRACT

Facility damage assessments are performed in the hours and days following a disaster event, and the individuals who conduct these assessments are often volunteers from the impacted community. Consequently, these volunteers typically have limited knowledge of facilities, which may cause biased and inaccurate damage judgments. To overcome these important limitations in skills and knowledge, a study was initiated as part of a collaborative effort between the civil engineering and psychology communities. This study investigated the damage assessment process using a virtual computer-based environment. Different types of damage were simulated on three-dimensional models of buildings, and participants were asked to view the damaged facilities in order to develop a description of the damage and an estimate of the quantity and magnitude of the destruction. Following their assessments, participants were asked a series of questions to ascertain how they arrived at their judgments. The data analysis revealed that engineers nearly always classified the severity of the damage correctly and relied on their technical and conceptual knowledge of engineering to form their judgments. In contrast, non-engineers nearly always incorrectly classified the severity of damage and relied on mental anchors, heuristics, and non-technical knowledge to form their judgments.

BACKGROUND

The accurate estimate of facility damage and repair costs is critical to the recovery of communities following a disaster event. Federal and state financial assistance is contingent upon the cost of the damage; hence, an inaccurate estimate that results in a denial of aid can negatively impact a community's ability to recover. Yet, communities often rely on volunteers with diverse non-technical skills to perform initial assessments of facility damage in the hours immediately following the disaster event. These volunteers often have little or no damage assessment training and limited knowledge of facilities. Consequently, these volunteers can learn a lot from experts with extensive knowledge of engineering and construction, and, as a result, the transfer of basic technical skills and cognitive processes from expert engineers to non-expert volunteers is a critical issue addressed in a preliminary research project conducted by a joint team of civil engineers and cognitive psychologists.

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The current approach to training non-expert volunteer damage assessors consists primarily of workshops and classroom training with little or no hands-on instruction. Yet research in cognitive psychology has established that people tend to underestimate physical factors when they have insufficient background knowledge, such as previous experience, or when the physical factors are complex (LeBoeuf and Shafir 2006). For example, volumes (which involve a complex multiplicative relationship among observable quantities) tend to be estimated less accurately than lengths (Stevens and Guirao 1963). Research has also shown that hands-on training is essential to acquire the skills to perform complex tasks. Actual conditions at the site of an extreme event will require decision-making accuracy that cannot be acquired in a classroom setting. Unfortunately, training non-experts to assess facility damage is difficult in real-time settings, where disasters have a low probability of occurrence and can create hazardous situations. As a result, virtual 3-D facility models were tested for their effectiveness at training individuals to conduct damage assessments. The models were also used to investigate the feasibility of conducting a more extensive study of the factors that impact assessment accuracy and the ability to transfer expert damage assessment knowledge to non-experts.

WHO ASSESSES DAMAGE AND HOW

The process of estimating the extent, severity, and cost of facility damage begins when an assessor – typically a volunteer – visits the damaged facility and makes a judgment about the extent of the destruction based on observable visual information (Federal Emergency Management Agency (FEMA) 2002). Communities often draw these volunteers from non-profit relief agencies, citizen action groups, and public agencies, and, as a result, most of the volunteers are not engineers or construction workers and have limited knowledge of construction materials and method. Consequently, these individuals use heuristics in conjunction with their limited knowledge of construction practices to develop judgments about facility damage, which may cause a systematic bias in non-expert damage assessments. Research in cognitive psychology has further established that people tend to generate mental anchors from physical cues they observe, and these *physical anchors* tend to cause biased estimates of physical values (LeBoeuf and Shafir 2006). As a result, non-expert assessments tend to be error prone and inaccurate, and government emergency response agencies are reluctant to view these assessments as credible or use them to establish financial aid entitlement (Menches 2007).

While the vast majority of initial assessments conducted after a flood or tornado are performed by volunteers, assessments following an earthquake are typically performed by structural engineers and building inspectors (i.e., “experts”) (Applied Technology Council (ATC) 1989). These expert assessors focus on structural safety, and, as a result, apply judgments that require a higher level of theoretical knowledge about engineering and construction (ATC 1989). The procedures for performing a post-earthquake safety evaluation were developed by the Applied Technology Council (ATC) in 1989 following several devastating earthquakes in California. The procedures are based on a review of existing literature and best practices reported by engineers, building inspectors, and disaster workers. As such, the procedures are largely practice-based and do not necessarily reflect the theoretical engineering

knowledge, perceptual cues, or mental models of damage that are actually drawn upon when an expert performs a damage assessment. Yet, evidence suggests that experts typically engage in perceptual hypothesis testing in which they use visual information about damage to generate a description of, and an explanation for, the type of damage observed (Markman 1999). This expert assessment also involves drawing upon their technical understanding of the particular type of structure that is damaged, the event that caused the damage, and more general principles of structural analysis and construction methods when forming judgments about facility damage. Unfortunately, this knowledge is typically lacking in non-expert volunteer assessors.

TRAINING AVAILABLE TO DAMAGE ASSESSORS

Training for volunteer damage assessors is limited and consists primarily of workshops and classroom training with no hands-on component (American Red Cross 2006; Federal Emergency Management Agency (FEMA) 2007; State of Texas 2006). This delivery method tends to lack the real-world element that is critical for acquiring the necessary skills to evaluate complex phenomena. The small number of actual disasters, combined with their low probability and hazardous nature, makes hands-on training difficult and often impractical. Yet, research has demonstrated that hands-on training is more effective at acquiring the necessary skills to perform complex tasks than classroom settings (Seidel et al. 2005).

Virtual computer-based training environments provide learning experiences that are similar to hands-on and provide flexibility in the types of disasters and severity of damage simulated. Virtual environments (VEs) are viewed by some researchers as providing the most effective methods for improving confidence in decision-making through demonstration of actual command and control scenarios (Gigley 1997). All aspects of the environment can be controlled digitally, the user can be monitored via camera or computer, and data can be collected automatically, affording experimental control that is not possible in an actual damage environment (Lotens and Riemersma 1997). A virtual training environment also provides an opportunity to mix real images with virtual components to best portray actual field conditions. Furthermore, there is evidence that the experience gained through virtual training can improve the mental mapping process of novices, reducing their tendency to randomly “explore” the environment and increasing their search for the familiar, much like the experts (Lotens and Riemersma 1997). The empirical evidence indicates that VEs are effective at transferring desired skills, particularly when the environment is designed to target the development of a *specific* skill, such as damage assessment.

TRANSFERRING SKILLS FROM EXPERTS TO NON-EXPERTS

A respectable body of knowledge exists on the transfer of skills from expert professionals to novice professionals, particularly among fire fighters, military commanders, and nurses (Klein 1998; Regian 1997). Regian’s work is particularly noteworthy because he investigated the transfer of complex skills, rather than basic skills, using 2-D and 3-D computer-based applications. Ultimately, both 2-D and 3-D environments were found to successfully train novices. Likewise, Gigley (1997) noted that some types of skills could be acquired just as well by using 2-D representations and pictures as they could by using virtual 3-D environments.

However, there is strong anecdotal evidence that rehearsal and visual recognition using 3-D environments provides more effective skills-transfer, and that “hands-on” training using 3-D environments is essential to transfer complex skills (Gigley 1997).

Currently, damage assessment training for non-experts occurs in a classroom setting. Gigley (1997) has demonstrated that this training does, indeed, result in some skills transfer when the training involves the use of pictures and videos. However, the relatively sterile classroom setting does not generally induce the realistic stress that is produced using a simulated real-world setting. The virtual environment gives participants a sense that their experience is real and gives them a more confident picture of what they can expect in the actual situation (Gigley 1997). Hence, there is strong support that VEs are an effective medium for transferring skills. Unfortunately, little is known about the technical and cognitive skills that can be transferred from an expert assessor to a non-expert volunteer, especially when the skills must be applied in a dynamic setting. While the transfer of skills is a complex process that is not easily understood, the measurement of skills transfer using different learning media is possible using carefully controlled experiments (Proctor and Vu 2006).

RESEARCH PROBLEM, GOALS, AND OBJECTIVES

Given the current knowledge gap between expert and non-expert assessors, and the importance of non-expert judgments on the initial estimate of the extent and severity of a disaster, there is a critical need to understand the factors that impact the judgments of expert and non-expert damage assessors and to develop strategies for effectively transferring expert knowledge to non-experts. The objectives of the preliminary study were, therefore, to (1) investigate the feasibility of using virtual environments to identify the technical, perceptual, and conceptual knowledge used by experts and non-expert volunteers to assess facility damage, and (2) develop an initial sense of the magnitude of the discrepancy in assessment accuracy between these two groups. The expectation is that the results of the study can be used to improve the decision-making process of non-expert assessors who have limited knowledge of facilities but are nevertheless a critical part of a damage assessment team.

EXPERIMENTAL METHODS

A preliminary investigation was conducted to evaluate the decision-making process of a broad range of people who might potentially become expert or non-expert damage assessors (i.e., architects, volunteer relief workers, etc.). The aim of the study was to understand the differences in judgments of severity by contrasting the damage assessment processes, knowledge, and perceptions used by experts to those used by non-experts. The experimental method involved assessing damage using 3-D models of residential structures so that the damage environment could be controlled. Two types of houses (expensive and inexpensive) with two types of damage (roof and crack damage) were simulated using 3-D computer models (Fig. 1). Two levels of damage were also simulated (moderate and severe) resulting in eight different damaged residential models. The event that caused the damage was not specified because the damage was meant to be general. The experiment took place at the Texas Homeland Security Conference in December 2007, and participants were solicited from the population of attendees. Overall, nine individuals were willing to

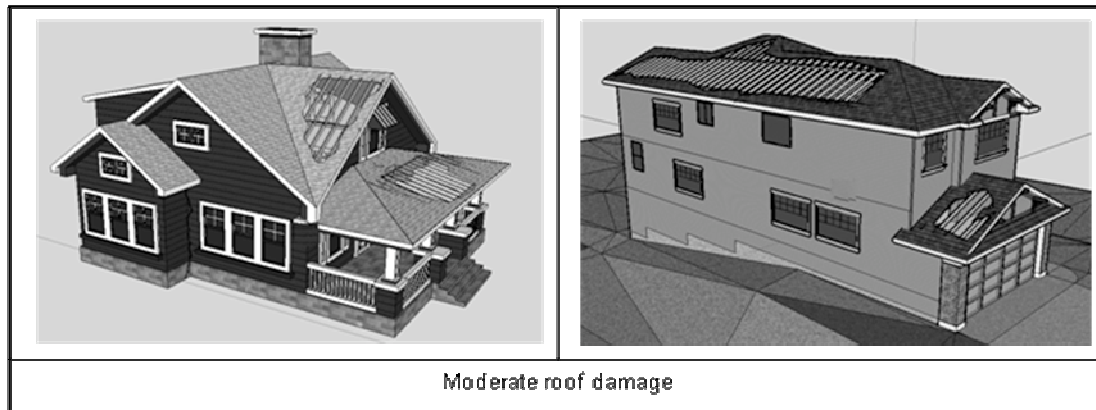


Figure 1. Sample 3-D facility models used in the initial research.

participate in the preliminary investigation. Approximately 55% were experts (i.e., engineers, architects) while 45% were non-experts (i.e., nurses, relief workers). Participants were asked to assess damage to one of the randomly selected 3-D models. Specifically, they were asked to develop a description and estimated quantity of damage and to record their assessment on a standard assessment form. Participants were observed by the proctor as they used the models to assess damage in order to record their actions. Following the observations, a brief interview was conducted to probe their decision-making processes and to document the heuristics and knowledge they used to prepare their assessments and estimates.

RESEARCH FINDINGS

Although the sample size was small, the results were compelling. Overall, 75% of those with non-technical backgrounds (i.e., non-experts) misclassified the severity of the damage, with 50% misclassifying severe damage as moderate and 50% misclassifying moderate damage as severe. In contrast, 80% of those with technical backgrounds (i.e., experts) correctly classified the damage (Table 1).

The participants were also asked to estimate the quantity of damage they observed, and *nearly all of the participants* (89%) inaccurately estimated the area of roof damage, while 80% of those who estimated the length of crack damage produced accurate estimates. This is consistent with previous research that found that areas tend to be estimated less accurately than lengths (Stevens and Guirao 1963). This finding also highlights bias that may be introduced by the experts, in addition to bias that is likely introduced by the non-experts, which will need further investigation.

When asked to describe the knowledge used to make decisions, 80% of those with technical backgrounds mentioned their engineering, design, or construction knowledge, while only 25% of those with non-technical backgrounds mentioned using direct knowledge of construction or facilities (Table 2). Most of the non-experts attempted to draw from knowledge that was most similar to buildings or construction, such as home ownership. Furthermore, technical experts were more likely (2 to 1) to use the known dimensions of building elements (windows, doors) to estimate the quantity of damage (i.e., “the hole is about two window-widths or six feet wide”), while many non-experts estimated proportions (i.e., “the hole looks twice as big as the window”). Hence, technical experts were more likely to draw from their knowledge

Table 1. Judgments of Severity and Accuracy of Estimated Damage

Occupation	Actual Roof Damage Severity	Est. Roof Damage Severity	Actual Damage	Reported/Estimated Damage	Accuracy (Roof/ Crack)
Mental Health Specialist	Severe	Moderate	Roof damage = 10.5 SF Found./siding crack = 21 ft	Roof damage = 16 SF Crack damage = No response (NR)	Inaccurate/ No Response
Business Developer	Severe	Severe	Roof damage = 12.5 SF Found./siding crack = 27.5 ft	Roof damage = 3 SF Found./siding crack = 25 ft	Inaccurate/ Accurate
Registered Nurse	Moderate	Severe	Upper roof damage = 312 SF Lower roof damage = 32 SF Foundation crack = 5 ft	Roof damage = 500 SF Garage roof damage = 24 SF Foundation crack = 3 ft	Inaccurate/ Accurate
Public Health Manager	Severe	Moderate	Roof damage = 7.88 SF Foundation crack = 5 ft	Hole in roof = 8 SF Foundation cracks = NR	Accurate/ No Response
Military Civil Engineer	Moderate	Severe	Upper roof damage = 312 SF Lower roof damage = 32 SF Found./siding crack = 26 ft	Roof damage = 240 SF Roof Damage = 9 SF Found./siding crack = 36 ft	Inaccurate/ Inaccurate
Architect	Severe	Severe	Roof damage = 12.5 SF Siding crack = 5 ft	Roof damage = 28 SF Siding crack = 4 ft to 6 ft	Inaccurate/ Accurate
Senior Civil Engineer	Severe	Severe	Roof damage = 7.88 SF Foundation crack = 5 ft	Roof damage = 25 SF Foundation cracks = 5-7 ft	Inaccurate/ Accurate
Warning Coordinator/ Meteorologist	Moderate	Moderate	Upper roof = 56 SF Lower roof = 60 SF Found./siding crack = 11.5 ft	Roof damage = 24 SF Lower roof damage = 18 SF Foundation crack = NR	Inaccurate/ No Response
Structural Engineer	Moderate	Moderate	Upper roof = 56 SF (5%) Lower roof = 60 SF (13%) Foundation crack = 2.5 ft	Roof damage = 20% of surface	Inaccurate/ No Response

of facilities while non-experts were more likely to use heuristics based on their perceptions.

Observations of how the experts and non-experts performed the assessment revealed that experts tended to immediately focus on a single damaged element and examine it until they came to a decision about the damage before they continued searching for the next damaged element. In contrast, non-experts examined the entire structure several times before selecting an element to assess, and while they spent less time assessing overall, they spent more time searching for the damage.

Table 2. Knowledge Used to Assess and Estimate Damage

Occupation	Knowledge used to make judgments
Mental Health Specialist	Geology class
Business Developer	Worked at lumber yard
Registered Nurse	Went through home inspections and had house built
Public Health Manager	No real knowledge
Military Civil Engineer	Engineering education and academic experience
Architect	Experience as architect in developing roof and framing plans
Senior Civil Engineer	Knowledge of structural components of roof
Warning Coordinator/Meteorologist	Professional experience physically assessing damage to buildings
Structural Engineer	Used specialty in design of marine structures and wind engineer with multistory wind testing experience

Based on an examination of expert performance, there are six key steps that experts engage in to assess damage (Fig. 2).

1. ***Experts think about the type of event that has occurred*** and form an expectation (initial hypothesis) about the overall damage they will observe. They are able to generate predictions for the type of damage that is likely to arise from a particular disaster (i.e., flood, tornado).
2. ***They search for and select a damaged element***, develop an expectation (hypothesis) about the damage to it, and inspect the element thoroughly, using their knowledge of engineering and construction to recognize the absence of missing elements. They are able to determine important aspects of the design of a building from its surface properties. Thus, experts can generate expectations of the properties of a building just by looking at it.
3. ***Experts think about and decide whether the damage is consistent*** with their expectations and knowledge (i.e., test the hypothesis).
4. ***If the damage is consistent with their expectations***, they combine the visible damage with their knowledge of engineering and construction to assess the severity of the damage to a building. Experts can use their technical knowledge to predict more subtle signs of damage that might not be obvious to a casual observer. They record their decisions and then move to the next damaged element and repeat steps 2 and 3.
5. ***If the damage is inconsistent with their expectations***, the experts search their technical knowledge for the next most plausible explanation and repeat steps 3 and 4 until satisfied. Because of the need to complete the initial assessment rapidly, they will only test additional hypotheses about the damage if their first theory is incorrect.
6. ***The assessment process continues until all elements have been judged***. Experts have a vocabulary for describing the damage that supports subsequent parts of the damage assessment process, such as generating a cost estimate for repair of the damage at a later stage.

This six-step model of expert damage assessment is a significant first step towards codifying the fundamental knowledge and cognition used by experts to make accurate decisions about facility damage.

The process used by non-experts to assess damage was also observed and some important differences were noted. First, the non-experts rarely inquired about the event that caused the damage and did not generate specific expectations about the damage they would observe. As a result, they examined the entire building several times before selecting an element to be examined in detail. They drew from whatever background knowledge they had about buildings, structures, or construction *that most closely related* to the damaged element in order to judge the extent and severity of damage. For example, knowledge of a home inspection that involved looking at the attic insulation was used to recall knowledge of a roof structure and judge the damage. The heuristics used by the non-experts to judge elements was broad, and,

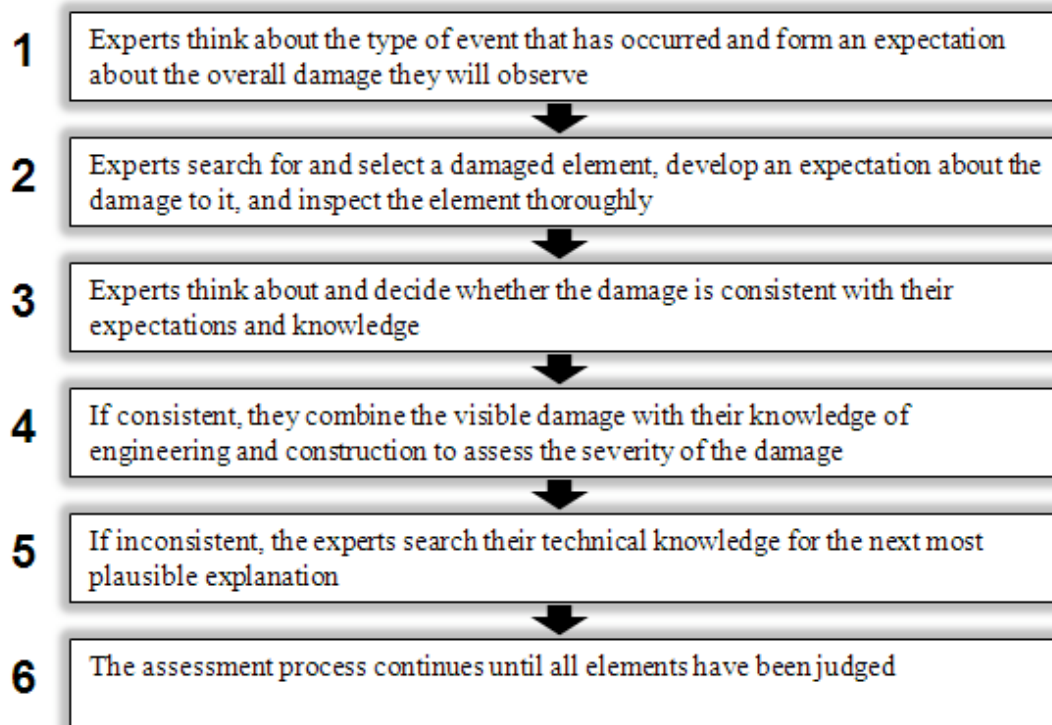


Figure 2. The six-step expert damage assessment process.

therefore, additional research is needed to understand these heuristics and how they can serve as leverage points for *effectively* transferring expert knowledge. Furthermore, non-experts were not trying to make sense of the damage based on the event that caused it but, instead, were searching for memory anchors that could help them identify the facility element and extent of damage to that element. Unfortunately, consistent with previous research (LeBoeuf and Sharif 2006), the use of memory anchors resulted in misclassifying the severity of the damage nearly 75% of the time.

The initial study shows promise that empirically-supported scientific principles for performing expert damage assessments can be established and that these principles can be used to transfer this knowledge to non-experts through the implementation of baseline interventions. However, additional research is needed to further validate the six-step expert damage assessment process and identify leverage points for effectively transferring knowledge.

FUTURE DIRECTIONS AND CONCLUSION

Eight models were developed during the initial studies and were used to elicit knowledge about expert and non-expert damage assessments. Damage unrelated to any particular event was simulated and was properly characterized by nearly all of the experts. However, many of them inquired about the event that caused the damage before beginning their assessment. Consequently, the initial studies highlighted the need for 3-D models that simulate damage to buildings that were impacted by *specific*

types of events, such as floods or tornados. Furthermore, the preliminary studies identified alarming inaccuracies in non-expert characterizations of damage severity, and, similarly, non-experts generated heuristics from knowledge that was *most closely related* to structures but was not technical, per se. Hence, there is a critical need to further investigate non-experts' processes for reasoning about facility damage and how these processes differ from the experts. The perceptions, heuristics, mental anchors, and general knowledge used by non-experts to assess damage will be explored in a follow-on study as well as how these cognitive processes facilitate or inhibit accurate assessments.

Evidence from the preliminary work suggests that experts apply top-down reasoning to perform a damage assessment by calling to mind relevant facts about disasters, structures, and construction practices that will allow them to search for evidence of damage. Non-experts, on the other hand, use a bottom-up approach to assessment in which the assessor searches for the damage and then speculates about a cause, in contrast to experts who search for a cause and then speculate about the possible damage. Furthermore, non-experts spent more time searching for damage and less time conducting the assessment overall, which might suggest that non-experts use mental shortcuts that reduce their problem-solving time but also reduce their efficiency and introduce bias into their assessments. However, both the experts and non-experts were unable to produce accurate estimates of the quantity of damage, and, as a result, bias that is introduced by *both groups* must be investigated as part of a follow-on study to understand how expert and non-expert bias impacts the damage assessment process.

The follow-on research, which will document the damage assessment process typically employed by non-expert assessors and contrast it to the expert process, has two primary aims: (1) to identify the points of divergence between expert and non-expert judgment processes, and (2) to identify leverage points for generating changes in the assessment tools and training (i.e., assessment forms, course modules) given to non-experts that will improve their damage assessment performance. This research program will ultimately address a problem that has plagued communities for decades. By identifying the scientific principles for conducting an expert-based damage assessment, and developing tools that facilitate knowledge-transfer, the research will provide data that should significantly reduce the inaccuracy of initial damage assessments that are prepared by non-expert volunteer assessors.

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Noise Control in Multipurpose Rooms

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Abstract

Throughout recent years, to minimize the cost of construction, a large number of multipurpose spaces have been built using lightweight, less expensive materials without considering or designing for noise control to mitigate any sound that is loud, unpleasant, unexpected, or undesired yet after construction is completed, noise issues are often evident within the space and, if severe enough, may render the intended function of the structure useless. To address this problem, this paper is intended to summarize some of the common solutions to resolve noise issues in multipurpose rooms. The report focuses on solutions for existing projects primarily, but it is also sensitive to budget constraints and the impact of renovation. Typical multipurpose rooms researched have a volume of 50,000-150,000 cubic feet and are expected to be used for speech activities, small music functions, and some physical sports activities. Therefore, this paper will introduce some fundamentals of room acoustics including interior surface materials and construction. Also included are typical noise issues from interior sources, solutions that can be taken within the building to attenuate noise, and the trade-offs associated with each solution.

Introduction

Multipurpose rooms offer unique acoustic challenges because owners often use these spaces for activities with differing acoustical requirements. For example, the same space used for athletic and sporting events may also be used for musical performances and speeches. The multipurpose rooms within the scope of this paper have a volume of 50,000 – 150,000 cubic feet and are expected to be used for speech activities, small music functions, and some physical sports activities. Two examples of this type of space include a school gymnasium that is also used as the school auditorium and cafeteria, and a church sanctuary that is also used for worship services, sermons, and youth activities and therefore often referred to by acousticians as a “sanctanasium.” Many multipurpose spaces exist that cannot be used to their fullest potential due to bothersome noise. In recent years, to minimize the cost of construction, a large number of spaces have been built with lightweight, less expensive materials without initial consideration or design for noise control. Consequently, after construction is completed, noise issues are often evident within the space and, if severe enough, may render the intended function useless. This paper will summarize some of the common noise problems and solutions within multipurpose rooms while being sensitive to budget constraints and the impact of renovation.

Section 1. Defining Noise

Why is noise something that owners are becoming increasingly concerned about? Noise within a building creates problems that often require significant funds to eliminate. Most commonly, noise can adversely affect a person’s health. For this reason, noise can be a liability for an owner because high sound levels can cause hearing loss and increased stress levels. Noise can also limit how effectively a person is receiving information, either speech or music, and can result in a potential loss in productivity for distracted workers. As a result of an increased liability for owners, “it is estimated that the total annual dollar sales of noise control measures has increased eightfold during the past decade in the U.S.” (Harris, 1.3).

Noise is defined as a sound that is loud, unpleasant, unexpected, or undesired (American Heritage Dictionary). However, noise can never be completely defined because it is more a subjective judgment of a listener than a measurable entity. This makes the design for noise control a difficult issue because even the most careful design may still create annoyances to some listeners while others have no objection. In all cases, regardless of who the listener is, noise is unwanted and will be affected by several factors (Long, 98):

- The level at which it is heard, or its perceived loudness
- The frequency range of the sound
- The duration and rate of recurrence of the sound

Noise is an annoyance to many people especially when it interferes with a person’s work or ability to clearly receive information. Unfortunately, annoyance is not something that can be directly measured. Therefore, to come up with some way to determine what a majority of occupants would consider reasonable levels of noise, the Noise Criterion (NC) curves were developed by Leo Beranek in 1957. These curves establish acceptable noise levels for various indoor environments and as a result, classifications for noise levels can be better defined. Figure 1 illustrates these NC curves as well as the corresponding noise classification, such as quiet,

moderate, noisy, and extremely noisy, on the right side of the figure. If the sound pressure level is measured in a room and it is determined that all or a portion of the values at specific frequencies are above the recommended NC curve, the space will be considered noisy by the occupants.

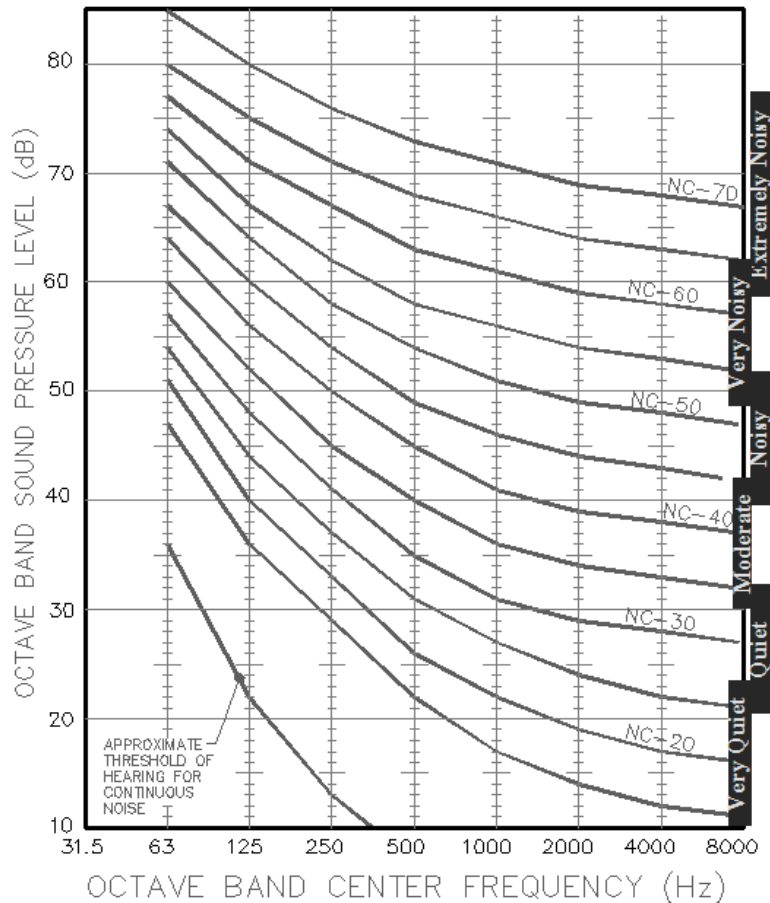


Figure 1. Standard NC Curves (Beranek)

Recommended NC values for design of different spaces have been established based on the type of occupancy of a space. Table 1 shows the recommended NC levels and approximate dB values that correspond to the type of space in an unoccupied condition. These are the standard values used to evaluate the noise level within a space. In the case of multipurpose rooms, a NC of 30-35 would be ideal and would be generally classified as a moderate noise level. This value will be the level of comparison for the multipurpose room of concern. After measurements are taken within the space, they are analyzed to determine the existing NC level and thus the amount of noise that must be reduced to fall within the acceptable range in decibels (dB).

Table 1. Recommended NC Values for Unoccupied Spaces (Modified from Mehta, Johnson, Rocafort, 168)

Space	Recommended NC value	Approximate dB value
Office Building:		
Open plan offices	30-40	38-48
Private offices	25-35	33-43
Lobbies	35-40	43-48
Churches		
Churches	25-35	33-38
School:		
Classrooms	30-35	38-43
Cafeterias	35-40	43-48
Multipurpose Spaces		
Multipurpose Spaces	30-35	38-43
Indoor Gymnasium		
Indoor Gymnasium	40-50	43-58
Performing Arts Space:		
Auditoriums and theaters	25 max.	33 max.
Music practice rooms	35 max.	43 max.

Section 2. Room Acoustics

How noise responds within a room is directly related to the construction materials used, room surface materials, and the shape of the room. As sound strikes a partition, such as a wall, floor, or ceiling, it will react in three basic ways: absorption, reflection, and transmission which are illustrated in Figure 2. How much of the sound is absorbed, reflected, or transmitted will depend on the construction and shape of the partition. This section discusses each of these characteristics and how they affect the spatial response of a room by changing the reverberation time.

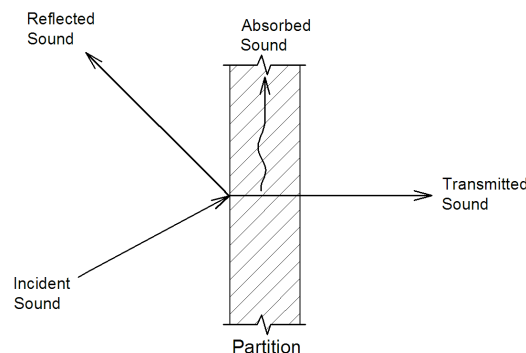


Figure 2. Reaction of Sound Striking a Partition

Spaces with a surplus of hard, reflective surfaces, such as plaster, gypsum board, brick, concrete, CMU block, metal, and so forth can become a noise problem. By promoting the continuation of the sound waves, multiple reflections will extend the amount of time before a sound becomes quieted, called the reverberation time. If the time is too long, speech will become difficult to understand and music will be difficult to hear due to the echoes. This is a common problem in spaces with parallel walls that both comprise hard, reflective materials. Sound

reflection between the walls will create flutter echo, shown in Figure 3, caused by multiple long reflections between parallel planes. As a result of flutter echo, a listener will hear several delayed reflections of the sound source and this can result in reduced clarity and understanding of the sound.

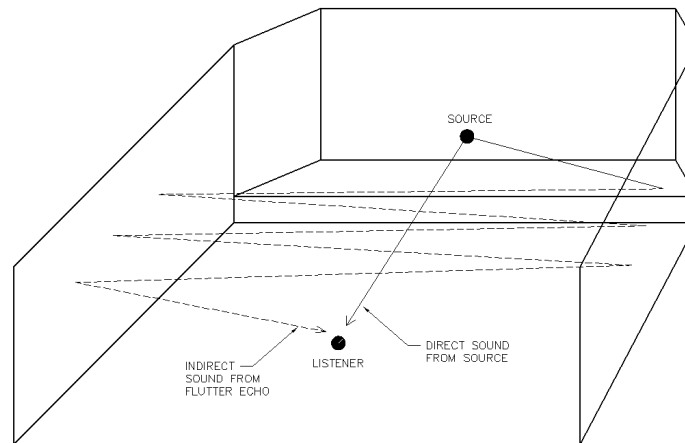


Figure 3. Flutter Echo

By contrast, materials that are absorptive in nature, such as fiberglass, fiberboard, and other porous materials, can help reduce long reflections and echoes to decrease the room reverberation time. Specifically, absorptive materials transform sound energy into small amounts heat, which is absorbed by the material. The absorption coefficient (α) is the unit used to evaluate the effectiveness of an individual sound absorbing material. It is a ratio of the amount of sound energy absorbed compared to the total amount of sound that comes in contact with the material (Mehta, Johnson, Rocafort, 61). The resulting absorption coefficient value is a unit-less number between 0.00-1.00, where a value of 0.00 indicates no absorption, and a value of 1.00 indicates that all incident sound is being absorbed. A thicker material will have a higher absorption coefficient and can absorb longer wavelengths at lower frequencies (Long, 268). Using the absorption coefficient and multiplying by the total surface area, seen in Equation 1, the absorption for a space can be calculated. Absorptive materials typically used in a multipurpose room application include fabric covered wall panels, acoustical ceiling tile, rigid fiberboard, and carpet.

Equation 1. Absorption $A = \alpha AREA$

Reverberation is the persistence of a sound in an enclosed space through multiple reflections of the sound waves. Reverberation time is the amount of time it takes for a tested sound to decay to a level of 60 dB (RT_{60}), the level that is perceived as inaudible. The desired reverberation time for a room depends on the volume of the space and the type of room use. Rooms that will be used for music are designed for a longer reverberation time, between 1.5 and 3.5 seconds. This will make the room lively enough for music to sound its best to a listener. Rooms used for speech such as classrooms or lecture halls require a lower reverberation time of 1.2 seconds or less (Olsen). For multipurpose rooms, which may be used for both speech and music, the ideal reverberation time is between 1.0 and 1.5 seconds (Mapp, 48). Reverberation

time can be approximated using the Sabine Equation shown in Equation 2 where V is the volume of the room and A is the total absorption calculated using Equation 1.

Equation 2. Sabine Equation $RT_{60} = 0.049(V/A)$

Transmission is the passage of sound from one space to another through the partition, which can be a wall separating adjacent spaces, or a floor/ceiling between vertically stacked spaces. Transmission loss is the calculated difference in sound levels between two adjoining spaces (Coffeen). As more sound passes through a barrier, the sound level difference between the spaces will be small, and consequently the barrier is considered to have a low transmission loss. The preferred assembly is one with a high transmission loss allowing less sound to transmit through the material resulting in a greater sound level difference across the partition. The effectiveness of the transmission loss of a partition will relate to the sound isolative properties of the components used in construction. A material with higher mass is more effective at reducing sound transmission because the sound must use up more energy to pass through that material which results in higher attenuation. For example, a concrete block wall will have a higher transmission loss than a standard stud wall. Also, adding fiberglass or other absorptive material between wood and metal studs will increase the transmission loss of the wall compared to a typical stud wall with no fill.

Section 3. Common Sources of Noise

Alongside noise issues due to room acoustic issues within multipurpose rooms, there are other common sources of noise from within the room and surrounding areas. Noise fits into two categories: structure-borne noise, and airborne noise. Unfortunately, analysis is complicated by the fact that a source can produce noise that falls under both categories.

Impacts to a structural element including walls, floors, and the roof result in vibrations that cause structure-borne noise. The structural element that is impacted will amplify the noise so that a small impact may result in a large noise. This amplification will depend on the ease with which the vibrations can be transferred along the structural path. Noise and vibration that occur along the structure have become a more prevalent issue in many buildings today because of lightweight building materials, which allow a higher transmission of noise along them. These materials do not reduce as much low frequency sound due to the lack of mass. A building structural system consists of joists, beams, studs, columns, and other members that are connected. Due to the support required, the members are typically connected in a rigid manner such as with bolts and welds for steel structures. These rigid connections provide pathways for sound energy to transfer (Long, 143). For example, the noise and vibrations of a floor in a remote area of the building can be transferred along the structural members and connections into more sound-sensitive areas. The noise and vibration transferred along structural members can be a source of great annoyance for occupants and can be a problem for equipment that is sensitive to vibrations. Also, the mass and stiffness of the structure will contribute to the amount of sound that is allowed to transfer through the structural member. For example, a member that is stiffer and has more mass will not allow vibrations to transfer as easily and will thereby act as a sound dampener (Mehta, Johnson, Rocafort, 144). In general, structural member noise is created by sources such as vibrations from mechanical systems.

Mechanical equipment is usually the prime source of structural noise in a multipurpose space. This is because equipment such as pumps and motors create vibrations that can be transmitted along structural members into surrounding spaces. A mechanical system also creates

other pathways beyond the standard structural members for noise and vibrations to be transmitted. These structure-borne pathways include ductwork, piping, and anchors that support these materials and connect them to the structure. Ductwork and piping are designed as rigid and leak-free members to maintain efficiency. They must penetrate walls, floors, and ceilings along their path to reach the desired location and are typically sealed to maintain proper fire barriers, which results in a rigid connection to the structural member and a perfect pathway for vibrations and noise transfer

Along with noise created and transmitted by the structure, airborne noise can also be created by sources both within the space and from outside sources. The difference between structure-borne and airborne noise is the path from the source to the listener. Airborne noises reach the listener through the pathway of air as opposed to the structural pathways and vibrations. Airborne noise can be generally classified into two categories: noise generated within the space and noise generated outside the space. The source of the noise can usually be found easily when it is occurring within the space concerned. Unfortunately, noise sources generated in outside spaces may be more difficult to find because the pathways into the space can be complex (Mehta, Johnson, Rocafort, 159). Transmission paths can include voids in and around the structure, through the openings created by ductwork, and through partitions such as walls, floors, and ceilings that create indirect sound paths known as flanking paths. Mechanical and electrical systems can also be a chief source of airborne noise. Noise can be created by nearly all pieces of equipment including fans, pumps, ductwork, generators, lights, and other components such as dampers and diffusers.

Section 4. Noise Solutions

Identifying the source of noise that is creating the problem for the multipurpose room will lead towards a decision of what is the best solution to attenuate the noise. There are multiple solutions available for each source of noise which consequently have trade-offs associated each. There are times when the most effective solution is not feasible due to budget constraints and an alternative must be used instead. However, most problems will accommodate several solutions to consider for each issue and this section cannot cover each one; instead, this section strives to give a general idea of the availability and result of some of the most commonly used solutions.

The most common problem experienced in multipurpose rooms is that the space is too reverberant, so the room experiences long reflections resulting in difficulty comprehending speech and music and a perceived loud space. This usually occurs because the composition of the surfaces in the room is primarily hard and consists of reflective materials with minimal absorption. A resolution for this problem is to install sound absorbing materials in multipurpose rooms experiencing reverberation times that exceed the recommended times of between 1.0 and 1.5 seconds. Sound absorbers improve the room acoustics by eliminating sound reflections. Cost will vary depending on the type of absorber used and the amount needed, but prices are typically based on a dollar per square foot value. Table 2 lists some approximate costs for typical room acoustical treatments including different types of absorbers and diffusers. These prices will change with the product that is used and the level of absorption of the material.

The recommended solution to resolve long reflections is to use enough absorptive materials that achieve the target reverberation time for the space. Using the Sabine Equation, shown in Equation 2, the space reverberation time can be calculated to determine what effect different materials will have in the space. Arguably, aesthetics will be the primary trade-off when using different absorptive materials. This means a less expensive product will not look as

polished as a more expensive product, although most manufacturers will offer different colors and patterns for fabric-covered treatments that can be adjusted to the owner’s preference.

Table 2. Approximate Installed Cost for Typical Acoustical Treatments

Acoustical Treatment	Noise Reduction Coefficient (NRC)		
	0.45-0.65	0.65-0.85	0.85-0.95
Wall Treatments		\$3 - \$6/SF	\$6 - \$15/SF
Lay-in Panels	\$0.5 - \$1/SF	\$1.50 - \$2/SF	\$2 - \$4/SF
Suspended Treatments			
<i>Baffles</i>	--	--	\$5 - \$7/SF
<i>Clouds</i>	--	--	\$6/SF
Roof Deck	--	--	\$6/SF
Spray On Treatments	--	--	\$2.50 - \$3.50/SF
Drapery			\$20 - \$25/SF
Diffusing Shapes			\$250 each

Structure-borne noise is related to noise and vibrations that transfer along the structure. In such cases, mass of the structure is the most important factor in reducing structure-borne noise. A lightweight structure will be more susceptible to vibrations and will therefore transmit a greater amount throughout the building. In contrast, a structural member with a higher mass will transmit less because it will be harder to vibrate the material. In the end, two basic strategies can be adopted to reduce the structure-borne noise in a building. First, a soft floor covering such as carpet, cork, or rubber should be used where possible to reduce footfall and impact noise before it can be transmitted into the structure (Doelle, 167). Of these three materials, carpet will provide the best improvement in impact noise reduction. Furthermore, each one of these materials is economical to install.

The second strategy to reduce structure-borne noise is to provide discontinuities along the structure. One method of achieving this includes using a floating floor which uses vibration isolators to decouple the floor from the surrounding structure. Another method includes isolating vibrating equipment from the structure. Disconnecting the structure using flexible connections eliminates the path for vibrations to move along (Mehta, Johnson, Rocafort, 146). Providing discontinuities along the structure is a more expensive option due to the intensive construction labor and materials that can be involved depending on the project size. As a result, this option may only be employed within a limited area that is particularly sensitive to vibrations or in areas closest to the source to dampen vibrations as quickly as possible. For example, a room that is located below another space that is creating vibrations may experience problems as the vibrations travel through the floor and into the ceiling of the lower space. Isolating the ceiling from the structure above using isolation hangers can prevent some of the vibrations from being transmitted along the structure.

In reducing airborne noise, the mass of a partition will be the most important factor in the reduction noise from outside sources. Sound loss can also be experienced due to sound leaks along the perimeter of a space. A primary location for sound leaks, both doors and windows will experience some level of sound leakage around the perimeter of their frame; however, all penetrations through partitions must be sealed with an acoustical sealant to prevent as little flanking noise as possible. An acoustical sealant is a non-hardening, non-drying sealant that

remains flexible to last longer and avoid cracking, which will eliminate the seal's effectiveness against preventing sound transmission. Many manufacturers provide acoustical sealants, such as Owens Corning, USG Corporation, Pecora Corporation, and Tremco. Older doors and windows should be resealed because their seals will degrade naturally over time due to wear. This is a relatively inexpensive maintenance option that will maintain the acoustic performance of a space.

Solutions to mechanical system noise must counteract or correct many issues that could be occurring at the mechanical source itself or along the transmission path due to air or water flow. Noise issues can be a structure-borne problem, airborne problem, or a combination of both. Regardless of the type of noise being generated from the mechanical system, the mechanical room is the first location that should be treated to control noise from equipment. Specifically, surfaces can be treated with acoustical material such as wall mounted absorptive panels to attenuate as much sound as possible before it can leave the room. Also, the mass of the walls, floor, and ceiling can be increased to reduce the amount of sound that can be transmitted through. For a piece of equipment that is creating noise, an economical solution is to construct an enclosure for the equipment. The closer to the noise source that the attenuation occurs will result in a higher reduction in sound loss.

The second location for mechanical equipment noise reduction is along the noise path. These paths include the structure where vibration energy can be transmitted, ductwork, piping, and connections. If left un-isolated, these vibrations will pass on into the space and create noise. Fortunately, there are methods to isolate equipment that will reduce the potential to transfer vibrations. The most economical method of isolation is using vibration isolators. Isolators are listed by their range of rated loads and deflections associated with those loads. The exact type of isolator used will be dependent on the type of equipment used and will vary from project to project. An acoustical consultant or vibration isolator manufacturer can be utilized to select the optimal isolator for each project. Flexible connections should be used at all vibrating equipment. This will also reduce the strain on and rigidity to the equipment which allows the isolators located underneath the equipment to operate as designed.

Section 5. Case Study

An existing church multipurpose room was experiencing noise issues within the space that were noticeable after construction. The noise minimized the space functionality by making it difficult for the occupants to comprehend speech and music. The owner desired to resolve as many issues as possible while staying within the available budget. The space is used for speech, music, and physical activities. The primary floor area is 69'L x 50'W x 37'H with a total room volume of 114,000 ft³. The room was experiencing several noise issues that caused occupants difficulty in comprehending speech and created an annoyance. The noise issues included excessive reflections, flutter echo, and mechanical system noise from the air handling unit. The room was analyzed under the existing conditions to determine the reverberation time of space and the ambient noise levels. A reverberation time calculation was run using the Sabine Equation and through field testing and was determined to be approximately 3.6 seconds. A more ideal reverberation time would be between 1.0-1.5 seconds; however, at a minimum "the reverberation time should not exceed about 1.6 seconds at mid frequencies for a space with the volume of this multipurpose room" to maintain speech intelligibility and lively music environment (Coffeen).

During the field testing, the room was also measured to determine the ambient sound level within the room. Without the air handling unit operating, the ambient noise level was measured as having a Noise Criterion rating of 30. With the air handling system operating, the

room ambient noise was measured as having a Noise Criterion rating of 49 (Coffeen). Comparing the rating NC-30 to the rating with the air handling unit of NC-49 shows that the air handling unit is causing a significant increase in noise for the space. A rating of NC-49 also exceeds the recommended levels for a multipurpose room of NC-30 to NC-35.

After weighing the benefits and costs of the recommended solutions for the church multipurpose room, the owner had to choose what solutions would meet the needs of the space while maintaining other needs including cost and ease of implementation for an existing facility. Unfortunately, the mechanical noise problem was not completely resolved for this space because the cost of eliminating all of the noise from the air handling unit was not within the owner's budget. However, the fan speed of the air handling unit was reduced to try and eliminate as much noise as possible without compromising the ability of the unit to condition the space. Also, three other solutions were employed to directly reduce the other two noise issues concerning reflections and echo.

The first solution chosen was to place 4'-0" x 5'-0" absorptive panels comprised of Tectum suspended from the wood ceiling. This worked to maintain the aesthetic appeal of the wood ceiling, but provide absorption at that location. Suspending the panels provided an air space behind the panel which allows it to be a better broadband absorber, meaning it can absorb a wider range of sound frequencies. Due to budget constraints, panels could not be placed in every location of the ceiling, therefore, the acoustician determined that the center bays were more critical to cover with the panels than the exterior bays without compromising the effectiveness too much.

The second solution to reduce flutter echo and long reflections was to place additional fabric covered porous absorptive panels on the walls. These panels are 2" thick fiberglass board and are surface mounted. The acoustician recommended that a larger square foot area of the walls be covered with the panels for the best reduction in excessive reflections; however, for economic reasons less was used on two walls. In the future, if the budget allows, additional panels can be installed in other locations.

The third solution chosen to improve the space was to use seating that has additional padding and provides more absorption in the room. These chairs are moveable, but will remain in the space for a majority of the functions.

With all three of these solutions in place, the space was re-evaluated to determine how effective the changes were. Consequently, the Sabine Equation was utilized to determine the approximate reverberation time for the space under the new conditions which was found to be 1.5 seconds and a field measurement found the time to be 1.6 seconds. This time is much more appropriate for this type and volume of space such that speech will be more intelligible and music will be lively and enjoyable for the occupants. The ambient noise level of the room after the changes was also field-measured. Reducing the fan speed created a slower air velocity in the system and reduced the noise that the higher velocity was generating. The new field-measured level has a Noise Criterion rating of 37. This NC rating is still slightly higher than the recommended maximum of NC-35, but it is enough of a change to be considered acceptable and will decrease the annoyance for the occupants. Final analysis of the space proves that all the changes made have reduced excessive reflections and noise, making the environment acoustically pleasing. Overall, the owner was pleased with the results, shown as the before and after images in Figure 4, and is now able to fully utilize the space as it was intended.



Figure 4. Before and After

Section 6. Conclusion

Likely, a large amount of money will be spent to build a multipurpose room that fits the owner's needs. Furthermore, for a multipurpose room, the function of the space may change so that a space used as a banquet hall in the morning, may serve as an auditorium for a concert in the evening. As a result, the design of the room acoustics becomes more challenging because an acoustical balance must be found to maintain desired characteristics such as speech intelligibility or musical richness. To achieve this balance, the multipurpose room designer must understand how sound works. Sound in an enclosed area is continuously moving and interacting with the surfaces of the space; therefore, the acoustic response of the space can be determined by analyzing the composition of each surface. First, absorbing materials will absorb incident sound thereby reducing the amount of sound reflection back to the space. Second, reflective materials have minimal absorption characteristics and will promote more reflection back to the space. Utilizing a combination of these materials, the room can be acoustically designed to maximize the function of the space.

Every building will experience some level of noise. Noise is an annoyance to occupants; however noise can also become more serious as it affects occupant health and productivity. Since noise is created by several sources including airborne and structure-borne noise, its control to an acceptable level based on the Noise Criterion or Room Criterion method must occur at the source or along its pathways. Each noise problem may have multiple potential solutions. However, there are trade-offs that must be considered when choosing one solution over another. Most often, a budget will establish which solutions are economically feasible.

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Six Surface Steady State Acoustic Radiosity

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ABSTRACT

The prediction of sound levels in rooms is usually accomplished in one of two ways: using simple diffuse field theory or full computer modeling using an acoustic CAD program. The simple diffuse field theory can be easily calculated by hand or on a spreadsheet but is limited in its accuracy. Acoustic CAD programs, usually based on ray tracing, beam tracing, or image methods, can be far more accurate, but require expensive and proprietary software and generating the computer models for use by the software can be time consuming. A method of sound level prediction with more accuracy than simple diffuse theory but that can still be implemented by hand or by spreadsheet would be very beneficial to consultants and engineers. In this paper, we present a simplified approximation to diffuse steady state radiosity for the prediction of sound pressure levels in six sided rectangular rooms. The steady state diffuse radiosity method is reduced to a six surface problem and a closed form solution for computing the approximate power on the entire enclosure surface is presented. Closed form expressions for the form factors between all surfaces and configuration factors to and from all surfaces are also presented. The simplified method is used to predict sound levels in an empty classroom.

Keywords: room acoustics, radiosity, radiative transfer, sound level prediction

INTRODUCTION

The prediction of steady state sound levels in rooms is one of the most basic, and necessary calculations in architectural acoustics. There is a need of a practical, but accurate method for predicting steady state sound levels in many types of enclosed spaces.

A variety of methods are currently used for estimation of steady state sound levels in enclosures including diffuse field theory, wave theory, and ray tracing, each with a number of advantages and disadvantages. Another method of computation is the bulk energy transfer method called radiosity, also known as radiative transfer. Radiosity is somewhat similar to ray tracing involves bulk energy trans-

fer assuming acoustic energy travels in a straight line, i.e. ignoring diffraction and refraction.

Radiosity methods were first developed for use in illumination, computer graphics), and thermal engineering (Siegel and Howell 2002).

In acoustics, radiosity methods were first used by Kuttruff (1971) but several researchers including Cianfrini et al. (1998), Kang (1999), Nosal et al. (2004), and Siltanen et al. (2007), have adapted the radiosity to both steady state and time dependent room acoustic computations.

THEORY

Diffuse Steady State Radiosity

To begin discussing the six-sided simplification to diffuse radiosity, a very short derivation of basic diffuse radiosity is required. The reader is referred to the works of Nosal et al. (2004) and Siltanen et al. (2007) for a more complete derivation.

One important assumption in this formulation of the problem is that the room enclosure surfaces can be treated as a perfectly diffuse reflectors. If the reflector is perfectly diffuse, the directivity of the reflected sound power is independent of the direction of incidence and instead follow Lambert's cosine law. Hodgson (1991, Hodgson (1996) has shown that this approximation is surprisingly accurate, even when the room enclosure is made of many large flat surfaces.

The acoustic radiosity is defined as the total sound power per unit area leaving a point of the enclosure surface and denoted by M . The radiosity differs from standard acoustic intensity, denoted by I in that the intensity is the total power per unit area traveling in a certain direction. With the assumption of a purely diffuse reflector, the radiosity leaving the surface patch of area dS will follow the cosine law and the acoustic intensity a distance r away and at an angle of θ from the normal of the surface can be written as

$$I(r, \theta) = M \frac{\cos\theta}{\pi r^2} dS. \quad (1)$$

If the room enclosure is broken into a finite number of patches, the average acoustic radiosity leaving the i^{th} patch, M_i , can be written as

$$M_i = M_{0i} + \rho_i \sum_j M_j \frac{A_j}{A_i} F_{j \rightarrow i} \quad (2)$$

where M_{0i} is the radiosity that comes directly from sources in the room, ρ_i is the average sound power reflection coefficient over the patch, A_i is the area of the patch, and $F_{j \rightarrow i}$ is the form factor for radiation from patch j to patch i which is the fraction of radiosity leaving patch j that reaches patch i .

In the absence of diffraction and refraction (i.e. using standard geometric acoustic assumptions) and assuming the energy absorption of the medium can be

ignored, the form factor is purely a function of the patch geometry and can be written as

$$F_{i \rightarrow j} = \frac{1}{\pi A_i} \iint_{A_j} \iint_{A_i} \frac{\cos(\theta_j) \cos(\theta_i)}{r_{ij}^2} dA_i dA_j . \quad (3)$$

where r_{ij} is the distance between two points on the patches i and j as shown in Fig. 1. The form factor is completely analogous to the form factors and configuration factors used in illumination engineering, thermal engineering, and computer graphics (Siegel and Howell 2002). Absorption by the medium can be taken into account, when needed, by scaling $F_{i \rightarrow j}$ by $e^{-mr_{ij}}$ where m is the standard acoustic attenuation coefficient defined in ANSI S1.26 (ANSI 1995) and r_{ij} is the average distance between patches i and j .

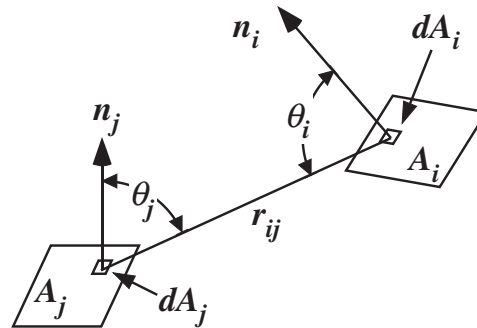


FIG. 1. Graphical description of form factor geometry. Sound power is radiated from a differential section of patch i with area A_i and travels a distance r_{ij} to a differential section of patch j with area A_j . The line between the patches is at an angle θ_i with respect to the normal on patch i at dA_i and an angle θ_j with respect to the normal of patch j at dA_j

With Eq. 3 as the definition for $F_{i \rightarrow j}$, the form factors have many special properties, one of which is an area reciprocity which states $A_i F_{i \rightarrow j} = A_j F_{j \rightarrow i}$ and a second which states $\sum_i F_{i \rightarrow j} = 1$. Using the area reciprocity relation Eq. 2 can be rewritten as

$$M_i = M_{0i} + \rho_i \sum_j M_j F_{i \rightarrow j} . \quad (4)$$

For an omnidirectional source of power W , the fraction of that power that will reach a particular patch i will be $\Omega_i/4\pi$ where Ω_i is the solid angle subtended by the patch i as seen from the location of the source. Thus, the source radiosity term M_{0i} that comes from a number of different sources can be written as

$$M_{0i} = \rho_i \sum_k W_k \frac{\Omega_{Sik}}{4\pi} \quad (5)$$

where Ω_{Sik} is the solid angle subtended by the patch i as seen from the source k .

To find the sound level at a given point in the enclosure, we must find the total mean square acoustic pressure at that point. Using reciprocity between sources and reflecting surfaces, the

Using reciprocity of radiation between points and planes, the total square pressure at the receiver point k that comes the enclosure surfaces with radiosity M_i will be

$$p_k^2 = \rho_0 c_0 \sum_i M_i \frac{\Omega_{Rki}}{2\pi} \tag{6}$$

where again Ω_{Rki} is the total solid angle subtended by the patch i as seen from the receiving point k . Notice that compared to Eq. 5, this equation only has a 2π in the denominator. That is because the planar source only radiates into half space with a solid angle of 2π while the point source radiates into full space with a solid angle of 4π .

Simplified Radiosity for a Rectangular Room

In this paper, we want to simplify Eq. ?? further by assuming the room enclosure is rectangular and consists of six uniform surfaces: four walls, a floor and a ceiling as shown in Fig. 2. The average reflection coefficient for each wall is obtained by standard area weighted averaging of the reflection coefficients of the different materials that make up the surface of the wall.

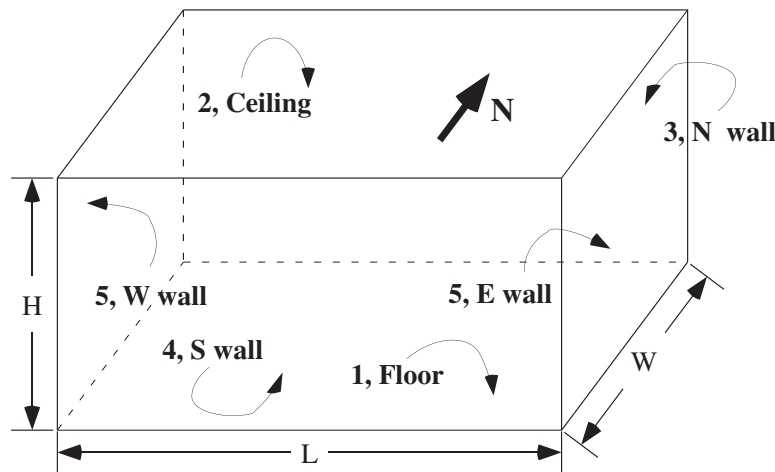


FIG. 2. Six surface room enclosure consisting of a floor (Surface 1), a ceiling (Surface 2), a North wall (Surface 3), a South wall (Surface 4), an East wall (Surface 5) and a West wall (Surface 6).

For a system with 6 surfaces, Eq. 4, 5, and 6 represents a set of 8 coupled equations. A closed form solution can be obtained by rewriting the equations in matrix form. Denoting the identity matrix as \mathbf{I} , defining the following vectors

and matrices

$$\begin{aligned}\mathbf{M} &= [M_1 \quad M_2 \quad \cdots \quad M_6]^T, \\ \mathbf{M}_s &= [M_{s1} \quad M_{s2} \quad \cdots \quad M_{s6}]^T, \\ \mathbf{R} &= \mathbf{I} [\rho_1 \quad \rho_2 \quad \cdots \quad \rho_6], \\ \mathbf{F} &= \begin{bmatrix} F_{1 \rightarrow 1} & F_{1 \rightarrow 2} & \cdots & F_{1 \rightarrow 6} \\ F_{2 \rightarrow 1} & F_{2 \rightarrow 2} & \cdots & F_{2 \rightarrow 6} \\ \vdots & \vdots & \ddots & \vdots \\ F_{6 \rightarrow 1} & F_{6 \rightarrow 2} & \cdots & F_{6 \rightarrow 6} \end{bmatrix}\end{aligned}$$

Eq. 4 can be written in matrix form as

$$\mathbf{M} = \mathbf{R} \mathbf{F} \mathbf{M} + \mathbf{M}_s \quad (7)$$

which has a closed form solution of

$$\mathbf{M} = (\mathbf{I} - \mathbf{R} \mathbf{F})^{-1} \mathbf{M}_s. \quad (8)$$

Eqs. 7 and 8 can be solved by a variety of standard methods including Gaussian elimination, LU decomposition, QR decomposition, or direct inversion; however, with the small size of the matrices involved and the fact that the matrix $(\mathbf{I} - \mathbf{R} \mathbf{F})$ is usually well conditioned, direct inversion is probably the easiest way to solve the problem.

Eq. 8 shows that the steady state radiosity on all surfaces can be directly computed from a product of the source radiosity vector and a single matrix inverse. The matrix inverse is essentially a propagation matrix which includes all the effects of interreflection of all the surfaces including surface absorption.

By defining the elements of the the $6 \times N_S$ source configuration matrix \mathbf{C}_S as $\Omega_{Sik}/4\pi$ and defining the source vector $\mathbf{W} = [W_1, W_2, \dots, W_{N_S}]^T$, Eq. 5 can be rewritten as

$$\mathbf{M}_s = \mathbf{R} \mathbf{C}_s \mathbf{W}. \quad (9)$$

By defining the elements of the $N_R \times 6$ receiver configuration matrix \mathbf{C}_R as of $\Omega_{Rki}/2\pi$ and the receiver vector $\mathbf{p}^2 = [p_1^2, p_2^2, \dots, p_{N_R}^2]^T$, Eq. 6 can be rewritten as

$$\mathbf{p}^2 = \mathbf{C}_R \mathbf{M}. \quad (10)$$

Finally, Eq. 8, Eq. 9, and Eq. 10 can be combined to write closed form solution for the mean square pressure at many locations from a multitude of point sources as

$$\mathbf{p}^2 = \mathbf{C}_R \mathbf{M} (\mathbf{I} - \mathbf{R} \mathbf{F})^{-1} \mathbf{R} \mathbf{C}_s \mathbf{W}. \quad (11)$$

Computing form factors

For this six surface enclosure there will be thirty six form factors which represent the fraction of energy leaving each surface and reaching every other surface. Because of symmetry and energy conservation, the thirty six form factors are not independent of one another. Reciprocity requires that $A_i F_{i,j} = A_j F_{j,i}$. The symmetry of rectangular room requires that $F_{1,3} = F_{2,3} = F_{1,4} = F_{2,4}$, $F_{1,5} = F_{2,5} = F_{1,6} = F_{2,6}$, and $F_{3,5} = F_{4,5} = F_{3,6} = F_{4,6}$. Since the walls are approximated as flat surfaces, no radiation leaving a surface will reach that same surface without being reflected so $F_{i,j} = 0$. Finally, energy conservation requires that $\sum_j F_{i,j} = 1$ and so $F_{1,5} = 0.5(1 - F_{1,2} - 2F_{1,3})$ and $F_{3,5} = 0.5(1 - F_{3,4} - 2F_{3,1})$. Between the reciprocity relation, the energy conservation summation rules, and the flat surface approximation the set of thirty six form factors actually reduce to just four independent form factors: $F_{1,2}$, $F_{1,3}$, $F_{3,4}$ and $F_{5,6}$.

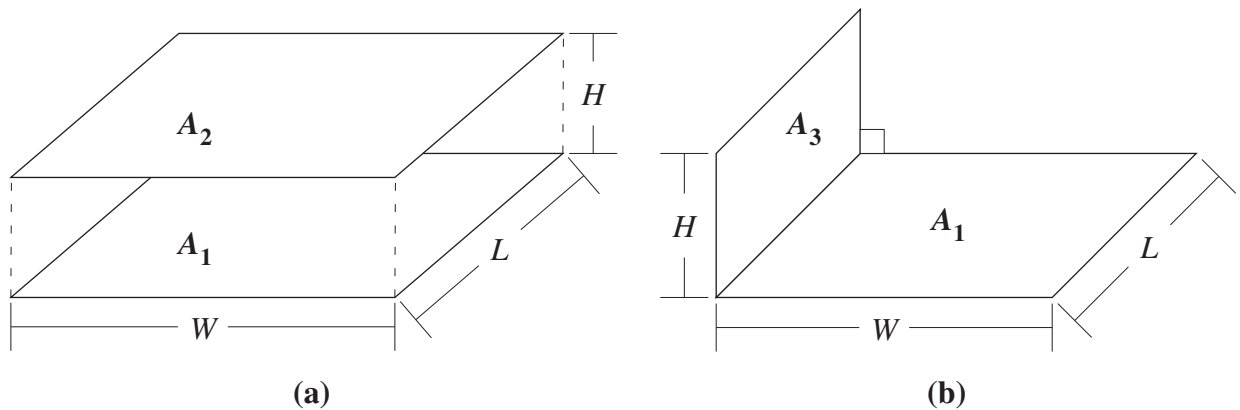


FIG. 3. Geometry to determine the form factors (a) $F_{1,2}$ between two parallel plates of side L and W separated by a distance H and (b) $F_{1,3}$ between two plates at a right angle with sides W and H and a common side of length L .

$F_{1,2}$, $F_{3,4}$, and $F_{5,6}$ are all form factors for radiation between two rectangular parallel planes. The form factor $F_{1,2}$ between two rectangular plates of length L and W separated by a distance H as shown in Fig. 3a is given by (Ref Hamilton) as

$$F_{1,2} = \frac{2}{\pi AB} \left(\ln \sqrt{\frac{XY}{1+Z}} - A \tan^{-1} A - B \tan^{-1} B + A\sqrt{Y} \tan^{-1} \frac{A}{\sqrt{Y}} + B\sqrt{X} \tan^{-1} \frac{B}{\sqrt{X}} \right) \quad (12)$$

where $A = L/H$, $B = W/H$, $X = 1 + A^2$, $Y = 1 + B^2$, and $Z = A^2 + B^2$. We can use this same formula to compute $F_{3,4}$ if we set $A = W/L$ and $B = H/L$ and also to compute $F_{5,6}$ if we set $A = H/W$ and $B = L/W$.

The form factor $F_{1,3}$ from the floor to a wall plate of width W to a plate of width H with a common joint of length L as shown in Fig. 3b is given by (Ref

Hamilton) as

$$F_{1,3} = \frac{1}{\pi B} \left(A \tan^{-1} \frac{1}{A} + B \tan^{-1} \frac{1}{B} - \sqrt{Z} \tan^{-1} \frac{1}{\sqrt{Z}} \right. \\ \left. + \frac{1}{4} \ln \left\{ \left[\frac{XY}{1+Z} \right] \left[\frac{A^2(Z+1)}{XZ} \right]^{A^2} \left[\frac{B^2(1+Z)}{YZ} \right]^{B^2} \right\} \right) \quad (13)$$

where $A = H/L$, $B = W/L$ and X, Y , and Z are defined as above in Eq. 12.

Computing solid angles between points and planes

The configuration factor matrices \mathbf{C}_S and \mathbf{C}_R both require computation of the solid angles subtended by the six surfaces as seen from the various source and receiver positions.

The solid angle from a point to a rectangle can be computed by breaking the plane into four smaller rectangles centered beneath the receiving point as shown in Fig. 4(a). The solid angles to each of the smaller rectangles are summed to get the total solid angle. The solid angle, Ω_r from a point a distance c from the corner of a rectangle of sides a and b as shown in Fig. 4(b) is (Hamilton and Morgan 1952)

$$\Omega_r = \tan^{-1} \left(\frac{AB}{\sqrt{1 + A^2 + B^2}} \right) \quad (14)$$

where $A = a/c$ and $B = b/c$. Thus the total solid angle Ω_j for a source located at a distance z away from rectangular surface of sides L and W will be $\Omega_j = \Omega_I + \Omega_{II} + \Omega_{III} + \Omega_{IV}$ where Eqn. 14 is applied to a rectangle with $A = x/z$ and $B = y/z$ for Ω_I , $A = (L-x)/z$ and $B = y/z$ for Ω_{II} , $A = x/z$ and $B = (W-y)/z$ for Ω_{III} and $A = (L-x)/z$ and $B = (W-y)/z$ for Ω_{IV} .

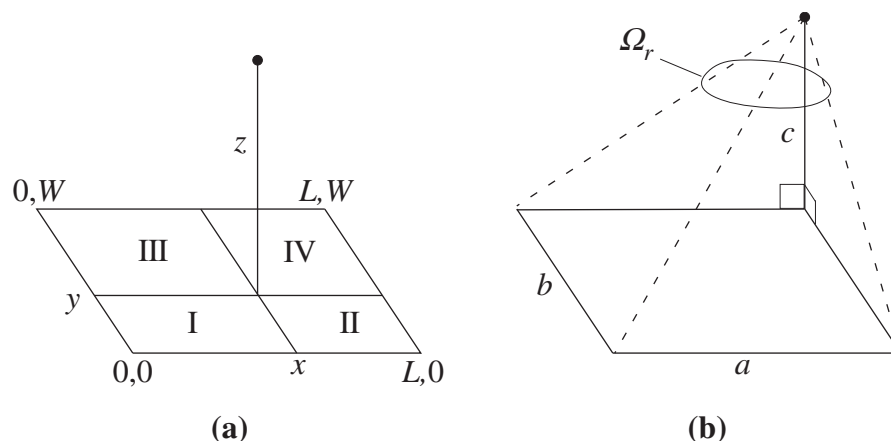


FIG. 4. Geometry for computing the solid angle of a point to a rectangle. (a) The solid angle of a general rectangle of sides L and W to a point at a position x, y , a distance z away is found by breaking it into four smaller rectangles, I to IV . (b) The geometry to compute the solid angle Ω_r from a point a distance c located over one corner of a rectangle of sides a and b .

APPLICATION

As an example application, we have computed the sound levels at several several locations in an existing classroom using the six surface radiosity method and will compare the results with computations made with the same room modeled with 31 and 126 patches and to measurements made at several locations in the room. A more complete description of the classroom and more information about the measurements themselves and a drawing of the room can be found in the recent publication by Hodgson and Nosal (2006).

The classroom is 13.7 m long, 7.8 m wide and 2.8 m high. The floor was vinyl tile with an assumed 1 kHz reflection coefficient is 0.97. The bulk of the walls were painted concrete with an assumed with an assumed 1 kHz reflection coefficient of 0.97. On the north wall is a 6.3×1.1 m blackboard and a 1.1×2.8 m window, both with an reflection coefficient of 0.99. On the west wall is a 6.0×1.1 m blackboard with a reflection coefficient of 0.99. On the south wall is a second 6.3×1.1 m blackboard. Also on the south wall there are also two 0.9×2.0 m doors with an assumed reflection coefficient of 0.93. The values used in this simulation do differ slightly from those used by Hodgson, but that is because we feel our values are more indicative of the values a consultant would assume when using the radiosity method for design purposes.

An omnidirectional source was placed at the position (2.0 m, 3.9 m, 1.3 m) and measurements were made at five locations located along the room width centerline at distances of 0.5, 1, 3, 5, and 8 m from the source. Additional computations were made at five additional randomly selected points in the room. The computations were made for the six sided approximation and for three levels of higher discretization shown in Table 1. For the six sided approximation the weighted average reflection coefficient for the north, east, and west walls was 0.98, for the south wall was 0.97, for the floor was 0.98 and for the ceiling was 0.7.

The results show that for all ten positions, the computational results using the six sided approximation is within 1.6 dB of the results obtained with 395 patches at all the positions. At the five measurement positions, the predictions are within 1 dB. One can also notice that the sound level well away from the source (when the direct sound is negligible) is not constant. There are small variations which simple diffuse field theory with a spatially constant reverberant sound field could not predict. Simple six sided radiosity

CONCLUSIONS

In this paper a reduction of the steady state diffuse radiosity method to six surface rectangular enclosures was presented. A closed form matrix solution for the mean square pressure at multiple locations from sound generated from multiple point sources was developed. The thirty six form factors that describe the radiation interaction between the six surfaces are shown to reduce to only four independent form factors. The propagation of sound from point sources to the six surfaces and from the six surfaces to a point in the room was shown to be directly related to the solid angle subtended by the surface as seen from the point. Closed

TABLE 1. Measured and computer predicted steady state sound pressure levels at ten positions in the empty classroom. Predictions were made using the six sided radiosity approximation as well as with higher levels of discretization into $N = 32$, $N = 126$, and $N = 395$ patches.

		L_p (dB)				
Number	Position [m]	Measured	Number of patches			
			6	31	126	395
1	(2.5, 3.9, 1.3)	93.5	93.3	93.6	93.9	94.0
2	(3.0, 3.9, 1.3)	89.9	89.5	90.0	90.5	90.7
3	(5.0, 3.9, 1.3)	86.8	87.0	87.6	87.5	87.5
4	(7.0, 3.9, 1.3)	86.4	86.7	86.5	86.1	86.0
5	(10.0, 3.9, 1.3)	85.6	86.6	85.3	84.6	84.5
6	(13.0,1.8,1.6)	-	86.6	85.1	84.1	84.0
7	(6.7,7.0,2.0)	-	87.7	86.7	86.1	86.0
8	(6.2,0.15,2.1)	-	86.8	87.2	86.3	86.4
9	(1.9,1.6,0.51)	-	87.4	88.2	89.0	89.0
10	(8.3,2.1,0.5)	-	86.8	85.7	85.3	85.2

form expressions for all four form factors and for the solid angle of a surface for a general point location are presented.

The reduction of the problem to a six surface rectangular enclosure and closed form expressions for the form factors and source propagation allows the computation method to be easily coded in a spreadsheet or web form, unlike the general radiosity and ray tracing methods which requires far more advanced computer programs. The implementation of the method is demonstrated by the computation of sound levels in an empty classroom. The predicted sound pressure levels for six surface radiosity are compared with a much higher order radiosity models (31, 126 and 395 surfaces) for the room and and measurements of the the sound pressure in the actual classroom. The method is shown to predict the sound pressure levels to within 1.6 dB of the 395 surface radiosity at all points in the computation and to within 1 dB of the measured value at all the measurement locations. Thus one can conclude that simple rectangular rooms like classrooms can be fairly accurately modeled using six surfaces radiosity.

ACKNOWLEDGMENTS

This work was supported by the National Science Foundation, the University of Colorado, and the Illinois Institute of Technology.

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BUILDING VENTILATION BY WIND

By Roozbeh Ghaderi¹, Member, ASCE and Mohammed Javad Khoshharf²

ABSTRACT: In this article, building air conditioning using wind as a natural factor, utilizing Iranian Traditional Architecture aimed at a reduction in energy use is discussed. One way to achieve this is using Baadgirs (wind catchers). In wind catchers, the clean and sustained wind energy is used such that wind enters the wind catcher and then into the residential space. If by using secondary means such as water one could wet and cool the wind, a more desired air would enter the building. This article purports to introduce two new types of wind catchers in an up-to-date form, to be used in modern buildings.

INTRODUCTION

Man's environmental conditions house got direct influence or his moods, physical condition, performance and all other aspects of his life. Since the major part of present human's life passes indoors, creating favorable environmental conditions in the building, either the workplace or the house is of great importance and most of all is ventilation for the inhabitants of the building, which should be done according to their type of activity. Ventilation could be called the main way of oxygen origination that in case of its shortage evens the most beautiful and expensive buildings will be impossible to live in. In air conditioning industry the goal is providing the relaxation conditions in

the building. Nowadays ventilation in modern buildings is impossible without advanced installations systems and this involves great energy consumption. Considering the lack of fossil fuels and the great price of energy especially in recent years, the use of natural element in ventilation is getting common between most of the designers.

NARURAL VENTILATION MECHANISM

Air movement in a building is caused by the difference in air pressure in its outer surface. This difference in pressure may be the result of two factors: first the difference between the inner & outer temperatures & second wind blow.

The pressure which is caused by the difference in the temperatures is usually very low in all cases.

So wind blow is considered the effective & major factor in air movement in the building. The pressure difference which is caused this way in a building's wall surfaces can be brought into use for creating the natural ventilation & wind flow in the building.

The most suitable ventilation is provided when the Wind blows obliquely to the window surface.

Iranian architects & engineers have been using wind flow & temperature variations during day and night through a year & created masterpieces that in so warm and desert like Iran conditions provide thermal relaxation without any energy consumption. One of these masterpieces is the design & construction of windcatchers.

WIND CATCHERS

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Wind catchers are towers that were built in Iran desert areas centuries ago in order to transfer outer air into the inhabitable space and help to maintain thermal comfort in summers. These towers are different in height and section, the shortest wind catchers has a height equal to 2 meters from the roof or about 5 meters from the yard in vicinity and the tallest which is ever made in Iran is 30 meters high from the yard surface. The wind catcher's most common height is about 8 meters.

Wind catchers' section varies widely depending on many items and their dimensions are also various. The most common section is rectangular shape, while square and hexagonal shape is also used.

WIND CATCHERS TYPES

Based on their appearances wind-catchers are divided into six groups. Which are named by the number of their entrance flanks.

In some types under the wind-catcher's channel a relatively big and beautiful basin used to be built and the dry and dusty air used to get so favorable in summer's warmth after contacting the water and attracting cool moisture and leaving the dust behind. In these regions where it hasn't been possible to make basins on the ground floor, they used to run the aqueduct's (Qanat's) water free on the ground and the tail of wind-catcher's channel used to stretch over this water flow. These spaces have been used as the living room by the houses inhabitants in the summer afternoons.

HOW THE TRADITIONAL WIND CATCHER OPERATES

Traditional wind catchers operate by two methods: using the wind blow and without it. (Figure-1)

Operation using the wind blow

When the wind blows past the wind catcher and the building connected to it, a pressure, called "wind pressure" is generated in all vents of the wind catcher and at different surfaces of the building.

This pressure can be obtained from the formula:

$$P = C_p \cdot \frac{1}{2} \cdot \rho v^2 \quad (1)$$

Where P and V are the wind pressure and velocity, respectively, P is the air density and C_p is a factor, called the wind pressure factor. The factor C_p can be obtained by testing the building and the wind catcher models in boundary layer wind Funnels. The following table shows wind pressure factors at different angles of wind blow relative to the wind catcher (table-1).

Generally, when the wind blows perpendicular to the wind catcher vent, C_p at the vent is positive and at other vents above the wind catcher, it is negative. At the building doors and windows, if these openings are opened upwind, C_p is positive for them and if they are downwind or parallel to the wind is in the wind shade of the opposite well, C_p is negative for them. Therefore, there is a pressure difference between the opening(s) upwind to the wind catcher and any other opening of the wind catcher and/or those of the residential space doors and windows according to the formula:

$$\Delta P_2 = P_i - P_j = (C_{pi} - C_{pj}) = \frac{1}{2} \rho V^2 \quad (2)$$

This pressure difference causes, the flow of a given amount of air through the wind catcher by the formula:

$$\dot{V}_{ij} = \frac{\Delta P_{ij}}{R_{ij}} \quad (3)$$

In the above formula, V denotes the volumetric air flow, ΔP denotes the pressure difference, R is the flow resistance, i and j denote i and j openings and i j denotes the air path from i to j.

With the pressure difference ΔP_{ij} between openings i and j, and depending on the resistance against it, an air flow is established between i and j, that is, air enters the opening i at the wind catcher, which is upwind, and exits the building through opening j.

In view of the fact that upwind openings or openings parallel to the wind at the wind catcher bear negative wind pressure factors, of the wind entering the wind catcher, a part exits from these openings and the remainder enters the rooms or the basement. In making traditional wind Catchers, the designers by designing a wall or septum in the middle of wind catcher, have tried to increase the air resistance R_{ij} in the path of the air exiting from openings above the wind catcher, so as to reduce this air loss and by reducing the air flow resistance from the wind catcher to the residential space, more air flows through that space.

One can provide a conduit under the courtyard and the residential building and under small gardens, which are always kept wet by irrigation of flowers and plants in the garden and penetration of water into them. Therefore, the air passing through the wind catcher, by flowing through this wet, underground conduit, gets cooled

significantly and by an evaporative any and then enters the residential space. One can use a water jet to cool down the air entering the residential space, too.

If groundwater flows beneath the residential space, one can design the wind catcher such that it is connected to this cool groundwater. (Figure-2)

THE WIND CATCHER OPERATION WHEN NO WIND BLOWS

During the day, the wind catcher gets heated well with a heat exchange with the outer air and receiving the sunlight. In desert areas, the air temperature at night may get about 20°C lower than that in the day. The air in the wind catcher, gets warmer by a heat exchange with the wind catcher's body, its density is reduced, and the so called "gets lighter". It is drawn up by the buoyant force. In this state, like a chimney, the wind catcher makes a suction, this suction or pressure difference between the outer air and the warm air inside the wind catcher causes the outer air enter the wind catcher through doors and windows and flows upwards. This way, when the temperature inside the wind catcher is higher than that in the outer air, a mild flow of air is established from outside, the courtyard to the room or the building and then into the wind catcher. This natural air flow continues until the wind catcher temperature reaches that of the air outside.

If the outer air has a temperature higher than that at the inner surfaces of the wind catcher, it is cooled down by a heat exchange and its density is increased, the so called, "gets heavy"

and a condition arises which is reverse to that of chimney. Therefore, the air enters the wind catcher through upper openings, gets cooled and then enters the room or the building.

Using a flow lattice, one can specify the air flow in any path. The air passing through a conduit has heat exchange with surfaces around it and its temperature varies. Therefore, it has a different temperature when entering a room or the building. The temperature of the entering air could be found using a thermal lattice.

In this study, a wind catcher with the following dimensions is considered. The height of upper wind catcher openings is 0.5m. and the total wind catcher height above the courtyard is 8m. (Figure-3)

The above Fig. shows a flow lattice for a traditional wind catcher. Assuming a wind flow at a 45° angle, the wind pressure factor at western W and northern N openings is positive, at eastern E and southern S openings it is negative and at doors and windows D and E. it is negative. Therefore, the air enters through western and northern openings. A part of it exits through eastern and southern openings above the wind catcher and the remainder enters the room through opening S and at last exits through opening D and E.

The amount of the air flowing in any path is found from the formula (3).

Therefore, by lowering or raising the flow resistance in any path of air, one can lower or raise the air flow in it. The presence of a septum in the wind catcher is for increasing flow resistance to prevent from the air escaping from downwind openings which have a negative wind pressure.

By writing down the above formula for any path and given the wind

pressure factors and, consequently, the wind pressure at different points, and computing the air pressure loss in any path and using the continuity equation or the mass conservation law, we have;

$$\sum \rho_i V_{ij} = 0 \quad (4)$$

which denotes the equality of the total mass of the air entering the wind catcher from different openings to the total mass of the air exiting from other openings of the wind catcher and opening surfaces. Using the above, one can find the air flow in any path.

Air flow resistances R_{ij} in the above formula (3) or pressure losses in the air flow path are functions of Reynolds' number, which, in addition to dependence on the air velocity, depends on air properties as well, which itself is a function of the air temperature. Therefore, for a more accurate computation of the pressure losses and consequently, a more precise determination of the air entering through the wind catcher to the building, it is necessary to find the air temperature at different points of the wind catcher.

Given the wind blowing direction and velocity, we can find the air flow direction in different ducts, southern, western, northern and eastern, of the wind catcher and given dimensions of different parts of the wind catcher and writing down equations for heat transfer and parts of the wind catcher and by writing down the equations of heat transfer and flow, we could find the air velocity and temperature at any part. The above computations could be carried out by developing a computer program and giving the data on the wind catcher dimensions, properties of the materials being used in making the

wind catcher, the wind velocity and direction, the maximum and the minimum temperatures of the outer air, and the air purity coefficient.

The findings highlight an important point in choosing dimensions of wind catcher openings relative to the direction of the blow of the prevailing wind, that is, one can design dimensions of the wind catcher openings relative to the courtyard such that with the blow of the prevailing wind, instead of the outer warm air entering the room, the courtyard air, which is cooler than the outside air for the flowers, water pond and numerous trees being there, enters the room, the building and/or the residential space, and is transferred upwards through the wind catcher.

Disadvantages and limitations of traditional wind catchers and new designs of wind catcher

1. Dust, insects and possibly small birds finding their way into the building
2. A part of air entering the wind catcher, exit from other openings and never enters the building. Of course, this is the case for wind catchers that use a single opening for entrance of wind.
3. The amount of cold that could be stored in the mass of the wind catcher structure is limited since the mass and specific heat of the material being used in the wind catcher are relatively low.
4. The efficiency of traditional wind catcher in the absence of wind is very low.
5. Due to the wet surfaces being at the lowest part of the wind catcher, the effective height (the height at which air density changes) is very low (with

a maximum of 1m). Therefore, the air buoyancy property is not being used optimally in the wind catcher column. To correct those disadvantages and limitations, one can provide the following designs:

WIND CATCHER WITH A WETTING COLUMN

The advantages of this design are:

1. Dampers are designed for the wind catcher head that can conduct wind into the wind catcher column from any direction and prevent from air discharge or the so called, "air loss" through other openings
2. An increase in the heat transfer area. A terracotta lattice is being used in the wind catcher column that increases the heat transfer area between the air and the wind catcher. This lattice is being used to generate wet surfaces as well.
3. The air cooling potential is being used very well.

The above items are explained in more detail below:

THE WIND CATCHER HEAD

For the areas where wind blows in different directions round the clock, wind catchers with numerous openings in different directions are used to transfer the air. One can use one-way dampers to prevent from exit of the air. When the wind pressure factor C_p in an opening is positive, then at the other openings where wind pressure factor is negative, dampers are closed and prevent from air to exit.

For that to take place, one can use electronic sensors that make possible opening and closing of dampers according to the wind direction.

But this system is costly. However, it is economically more feasible than the traditional type, if its efficiency is improved.

Instead of the damper, one can use a screen. When the wind velocity is zero or the wind pressure factor is negative, that is, the opening is downwind or parallel to the wind direction, dampers or screens remain closed. Also, these dampers or screens prevent from insects or small birds entering the wind catcher. Also, by providing a filter in the entrance duct of the wind catcher, one can achieve a filtered air. (Figure-4)

THE WIND CATCHER COLUMN

The convection heat transfer coefficient between the air and materials being used for the storage of the cooling energy in the wind catcher is relatively small. Therefore, for the transfer of a given amount of heat, the heat transfer area must be increased and this must be such that the air pressure loss in the wind catcher does not exceed a permissible level. For that purpose, the wind catcher column could be made of a terracotta lattice; the cross section of any of its ducts is in the form of a square. The thickness of the walls of this lattice is taken as 10mm, or less or more. If the cross sectional area of the wind catcher is 1 square meter, the dimension of its terracotta ducts is 10cm, and the thickness of its walls is 1cm., the heat transfer area, the mass and the heat capacity for 50 pieces of these terracotta being mounted in the wind catcher are 36 square meters, 306 Kg. and 256 KJ/Kg for 1m height of the wind catcher.

Therefore, in one cubic meter of the wind catcher, about 306 Kg terracotta for storage of the cooling energy and 36 square meters area for heat transfer are available and, compared with ordinary wind catcher, their heat transfer area with the same amount of material used, is 5-10 fold.

These terracotta lattices could be prepared with a height of 50 cm in brick-making factories and mount in-situ inside the wind catcher column.

EVAPORATIVE COOLING IN THE WETTING COLUMN

The terracotta lattice being used for an increase in heat transfer area and storing the cooling energy, is wetted by wet water jets being mounted on top of the column.

Water dripping is such that the surfaces are wetted uniformly. Excess water gets its way downward to the wind catcher bottom and is collected in a small bath there. This water is pumped through a pump to the column top to be reused there. Using this terracotta lattice, one can use well the air cooling potential caused by water evaporation in it. Due to a large contact area between the air and the terracotta lattice, the time required for water evaporation is short and thus, one can have a much lower temperature at the end of the wind catcher column and, in desired conditions, it may even reach the wet temperature of the ambient. Along the path of the column where water present in the air is evaporated, the air density and relative humidity both increase. Of course, the amount of water evaporation at the top of the column is more than that at the bottom, there is a pressure difference

of $\Delta\rho gh_e$ between the air outside and that inside the column. Therefore, in wind catchers with wetting column, in addition to air temperature lowering, we will have a useful air pressure difference as well, caused by the buoyancy force, which helps the pressure difference caused by wind blowing.

The Dust-containing air loses some of its suspended particles with passing by a wet surface and, consequently, a cleaner air enters the building.

WIND CATCHER WITH WETTING SURFACES

The structure of these wind catchers is similar to that in traditional ones, except that in openings on the top of the wind catcher, wetting pads are mounted. The pads are wetted with a water pump, the air passing through the pads is cooled by evaporation and its specific gravity or density increases. The density difference between the air inside the wind catcher and that outside it produces a pressure difference equal to:

$$\Delta P = gh_e \Delta \rho \tag{5}$$

In this formula, $\Delta\rho$ denotes the density difference, h_e denotes the effective height of the wind catcher and g is the gravitational acceleration. This pressure difference builds up an air flow from outside, through the pads, into the wind catcher and then into the room, the building or the wind catcher.

COMPARISON OF THE PERFORMANCE OF TRADITIONAL WIND CATCHERS WITH THOSE EQUIPPED WITH WETTING

COLUMN AND WETTING PADS

These wind catchers are quite similar as regards dimensions and materials used, but differ from one another as regards temperature, mass flow and the relative humidity of the air entering the building. A comparison of wind catchers as regards the temperature of the air entering the building is so: In different wind velocities and directions, the lowest temperature of the air entering the building belongs to wind catchers with wetting column caused by an increase in the heat transfer and mass transfer area in that type of wind catcher. Also, in the wind catcher with wetting column, in contrast to the other two types, air temperature lowers with height and for the zero velocity after a height of 5m, the air temperature is constant.

A COMPARISON OF WIND CATCHERS REGARDING THE MASS FLOW OF THE AIR ENTERING THE BUILDING

In zero wind velocity, the maximum mass flow of the air entering the building belongs to the wind catcher with wetting pads.

At a wind velocity of 5m/s, the maximum mass flow of the air entering the building up to a height of 9m belongs to the wind catcher with a wetting column. Of course, at higher heights, the wind catcher with wetting pads direct more air flow into the building. For a wind velocity of 10m/s, the maximum air flow entering the building at all heights belongs to the wind catcher with a wetting column.

WIND CATCHER CONTROL

The temperature of the air exiting from wind catchers with wetting column and wetting pad could be controlled with a thermostat and turning on/off of a water pump wetting the wind catcher column or pads. The mass air flow could be controlled by opening and closing of a vent mounted at the bottom of the wind catcher, conducting the air exiting from the wind catcher to the building.

In winter, these vents and openings on the top of the wind catcher should be kept closed well to prevent from the cold air entering the building through wind catchers.

GENERAL CONCLUSION

Wind catchers are structures that can direct the outside air into the building without using electrical energy. In places where water is scarce, traditional wind catchers can be used. Of course, new designs, that is, wind catchers with wetting columns and wind catchers with wetting pads could supply the building with a much cooler and with more mass flow of air than traditional wind catchers.

In places where the wind direction varies round the clock, and the wind

velocity is relatively good, (at least 3 m/s), wind catchers with wetting column are efficient, while in places where the wind velocity is much low (about zero), using wind catchers with wetting surfaces is more appropriate.

In these places, one can mount a pump at the entrance to modern wind catchers to produce suction and thus correct the limited wind blow to a large degree.

In places where there is no limitation for increasing the wind catcher height, and the wind velocity variation with height is more than other places, one can use two – level wind catchers with changing in dimensions.

Also, to increase cooling power, one can use metallic pipes conducting cold water in the inner body of the wind catcher.

It is noteworthy that, for increasing the heating efficiency of this system even in the cold season; one can pass warm water through these pipes as a suggestion worth evaluation.

The above all highlight the feasibility of a modern wind catcher in low-rise single family buildings.

Of course, it may be feasible in high-rise buildings as well, using advanced installations.

Tables & Figures

Table-1: Wind pressure co-efficient (c_p) in wind-catcher's upper entrances of (1st to 4th levels) & doors & windows of the building by the side of the wind-catcher (5th to 7th levels)

Wind angle (α)													location	Region type
180	165	150	135	120	105	90	75	60	45	30	15	0		
-0.39	-0.42	-0.52	-0.61	-0.65	-0.68	-0.74	-0.49	0.03	0.41	0.70	0.91	0.86	1	The suburb area
-0.61	-0.55	-0.49	-0.46	-0.42	-0.38	-0.35	-0.36	-0.35	-0.35	-0.39	-0.44	-0.49	2	
0.83	0.83	0.72	0.48	0.11	-0.52	-0.7	-0.49	-0.36	-0.29	-0.28	-0.30	-0.30	3	
-0.64	-0.57	0.04	0.61	0.85	0.91	0.88	0.86	0.83	0.67	0.26	-0.35	-0.47	4	
-0.13	-0.12	-0.22	-0.28	-0.31	-0.36	-0.35	-0.30	-0.23	-0.13	0.03	-0.03	0.05	5	
-0.30	-0.32	-0.41	-0.46	-0.43	-0.35	-0.28	-0.23	-0.16	-0.08	0.03	0.06	0.08	6	
-0.34	-0.30	-0.15	-0.18	-0.33	-0.47	-0.43	-0.37	-0.28	-0.17	-0.05	0.00	0.01	7	
-0.43	-0.44	-0.56	-0.61	-0.67	-0.70	-0.85	-0.64	0.01	0.38	0.69	0.9	0.84	1	Open area
-0.63	-0.59	-0.53	-0.50	-0.48	-0.44	-0.42	-0.41	-0.35	-0.36	-0.43	-0.46	-0.52	2	
0.77	0.78	0.72	0.46	0.07	-0.70	-0.75	-0.51	-0.35	-0.32	-0.29	-0.30	-0.30	3	
-0.64	-0.66	0.01	0.64	0.93	0.88	0.85	0.83	0.79	0.66	0.26	-0.43	-0.51	4	
-0.20	-0.18	-0.31	-0.37	-0.39	-0.48	-0.45	-0.40	-0.34	-0.20	0.01	-0.12	0.07	5	
-0.37	-0.37	-0.49	-0.54	-0.51	-0.44	-0.37	0.32	-0.22	-0.11	0.06	0.00	0.02	6	
-0.39	-0.37	-0.21	-0.24	-0.40	-0.58	-0.53	-0.48	-0.38	-0.24	-0.1	0.03	-0.03	7	

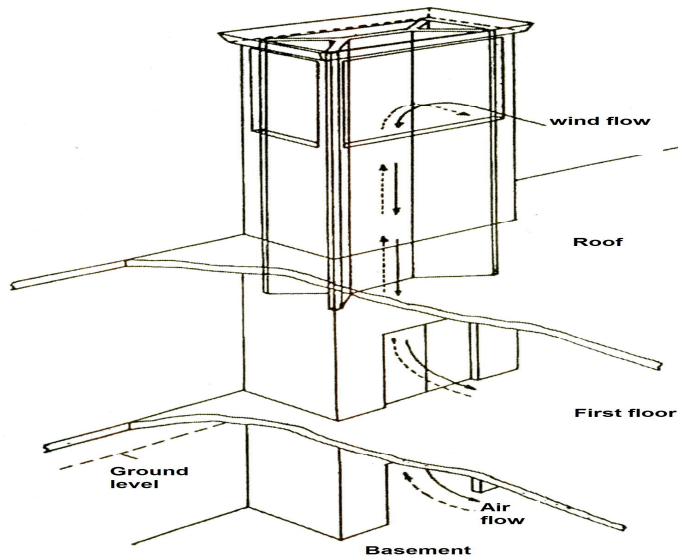


Figure-1: Common or traditional wind-catcher section basement
 -----> The arrow shows air flow when the wind catcher is sucking the air.

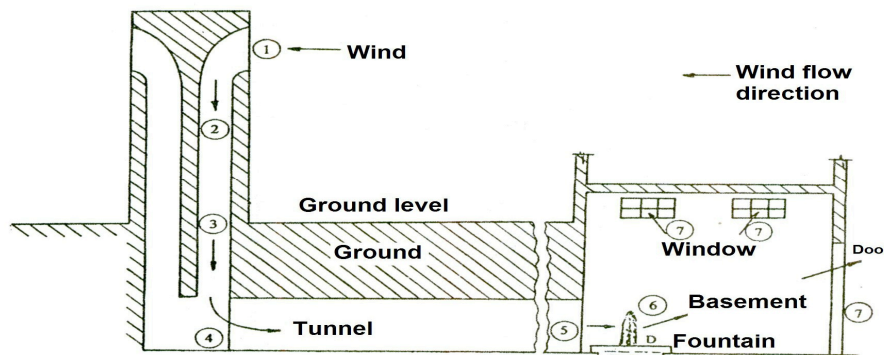


Figure-2: Wind-catcher & under ground section in Bam (Iran)

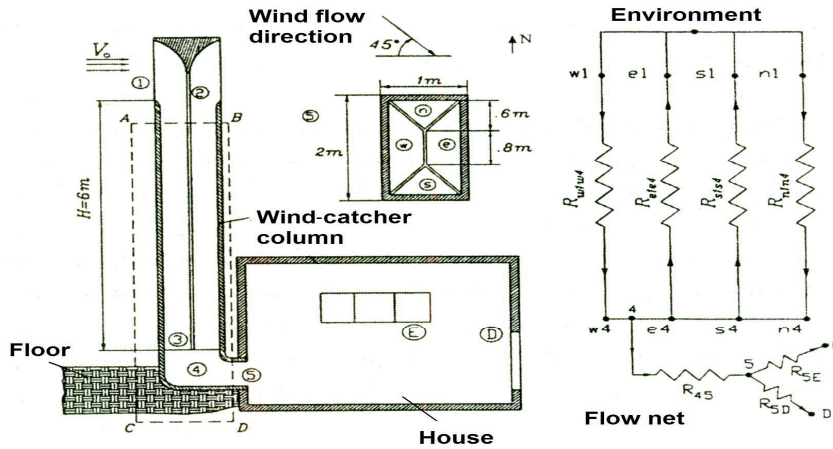


Figure-3: Traditional wind-catcher section & its flow net

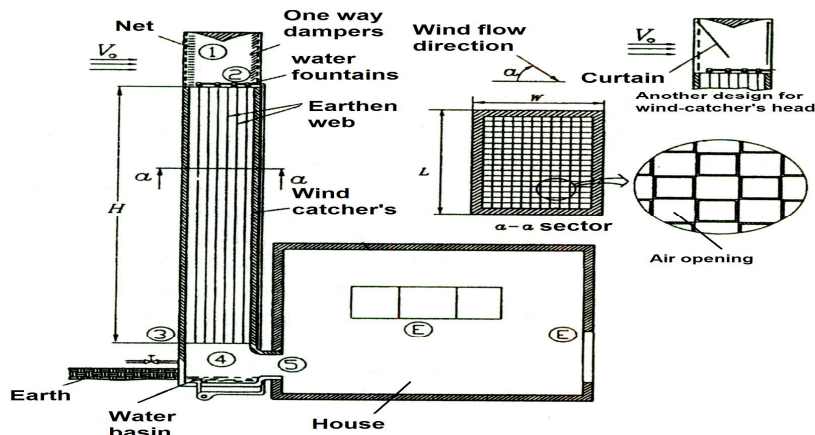


Figure-4: Wind-catcher's section & its water absorbing column

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A KNOWLEDGE-BASED SYSTEM FOR MIX DESIGN OF CONCRETE CONTAINING POZZOLANIC MATERIALS

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ABSTRACT

Design of concrete mixtures is the process of selecting the most economical and practical proportions of all the ingredients to produce quality concrete. Because of the nature of the mix design process and all the heuristic knowledge that is associated with it, concrete mix design lends itself well to a knowledge-based application. This paper describes a prototype knowledge-based system called HPCMIX that can be used to load the experimental data, develop several statistical models to determine the responses, evaluate the predicted models, and predict the 28 days strength and slump. It can also be used to design HPC mix proportions that are subjected to specified constraints (e.g. strength and slump). It is capable of selecting proportions of mixing water, cement, supplementary cementitious materials, aggregates and superplasticizer, with consideration being given to the effects of air content as well as water contributed by superplasticizer and moisture conditions of aggregates. As most of knowledge-based systems, this system has explanation facilities, can be incrementally expanded, and has an easy to understand knowledge base. The system was tested on a sample project. The system's selection of the concrete proportions compared favourably with the expert's selection. The system is very useful for civil engineering students as well as practicing engineers.

KEYWORDS: high performance concrete; mix design; statistical analysis; knowledge-based system; superplasticizer.

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1. INTRODUCTION

The selection of mix proportions is the process of choosing suitable ingredients of concrete and determining their relative quantities with the object of producing as economically as possible concrete of certain minimum properties, notably strength, durability, and a required consistency (Neville 1995). Because the ingredients used are essentially variable and many of the material properties cannot be assessed truly quantitatively, selecting proportions for concrete can also be defined as the process of finding the optimum combination of these ingredients on the basis of some empirical data as stated in relevant standards, experience, and some rules of thumb (Akhras & Foo 1994).

Concrete mix design involves complicated issues, and the correct ways to perform this can be achieved with expert advice and experience (Bai & Amirghanian 1994). Mix design of pozzolanic concrete is more complicated because pozzolanic concrete includes new materials, like superplasticizers and supplementary cementitious materials (e.g., silica fume, fly ash, fillers etc.). Traditionally, experienced civil engineers, largely based on their experiential knowledge, do the job of mix design. However, experts are not always available, nor do they always have the time to consult all possible references, review available data, and so on. Some companies do not have personnel with the experience to make the necessary decisions regarding concrete mix design. The conventional computer programs are useful only in manipulating the numerical data and providing mathematical reasoning for the final selection. They lack the intuition reasoning based on heuristic knowledge such as experience and rules of thumb (Foo & Akhras 1993). It is believed that the problem of mix design of pozzolanic concrete such as high performance concrete (HPC) can be alleviated if the engineer can be augmented with some “knowledge-based system” for affirming his/her judgment.

The conceptual computer-integrated knowledge system for HPC incorporates an expert system along with models and databases (Stoelhorst & Den Boer, 1998). It maps a methodology for designing a concrete mixture to obtain a specific service life. First, an optimization techniques and mathematical models are used to define the necessary material properties required by the concrete to give the desired service life. Then, the distributed databases are searched to determine if existing mixture design can be found which give the necessary material properties at an acceptable cost (Clifton, Bentz & Kaetzel 1997). If such mixture design is found, it is specified. If not, a new mixture must be formulated.

There are a few methods use statistical and computer programs to predict and optimize the HPC mix design. One of them is neural networks which are a family of massively parallel architectures that solve difficult problems via the cooperation of highly interconnected but simple computing elements (or artificial neurons). The processing elements of a neural network are similar to the neuron in the brain, which

consists of many simple computational elements arranged in layers (Jung & Jamshid 2001). The basic strategy for developing a neural network-based model for material behavior is to train a neural network on the results of a series of experiments using that material. If the experimental results contain the relevant information about the material behavior, then the trained neural network will contain sufficient information about materials behavior to qualify as a material model. Such a trained neural network not only would be able to reproduce the experimental results, but also it would be able to approximate the results in other experiments through its generalization capability. Another method is genetic algorithm, known as a very efficient heuristic algorithm that has been widely used in the various fields of engineering (Caldas & Norford, 2002), gives a more accurate results than other algorithms in the mix proportioning problem having many local solutions. For example, Lim (Lim, Yoon, & Kim, 2004) proposed a mix design method for high-performance concrete using genetic algorithm. By applying the genetic algorithm for design of high performance concrete mixtures, the number of trial mixtures with desired properties in the field can be reduced.

This paper presents intelligent software for prediction and optimisation of concrete mix proportion. The software is developed in Matlab environment using advance statistical method and non-linear optimisation. Developed software is to assist engineers, concrete producers, and researchers in optimizing concrete mixtures for their particular applications. Software is intended to introduce to concrete practitioners who are unfamiliar with the concepts and process of applying statistical analysis to concrete mixture proportioning. Software allows users to learn how to apply statistical analysis to the problem of optimizing concrete mixtures. The software could be applied to proportion a concrete mixture to meet a set of specifications at minimum material cost.

2. HIGH-PERFORMANCE CONCRETE MIX DESIGN METHOD

Aitcin (1998) provided guidelines for the proportioning of HPC mixtures. The method by itself is very simple: it follows the same approach of ACI 211.1-91 (ACI Committee 211 1991). It is a combination of empirical results and mathematical calculations based on the Absolute Volume Method. The water contributed by the superplasticizer is considered as part of the mixing water. A flow chart of the method is available elsewhere (Aitcin 1998). The procedure is initiated by selecting five different mix characteristics or materials proportions in the following sequence: water-binder ratio, water content, superplasticizer dosage, coarse aggregate content and air content.

3. SOFTWARE DEVELOPMENT

The following are some of the considerations that shaped the development of the prototype software:

- 3.1 Introduce statistical optimization approach in the context of concrete mixture proportioning. The purpose of the prototype software is to introduce the statistical approach as a means of predicting and optimizing concrete mixtures.
- 3.2 Minimize the required number of experimental runs needed to produce an optimal mix. Concrete producers want to minimize the effort (and cost) needed to identify optimal mixtures. Therefore, the number of factors was limited to six.

4. OVERVIEW OF PROTOTYPE STEPS

The tasks required to optimize a concrete mixture using statistical methods have been assigned to the steps listed below which also can be found in Figure 1.

4.1 Load data

As shown in Figure 2, user can load the raw data from the location where it is stored on the hard disk. Data analysis push button is used for the analysis of the raw data such as mean, minimum, maximum, range, standard deviation and variance. A plot of any of the concrete constituents verses slump or strength can be done from plot push button. The linear relation of any of the concrete constituents with strength or slump can be determined using determine model parameters button. User can enter the concrete constituent's properties as shown in Figure 3

4.2 Model development

Figure 4 shows the model development menu. Update the data from the data location. Pushbutton build math model, builds the math model (linear, pure quadratic, interaction and full quadratic).push button choose model, allows the user to switch among developed models. View model button, shows the constant term and all the coefficients. Reset button initializes all values to be equal to zero.

4.3 Model evaluation

Figure 5 shows the model evaluation menu. Update data from the data location can be done from pushbutton update data. Evaluate all models (linear, pure quadratic, interaction and full quadratic) can be done from pushbutton evaluate models. Data analysis button analyses data (f-value, mean square error...) .Plot button enables user to plot the residuals and the predicted values against the mix number. Best models can be determined using pushbuttons best model slump and best model strength.

4.4 **Response prediction**

Figure 6 shows the response prediction menu. User can update the data from the data location. Evaluation pushbutton evaluates all models. Enter the mix proportions allows user to enter the mix proportions to predict the strength and slump for all developed models. Choose model for strength and choose model for slump buttons allow user to determine which model to use for the prediction of strength and slump. Strength estimation and slump estimation buttons show the estimated values for strength and slump. Using response surface button a prediction plot of any of the models can be done as shown in Figure 7. Lower and upper limits of concrete ingredients can be entered from this menu. For any two variables, surface and contour plots against strength or slump can be plotted. The plots can be exported, saved, or printed using the file options.

4.5 **Optimization**

Figure 8 shows the optimization menu. This menu allows the user to do the optimization easily by choosing the model, entering material properties, lower and upper bounds, initial values, cost of materials, required strength and slump, air content and water binder ratio. The run optimization pushbutton gives suggested mix proportions; total cost of the mix; strength; slump; w/b ratio and air void content. In addition, if the optimization is done successfully a message “optimization successful” will be displayed, if not a message, “optimization terminated” will be displayed.

5. **ADVANTAGES AND DISADVANTAGES OF THE SOFTWARE**

5.1 **Some advantages of the software are:**

- a. Software is easy to use.
- b. Software saves time, reduce waste material and decrease design cost.
- c. Generates and presents a good quality graphics.
- d. Software uses mathematically rigorous numerical optimization.
- e. The user can save the results on the computer hard disk.

5.2 **Some disadvantages of the software are:**

- a. The software is limited to six components.
- b. It runs under Matlab.

6. **CONCLUSIONS**

This paper presented the development and application of a knowledge-based system called HPCMIX for the proportioning of HPC mixtures. The prototype software that can be used to optimize concrete mixture proportions using

statistical analysis approach that give concrete practitioners an opportunity to apply this approach to their own mixture development. Because statistical approach was likely to be unfamiliar to many practicing engineers and producers, the aim was to make it as user-friendly as possible. The research demonstrated that HPC could be successfully optimized using statistical analysis.

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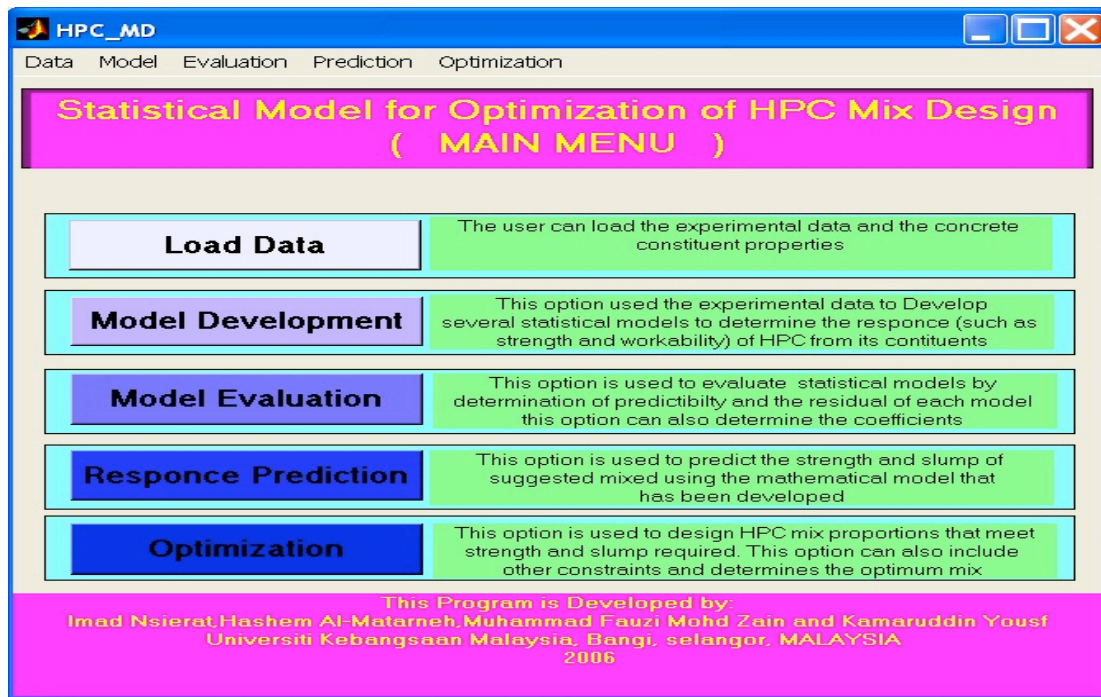


FIGURE 1 Main Menu of the Prototype

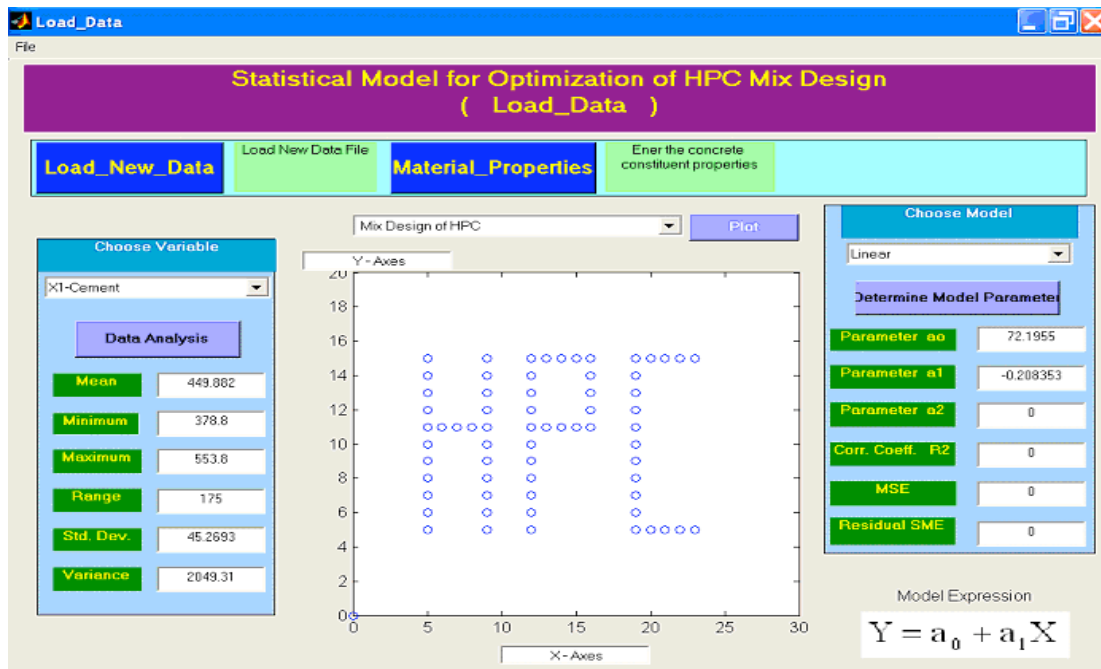


FIGURE 2 Load Data Menu

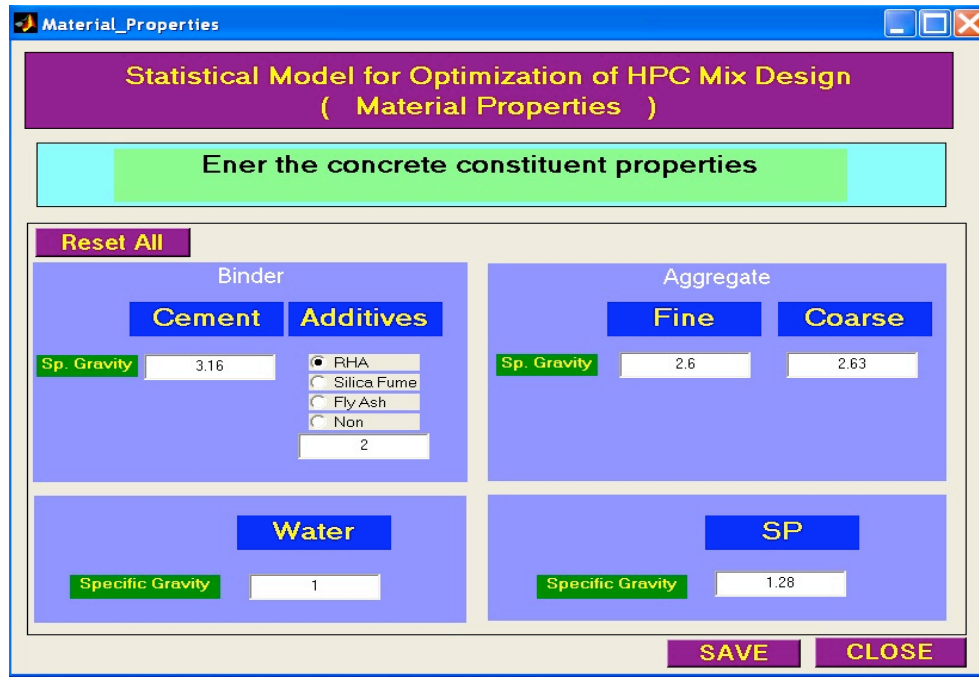


FIGURE 3 Material Properties Menu

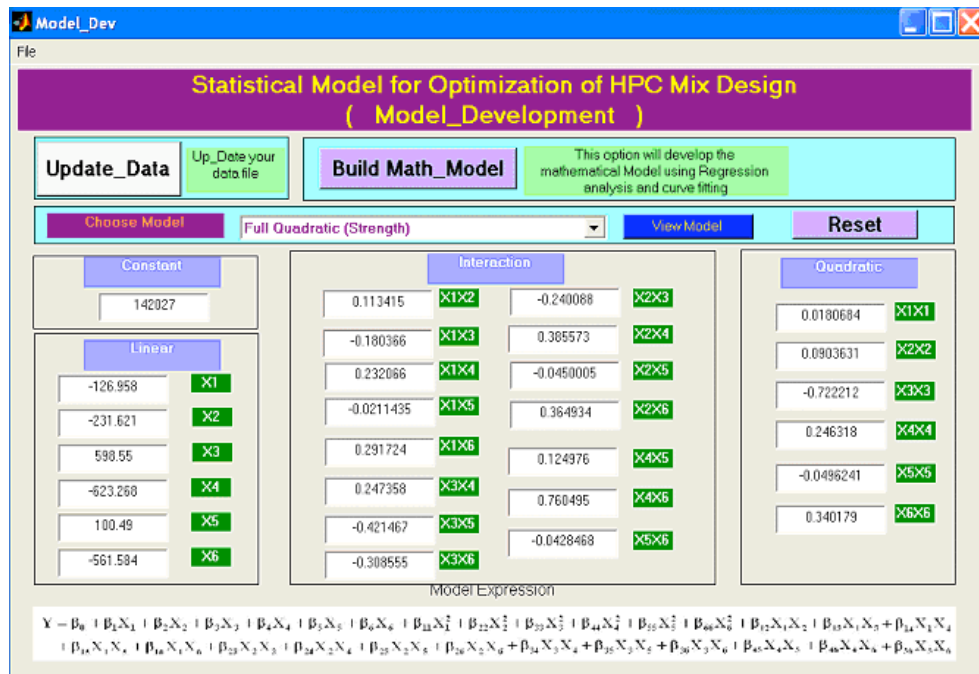


FIGURE 4 Model Development Menu.

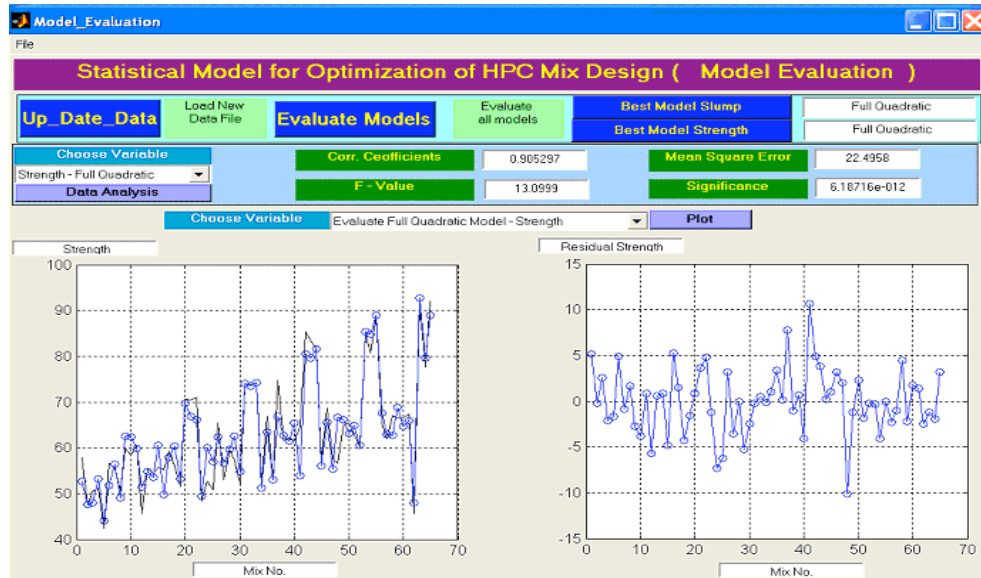


FIGURE 5 Model Evaluation Menu

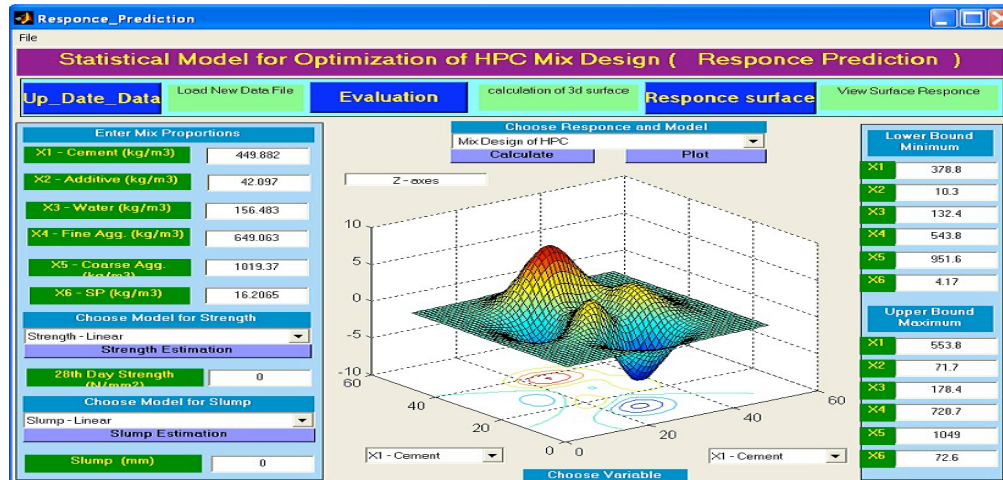


FIGURE 6 Response Prediction Menu

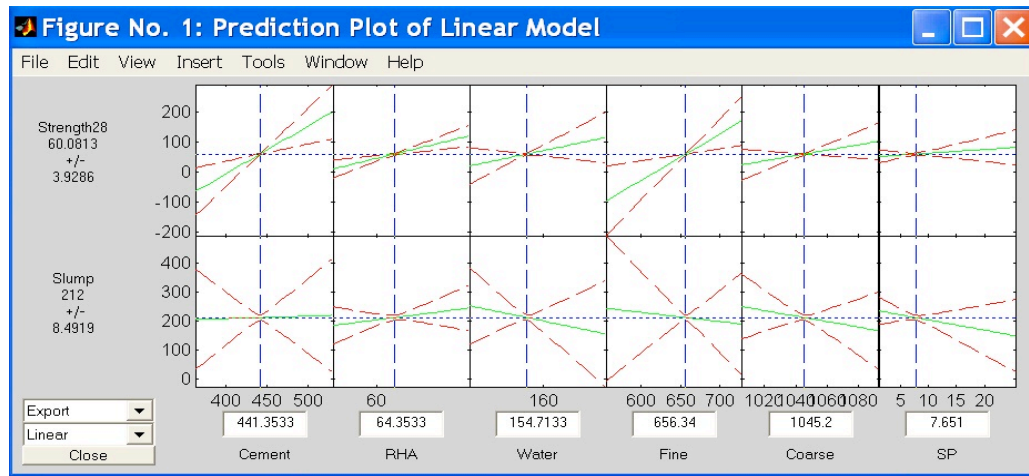


FIGURE 7 Predictions Plot of Models

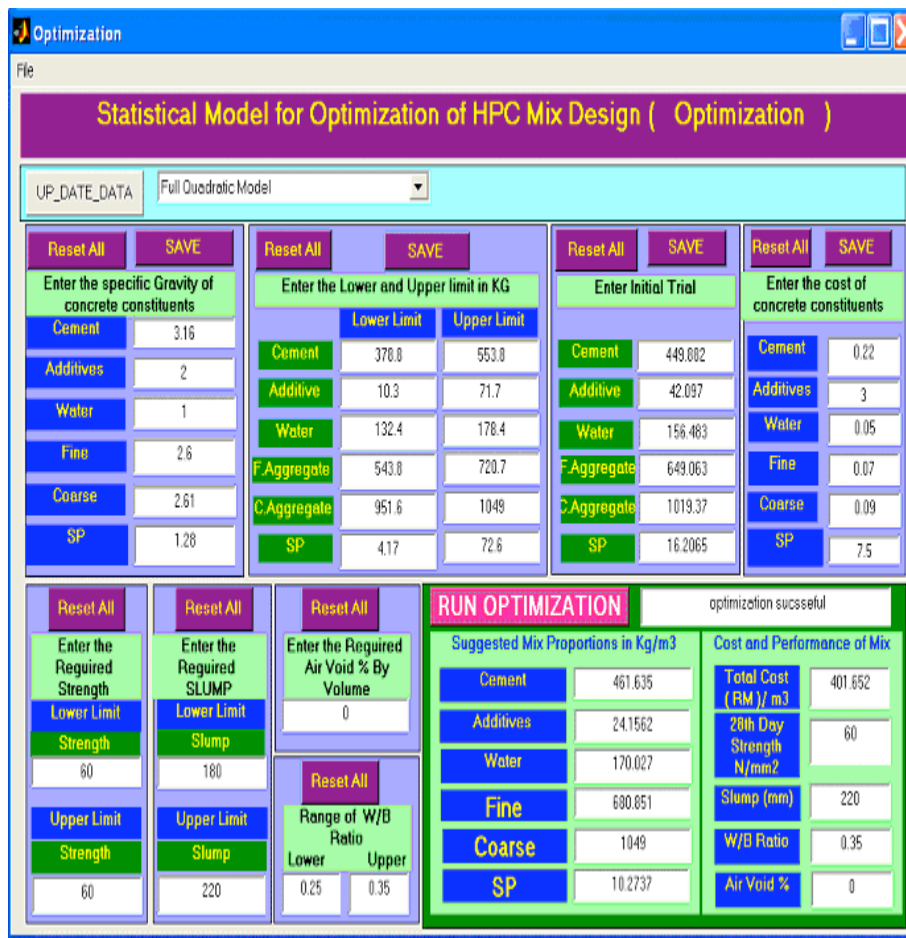


FIGURE 8 Optimization Menu

ENVELOPE AND BUILDING SYSTEMS: A TOOL FOR CONTROLLING AND MANAGING SMART BUILDINGS

Guido R. Dell'Osso¹, Francesco Iannone², Michele Palella³ and Giovanni C. Migliaccio⁴

ABSTRACT

This paper describes a software program developed to control smart building components (SBCs) and improve comfort in buildings that use SBCs. In a virtual environment, this software package allows to control automatically the performance of buildings that include SBCs in their envelope and systems. The software has been calibrated on a simple model of a virtual building that is characterized by an SB intelligent skin, so that it is enabled to change its own characteristics and to modulate them in accordance to variability of boundary conditions. Internal comfort is guaranteed by hourly check and automatic management of the following elements: (a) Electrochromic glasses, (b) External vertical screening, (c) Internal vertical screening, (d) Ventilation grids, and (e) Radiant floor heating system.

Keywords:

Smart Building, Building Performance, Energy Efficiency, Control Systems

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1 Introduction

Widespread concerns on global warming and a recent surge in energy prices are driving the desire to implement sustainable concepts within the building development process. However, a conscious identification of actions that may make a building project sustainable is difficult to achieve because the effectiveness of the many innovative technologies has been rarely assessed in terms of comfort performance. In addition, the systemic nature of buildings often makes the implementation of a single technology impossible for the absence of adequate complementary systems.

While suppliers of construction components are flooding the market with new technological products designed to make buildings more sustainable, early implementers often face substantial problems in employing these products. Among many other technologies, smart building components (SBCs) are being proposed as the solution to the problem of making buildings sustainable. However, the implementation of SBCs requires the use of sophisticated control systems often including complex sensor networks. Therefore, there is the need to develop, calibrate and validate applications that collect and analyze data coming from these sensors and control the SBCs.

This paper includes findings from a research study on the implementation of SBCs in buildings. In a virtual environment, this software package allows to check and to manage automatically the performance of buildings that include SBCs in their envelope and systems. The software architecture is designed to allow tuning up the building to achieve the desired levels of comfort according to internal needs and external climatic conditions. Three seasonal working rates are allowed: winter, half season and summer. The software has been calibrated on a simple model building that is characterized by an SB intelligent skin, so that it is enabled to change its own characteristics and to modulate them in accordance to variability of boundary conditions. Internal comfort is guaranteed by hourly check and automatic management of the following elements: (a) Electrochromic glasses, (b) External vertical screening, (c) Internal vertical screening, (d) Ventilation grids, and (e) Radiant floor heating system. This implementation confirms that it is easily possible to control and manage complex dynamical system integrated in buildings. The authors also discuss how architects and engineers can use the simulation feature of this tool for design purposes.

2 Control system and virtual model

The purpose of this study was to develop a system for controlling indoor thermal and lighting performance of a smart building. To validate the effectiveness of this system, the authors tested it on a virtual building model that is composed by a parallelepiped suspended on columns with all its surfaces exposed to the external environment as shown in Figures 1 and 2. While the dimensional characteristics of the building are treated as parameters, its orientation is fixed. Additional constraints were assigned to the window system to simplify the implementation: one window

was added to each vertical surface with the southern window having a larger surface than the northern one.

The regulation system is managed by a software program. This software allows users to input values for each system's variables. It also stores and processes external and internal climate data, and sends signals to control the building components that regulate the internal microclimate. Characteristics of the building envelope have been defined according to maximum thermal transmittances imposed by Italian law [12] that are shown in the Table 1.

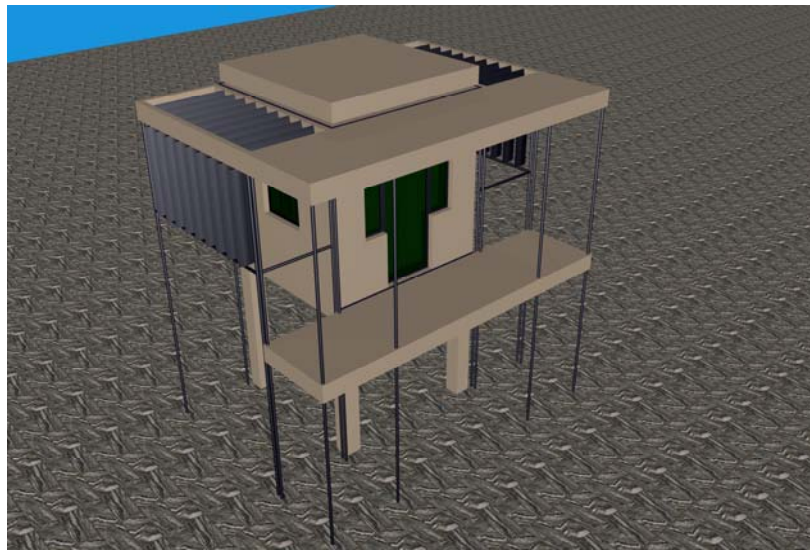


Figure 1: Virtual building model

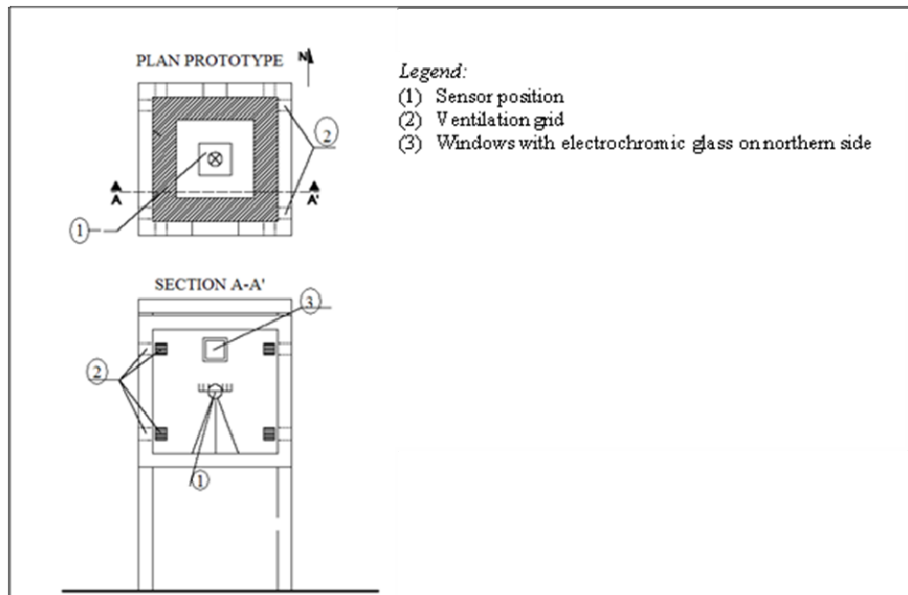


Figure 2: Plan and section of virtual building model

Table 1: Characteristics of Building Envelope

<i>Building components</i>	<i>Maximum transmittance [W/m²K]</i>	<i>Time lag [h]</i>	<i>Decrement factor</i>	<i>Solar factor</i>
Vertical surfaces	0,4	7-24	<= 0,3	-
Windows	2,1-2,6*	-	-	0,7-0,5*
Roofing floor	0,38	7-24	<= 0,3	-
Floor on pilotis	0,42	7-24	<= 0,3	-

(*) respectively for transparent and opaque glass

The thermal insulation layer of the external walls is on the external surfaces of the virtual building model. The control of the sun radiation is made through external and internal screenings and electrochromic glasses. Southern external screenings are horizontal and fixed as shown in Figure 2a, which includes a section north-south. Eastern and western screenings are movable and managed by the central control unit; they have been designed for windows shielding as shown in section east-west in Figure 2b. Electrochromic glasses, which section is represented in Figure 2c, get transparent when a low rating power (about 2 W/m²) is given. The optical transparency is related to the sun radiation and to the seasons' regime.

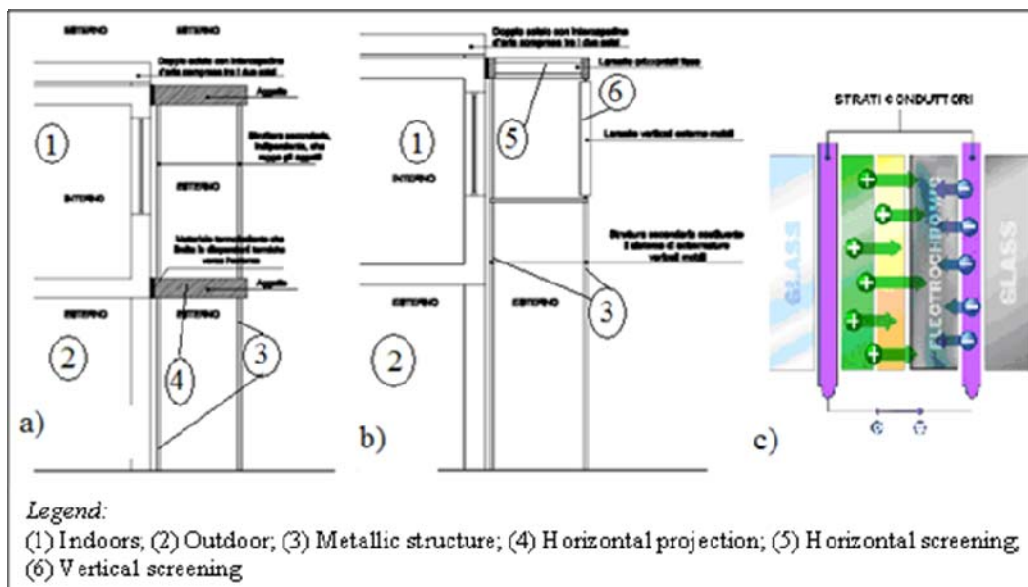


Figure 3: Characteristics of vertical envelope

The ventilation system is hybrid, and it is composed by a fan and by ventilation grids provided with a soundproofing system, a filter, and anti-raining system. During the winter, spring and fall seasons, the hourly ventilation rates are adjusted to guarantee desired levels of indoor air quality; in summer, the ventilation system provides also

for natural cooling. The ventilation grids offer eight positions to adjust the ventilation rates according to internal air speed and relative humidity.

The heating system includes a gas methane condensing boiler and a radiant floor. A radiant floor operates at low temperatures to allow comfortable standings of the users, but also to recover energy normally discharged to the atmosphere through the chimney. This characteristic makes this type of heating system very efficient. In addition, the use of a modern gas methane condensing boiler with linear continuous modulation (LCM) assures effectiveness even for low thermal loads.

The Domestic Hot Water System (DHWS) is powered by solar energy. The solar thermal collectors are integrated on the railings of the southern balcony. The DHWS includes a water tank for heat storage and an electric heating system as auxiliary energy source.

3 Control System Software

3.1 Software Architecture

A software program for managing and controlling the virtual model was developed. This program included a pre-processing component to input data, a post-processing component to report output data and command SBCs, and a processing component that embeds all the logic relationships of the control system. Figure 3 shows an excerpt of the logic used to control operations of the internal vertical screenings. The control system used its internal clock and real-time measurement of the daylight factor to control the screenings. Figure 2 shows the presumed position of the sensor station.

Algorithm for the regulation of the vertical inside screenings (Sentence in correspondence of every present window in the output of the FLDm)				ESP		
if	hc:6:00 or hc:18:00		then	Closes the vertical inside screenings. The consumer is left to the possibility to turn on the lamps. (Pos. 0 - Thin plates parallel to the glass)	N.S.O.E	
if	6:00:hc:18:00	and	FLDm<3%	then	Closes the vertical inside screenings. The consumer is left to the possibility to turn on the lamps. (Pos. 0 - Thin plates parallel to the glass)	N.S.O.E
if	6:00:hc:18:00	and	FLDm>5%	then	The vertical inside screenings are closed for bringing the values acceptable FLDm. (Pos. 0 - Thin plates parallel to the glass)	N.S.O.E
if	6:00:hc:18:00	and	3%<FLDm<4%	then	The vertical inside screenings are totally open. (Pos. 1 - Thin plates perpendicular to the glass)	N.S.O.E
if	6:00:hc:12:00	and	4%<FLDm<5%	then	The vertical inside screenings are totally open. (Pos. 1 - Thin plates perpendicular to the glass)	N.O.
if	12:00:hc:18:00	and	4%<FLDm<5%	then	The vertical inside screenings are totally open. (Pos. 1 - Thin plates perpendicular to the glass)	N.E.
if	6:00:hc:12:00	and	4%<FLDm<5%	then	The vertical inside screenings have to rotate of 45° to sx. (Pos. 2 - Thin plates rotated of 45° to sx looking at the glass)	E.
if	12:00:hc:18:00	and	4%<FLDm<5%	then	The vertical inside screenings have to rotate of 45° to dx. (Pos. 2 - Thin plates rotated of 45° to dx looking at the glass)	S.
if	6:00:hc:12:00	and	4%<FLDm<5%	then	The vertical inside screenings have to rotate of 45° to dx. (Pos. 3 - Thin plates rotated of 45° to dx looking at the glass)	S.
if	12:00:hc:18:00	and	4%<FLDm<5%	then	The vertical inside screenings have to rotate of 45° to dx. (Pos. 3 - Thin plates rotated of 45° to dx looking at the glass)	O.

Figure 4: Control system logic

3.2 Input data

Figure 4 shows the input screen. Yellow fields contain values of fixed or parametric variables, while white fields contain the environmental data monitored through sensors. The software also allows the input of boundary conditions to allow a preliminary evaluation of the software without a real system.

A geographic location for the virtual building model is required to identify local weather data. In the demonstration, the chosen location was the city of Bari in South Italy. Degree-day value for this location is equal to 1,185. Table 2 includes information on seasons for the given location.

Table 2 - Operating Seasons

Winter Season	Half Seasons	Summer Season
Nov 15 – Mar 31	Apr 1 – Jun 14 Sept 15 – Nov 14	Jun 15 – Sept 14

POLITECNICO DI BARI
TESI IN PROGETTO DI SERVIZI TECNOLOGICI
di Michele Pajolla

	Orientamento	Coeff. Esp.	Sup. lorda [mq]	Sup. finestra [mq]	U opaco [W/mqK]	U fin. ON	U fin. OFF	F.S. ON	F.S. OFF	Sfasamento [h]	Smorzamento [%]	Irradiazione [W/mq]
Sup. Disp. 1	NORD	1,20	9,00	0,50	0,40	2,60	2,10	0,70	0,50	7	0,30	40
Sup. Disp. 2	SUD	1,00	9,00	2,00	0,40	2,60	2,10	0,70	0,50	7	0,30	20
Sup. Disp. 3	OVEST	1,10	9,00	0,00	0,40	2,60	2,10	0,70	0,50	7	0,30	20
Sup. Disp. 4	EST	1,15	9,00	0,00	0,40	2,60	2,10	0,70	0,50	7	0,30	40
Pavimento 5		1,00	9,00		0,42					7	0,30	
Copertura 6		1,00	9,00		0,38					7	0,30	

TEMPERATURA ESTERNA [°C] DIREZIONE VENTO ORA DEL GIORNO O NOTTE [hh:00]
 TEMPERATURA INTERNA [°C] VELOCITA' ARIA INTERNA [m/s] ILLUMINAMENTO MEDIO ESTERNO [lux]
 ALTEZZA INTERNA LOCALE [m] UMIDITA' RELATIVA INTERNA [%] ILLUMINAMENTO MEDIO INTERNO [lux]

Legenda:
 Dati rilevati o immessi manualmente. (Campi obbligatori)
 Dati di progetto, rimangono fissi. (Campi obbligatori)

Giorno Mese
 [RESETTA] [START] [ESCI]

Figure 5: Software input form

3.3 Output data

The software provides output data in several screens that include the following information: (a) input data (white cells); (b) design information (yellow cells); (c) performance data (orange cells); (d) sensor data (pink cells); (e) system warnings that are triggered when the system is functioning outside the optimal ranges (red written). These output data are provided through different screenshots depending on the seasonal regime (e.g., winter, summer, half seasons).

In winter regime, output data include the energy emitted by the terminal of the heating system, the transmission energy loss, the ventilation energy loss, the solar radiation that enters the room, the thermal balance, and the day-lighting level that regulates the inside screenings. Figure 6 shows the output related to the radiant floor. The data are (1) the surface of the floor, (2) the thermal efficiency of the radiant floor, (3) the water temperature in the radiant floor, (4) the thermal energy emitted by the radiant floor, (5) the necessary number of iterations to convergence the calculation algorithm, and (6) the warning related to the operation of the condensing boiler. As example, in the hypothesis of very low external temperature, the condensing boiler increases the temperature of water and it could result in a significant reduction of the boiler efficiency.

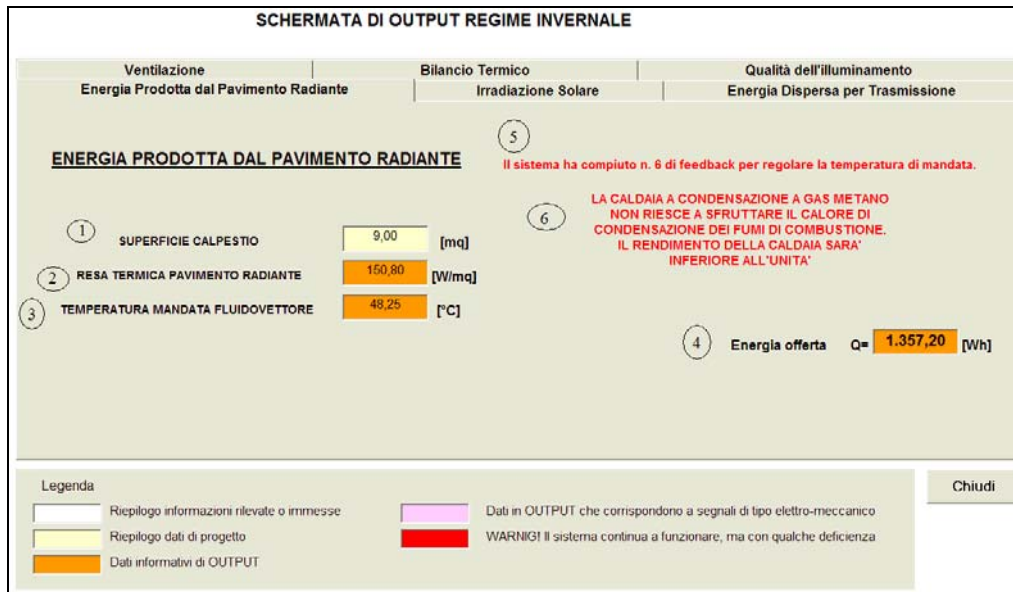


Figure 6: Screenshot of output for radiant floor

In summer regime, the system outputs include the following information: the time lag of the thermal wave that governs the possible surplus of ventilation to reduce the overheating; the solar radiation entering the room that governs the screenings and the transparency of the electrochromic glasses; the hourly air-change rates that govern the ventilation system; and the day-light level that governs the inside screenings. In Figure 7, outputs of the software in summer regime are shown for the solar radiation screenshot where values of the following variables are provided: (1) the solar radiation, (2) the operational status (on/off) of the electrochromic glasses, (3) the angle between the straight line perpendicular to the wall exposed to east and the external screens (alpha), and (4) the angle between the straight line perpendicular to the wall exposed to west and the external screens (beta).

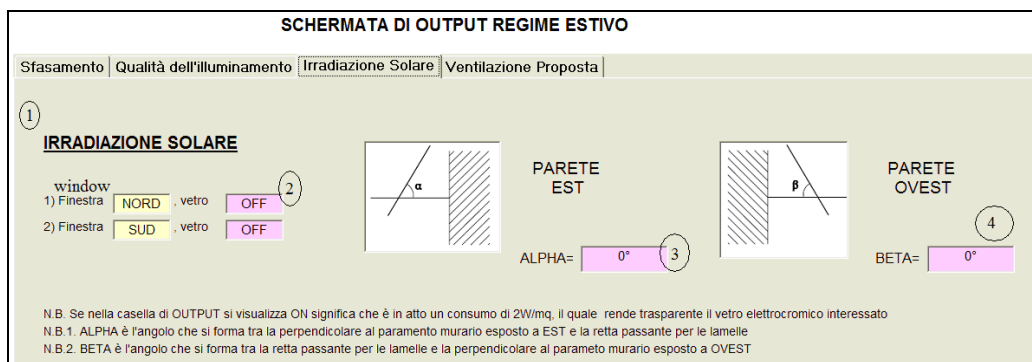


Figure 7: Output of the software in summer regime: solar radiation

4 Conclusions

Recently, vendors are making available high performance building components, such as SBs. The implementation of these components into a building require the use of more sophisticated building management strategies that guarantee an integrated control of façade, building components and building in the whole. This paper described a system for controlling SB components in buildings. The logic of this software program was tested on a virtual building model to demonstrate that technology is available to make easier and intuitive the management of automated systems and very complex technologies. Authors adopted a building model that included the following not traditional SB components: (a) Electrochromic glasses, (b) External vertical screening, (c) Internal vertical screening, (d) Ventilation grids, and (e) Radiant floor heating system. The described implementation confirms that it is easily possible to control and manage complex dynamical system integrated in buildings. Nevertheless, the use of dynamic control systems should provide a greater flexibility to the designers allowing a more active control of variable loads. These systems would guarantee, in every climatic condition, the thermal and lighting indoor comfort.

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ENERGY EFFICIENCY AND SUSTAINABILITY IN BUILDINGS

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Abstract:

The importance of energy efficient buildings has assumed great urgency in light of fast depleting energy resources, energy scarcity and increasing environmental pollution. Innovative ways to cut down energy consumption are necessary. The construction industry is one of the largest energy consuming sectors. In modern buildings significant amount of energy is consumed to keep the building environment comfortable. In developing countries like India, rising population, increasing standards of living and rapid urbanization result in an increase in building construction activities. To achieve the collective objectives of energy security and environmental protection, eco-sensitive buildings that utilize their resources judiciously, minimize their emissions and have efficient waste management systems, should be considered and designed. The available options in architectural intervention, building materials and design methodologies need to be carefully evaluated to minimize energy usage, minimize the ecological degradation that may be caused by the construction of the building and provide cost effective solutions. The aim is to achieve the desired comfort with the least input of conventional energy. Architects and designers accomplish the task through solar passive design, use of renewable energy technology systems, and/or natural building materials. While designing such buildings, not only new building stock can be targeted but also existing buildings can be retrofitted with energy efficient and eco-friendly technologies, thereby substantially reducing energy consumption. All put together is Energy Efficient Housing.

1. INTRODUCTION

If we are about to build or renovate our home, we may wish to consider making it as energy efficient as possible. Naturally warm in winter and cool in summer. There are several advantages to living in an energy efficient home - saving money on energy costs is the most obvious. Other benefits include the satisfaction of knowing the reduction of impact on the environment through the decreased use of fossil fuels, the increased comfort of effective natural lighting and ventilation and the improved resale value of home due to lower power bills. The easiest way to ensure our home's energy efficiency is to design it to suit the local climate. By taking advantage of free natural warmth from sunlight and cooling from breezes. Careful building design can easily achieve internal temperatures that are 5°C warmer in winter and 10°C degrees cooler in summer. Most features such as improved layout, appropriate window placement and sensible garden design, will make little difference to initial building cost. Other features such as fully insulated home may add to costs, but the savings in energy bills will quickly pay them back. After that saving money in the face of rising energy costs year after year after year. Basic energy sources in an eco-friendly building complex are PANCH-TATVA, i.e., Sky, Sun, Air, Earth, Water. The main features of energy efficient housing relate to: • Building orientation • Internal room layout • Window placement, sizing and shading • Use of insulation • Ventilation • Draught proofing • Use of heat absorbing building materials • Landscaping • Use of energy efficient appliances. In general, energy efficiency in new buildings can be achieved through:

- Bio climatic architectural principles;
- Load minimization by the incorporation of solar passive techniques in building design;
- Design of energy efficient lighting and HVAC systems;
- Use of renewable energy systems to meet a part of the building load; and use of low energy materials and energy efficient methods of construction;

2. ENERGY EFFICIENCY

2.1 PASSIVE SOLAR DESIGN: Passive interventions and reduction of loads on conventional systems are integral parts of any architectural components similar to walls,

windows and roofs in a conventional structure. In any design, an architect can achieve energy efficiency and at the same time maximize comfort levels by studying the micro and macro climate of the site and applying bio climatic architectural concepts in terms of building orientation with respect to the sun, shading of windows, color, texture, landscaping etc. The solar passive design generally varies according to the climatic condition prevalent at the site. In cold climates, south-facing windows designed to let in the sun's heat while insulating against the cold are ideal. In hot and moderate climates, the strategy is to admit light while rejecting heat. Water bodies may be used in hot and dry climates for evaporative cooling as well as heat sinks for reducing the thermal heat gain. Useful daylight is harnessed through appropriate windows, skylights and light shelves. One of the passive solar cooling devices is the thermal chimney, which can be designed like a smoke chimney to vent hot air from the house out through the roof. However, one has to resort to advanced techniques of passive conditioning such as roof ponds, wind towers, etc in extreme climatic conditions.

2.2 ENERGY CONSERVATION: Energy conservation is possible by judicious design of lighting and HVAC (heating, ventilation and air conditioning) systems, controls and operation strategies. Increasing insulation levels in conditioned buildings is regarded as the most cost-effective investment in energy efficiency. Thermal insulation of external walls, roofs and floors, and double-pane windows can reduce energy consumption for space heating by lowering heat losses through the envelope of the building. Energy consumption for cooling is also reduced because of lesser heat gains from outside through the envelope. Energy efficient windows with their high thermal insulating values and spectral selectivity can make air-conditioning systems work more effectively. This can lead to reduction in AC loads, lower consumption of electrical energy and reduction in peak load demand. The use of energy efficient glazing helps in minimizing unwanted solar gains in summer and heat losses in winter, while maximizing the amount of useful daylight in buildings. Lighting load constitutes about 10 to 15 percent of the total electrical load of a building, and so energy efficient lighting systems, such as compact fluorescent lamps and fluorescent tubes with electronic ballast, are generally recommended instead of conventional lighting fixtures to reduce the lighting load. Although the initial cost of such installation is high, they last longer and the running cost is also less. As electricity is efficiently converted to light in energy efficient lamps, the amount of heat generated is also less.

2.3 USE OF RENEWABLE ENERGY TECHNOLOGIES: Renewable energy systems are installed for meeting a partial load of the building, thus considerably reducing the overall electrical and thermal load. Solar energy could be utilized for a variety of purposes and in a number of ways: generating electricity, providing hot water, and heating, cooling, and lighting buildings. Solar photo voltaic (PV) cells can provide electricity for lighting. Solar thermal systems may be used for heating water or space heating and transpired solar collectors can preheat air for the building's ventilation system. Solar water heaters generate hot water at 60 to 85 °C and trombe wall is a thermal storage wall with vents provided at the top and bottom. It is made of concrete, masonry or composites of brick and sand and is generally located in the southern side (in the northern hemisphere) of a building to maximize solar gains. Conveniently be used for domestic use. The usefulness of a solar water heater can be ascertained from the fact that a standard 100-liter capacity system can be installed as an alternative to electric geyser for residential use that can save 1500 units of electricity annually and could prevent emissions of 15 tons of carbon dioxide into the atmosphere every year. It is estimated in India that the use of 1000 solar water heater of a 100-liter capacity each can contribute to a peak load saving of 1 MW. In cold and sunny climates, solar air heaters can be used extensively to reduce demand for electricity and firewood for space heating. Solar electricity can be generated by integrating solar cells in buildings, on roofs and facades, or by installing grid connected photovoltaic plants. It is an expensive proposition where grid

connected electricity is available, but for locations where conventional electricity is not available or the grid is far away, it makes an ideal alternative. Further, 'building integrated PV' (BIPV) systems may be installed for the generation of electricity, and surplus electricity generated during non-working hours could be fed to the grid. Inclined roofs, if oriented in the right direction, are an ideal support structure for PV modules. In moderate to high wind speed locations, roof top aero generators may be installed for the generation of electricity. A sunspace or solarium, which comprises of a combination of direct and indirect gain systems, may be used to heat up the living space by convection and conduction through the mass wall. Technologies have been commercialized to convert the solid and liquid waste generated in a building to productive use in the form of bio-fertilizer, gas for power generation, cooking etc. The long-term objective is thus to reduce building energy loads in a cost-effective manner such that renewable sources of energy can meet and exceed energy demand in the building.

2.4 ORIENTATION: A major principle of energy efficient building design is to allow the sun's heat into the home in winter while excluding it during the long hot days of summer. This can be achieved because the angle of the sun changes from season to season. In summer the sun rises earlier, south of due east and climbs high in the sky before setting south of due west. Major summer heat gain occurs through the roof and through the east and west-facing windows and walls of the home. In winter the sun rises later, north of due east and stays low in the northern sky before setting north of due west. North-facing windows and walls receive maximum winter sun and warmth. To achieve the design goal of optimal energy efficiency, an effective rule of thumb for a home in the southwest is to have north and south-facing walls 1.5 to 2.0 times the length of east and west-facing walls. This allows reasonable access to the winter sun from the north side of the home, while reducing the exposure of walls and windows to early morning and late afternoon sun on the east and west sides of the home. True north is the ideal orientation for windows. However, if your eaves are designed correctly, windows oriented between approximately 20° east or west of north still allow good sun entry in winter while excluding most of the direct summer sun.

2.5 INTERNAL ROOM LAYOUT: Arrange indoor living and entertaining areas on the north side of the home where possible, with other rooms to the south. This will create warm and bright living areas in winter since north-facing windows and walls receive maximum winter sun. The south side of the home receives a small amount of direct sun in summer, so by placing bedrooms to the south they'll be more comfortable for sleeping in summer. Group rooms with similar uses together to create zones and use doors to separate these zones. This type of design is more energy efficient than open plan living because one can close off rooms that are cooled or heated from those that are not. If we still like the ambience of open plan living, consider using glass doors to separate zones. One can try to group the kitchen, laundry and bathroom together. These are the hot water-using areas of the home and by grouping them together we can minimize the need for long hot water pipes. This will reduce the amount of heat lost from the pipes and consequently hot water use.

2.6 WINDOW PLACEMENT, SIZING AND SHADING: Windows serve many important functions. They can act as solar collectors trapping heat from the sun, which is useful in winter but not in summer. They also act as ventilators during summer, funneling cool late afternoon and nighttime breezes to rid your home of heat accumulated during the day. Windows also let in daylight. A balance needs to be struck between controlling the sun's access and allowing adequate cross ventilation from breezes, as well as allowing natural light to enter.

2.7 NORTH-FACING WINDOWS: It is recommended that around a third to a half of the north face of your home be glass, as it is very effective at trapping winter warmth and can be easily shaded from summer sun with correctly designed eaves. To calculate the overhang needed, multiply the distance from the eaves-line down to the bottom of the window by 0.7. This will ensure the glass is adequately shaded from September until March. For cooler regions, multiplying by 0.4 will provide suitable shade from October until February. Deciduous trees and shrubs or creepers growing on an open pergola on the north face of a home can also provide window shading in summer, while allowing the sun through to warm your home once they've lost their leaves in winter. Alternatively, a solar pergola is designed to achieve the same result. It is important that shading devices, whether in the form of eaves, pergolas or appropriate landscaping, do not block the sun's access to the interior of your home during winter.

2.8 EAST AND WEST-FACING WINDOWS: While east and west-facing windows provide warmth in winter from the early morning and afternoon sun, they are difficult to protect from the sun in summer. This makes rooms on the east and west sides of your home uncomfortably warm, particularly west-facing rooms which receive the hot afternoon sun. To keep cool in summer, it is recommended that the total area of east and west-facing windows be kept to a minimum. External shading devices provide some protection from the summer sun, with complete protection achieved only with full vertical screening, such as outside blinds or shutters. This is because the angle of sun will be close to horizontal early in the morning (east) and in the late afternoon (west), and only vertical screening can block the sun at these angles. Deciduous trees or vines growing on a trellis can also provide shading during summer.

2.9 SOUTH-FACING WINDOWS: South-facing windows receive no direct sun in winter but will receive a few hours of morning and afternoon sun in summer months. For this reason, they lose heat in winter and gain some undesirable heat in summer. South-facing windows should be large enough to allow good ventilation and light to enter the home without losing too much heat in winter. Vertical elements such as external screening or landscaping in conjunction with internal blinds will be most effective at shading south-facing windows, since the majority of this sun is at low angle. Basic 'eaves overhang' in combination with internal window treatments will also assist solar control to south-facing windows. This is because in mid-summer the sun can fall on an unshaded southern facade for approximately 4 hours in the morning and 4 hours again in the afternoon. For the more northerly latitudes provision of shading to south-facing windows is even more important. This is because at this latitude there can be an additional 45 minutes of mid-summer sun falling on the south face of a building, morning and afternoon.

2.10 INTERNAL WINDOW TREATMENTS: While external window treatments are the best way to reduce summer heat gain, internal window treatments are most important for reducing winter heat loss. A window can lose heat five to ten times faster than an equivalent area of wall. This heat loss can be minimized by keeping warm air inside the room away from cold windows. Closed curtains can be effective insulators and should be made from a heavy fabric with insulating backing for maximum effectiveness. They need to be long enough to reach the floor and should include a closed pelmet. The pelmet is an integral part of the curtain as it reduces air circulation and consequent heat loss through the window glass during winter and heat leakage into the home during summer when the curtains are drawn.

2.11 SKYLIGHTS: Skylights can reduce your daytime lighting needs. However, a typical Perth home consumes approximately six times as much energy for heating and cooling than for lighting, and heat can be lost from your home through skylights on winter nights and gained during hot days. To reduce this problem, position your skylight so it is shaded in summer or consider buying one with special glazing that minimizes heat transfer and can be closed at night. Non-vented ducted skylights lose less heat in winter, as the air trapped in the duct acts as a thermal buffer.

2.12 TINTED GLASS AND REFLECTIVE FILMS: Tinted glass and reflective films absorb and reflect heat, keeping your home cooler. However, be aware that using them reduces the amount of light and heat entering rooms in winter as well as in summer. During summer the glass itself also becomes hot as it absorbs energy, which will cause some heat to be radiated into the room. These products may be useful where large areas of east and west-facing glazing are unavoidable due to design reasons. However, tints and films will generally not reduce heat gain as much as external shading.

2.13 DOUBLE-GLAZING: Two panes of glass separated by at least 10 mm can reduce winter heat loss but is generally only cost effective in situations with high heating requirements. Double-glazing can also reduce conductive summer heat gain. However, when exposed to sun double glazed windows will still allow significant heat transfer, which means that full shading is still required.

2.14 INSULATION: Insulation makes your home more comfortable by reducing the amount of warmth escaping in winter and heat entering in summer. By insulating, you can significantly reduce your heating and cooling bills and help to reduce your impact on the environment. In an un-insulated home most heat is lost or gained through the ceiling and roof, so this is the most important part of your home to insulate. There are two main types of insulation - bulk and reflective. Bulk insulation works by trapping small cells or layers of air within the insulating material. Many pockets of still air are very effective at retarding heat transfer. Reflective insulation works by reflecting significant proportions of light and heat.

Some reflective foils can be used as both a vapor barrier and to reduce heat transfer. When selecting the type of insulation best suited to your needs, remember that while each of the different types has a varying physical look and size, you can compare the insulating abilities by comparing the R-value. The R-value is a comparable measure of resistance to heat transfer - the higher the R-value, the greater the resistance and the better the product is at insulating, provided the insulation is properly installed. The insulation of ceiling and external walls is very cost-effective and is highly recommended, but make sure that all windows and doors are well sealed. A home with an insulated ceiling can still lose up to 25% of winter warmth and gain 20% of summer heat from unwanted air leaks and draughts alone. External wall insulation is recommended for most of Western Australia and is easiest to install when a house is being built. When insulating any type of wall, care must be taken to ensure that the insulation does not form a moisture bridge, allowing moisture from the outer wall to be transmitted to the interior.

3. LOW ENERGY MATERIALS AND METHODS FOR CONSTRUCTION

The choice of building materials also substantially contributes towards reducing the energy load of buildings. The use of conventional energy can be minimized by use of low energy materials, efficient structural design and reduction in transportation energy. Thus when building an energy efficient structure, it is necessary to closely examine the issues of building materials and to make appropriate decisions according to local conditions. Bamboo which is called the poor man's timber in India is also getting recognized globally as a suitable building material because it is eco-friendly and highly suitable for energy efficient buildings. Dense materials such as brick, stone, concrete and rammed earth heat up and cool down slowly - they have what is called a high 'thermal mass'. Lightweight materials such as weatherboard and fiber cement allow the home to heat up and cool down quickly. These materials have a low thermal mass. Thermal mass is simply the ability of a material to store heat. A 200 square meter home in the south west with good solar access to the north needs about 20 cubic meters of concrete and 20 to 30 cubic meters of internal brick or equivalent depending on your location (20 cubic meters for Geraldton and 30 cubic meters for Perth to Albany) to adequately store winter daytime warmth and gradually release it at night. Thermal mass is most beneficial in homes which have good solar access to north-facing windows. If solar access is limited, large amounts of thermal mass can increase your home's heating requirements during winter. In summer, thermal mass can also help keep your home cooler during the day, provided you properly ventilate your home overnight. The aim is to allow the night air to cool down the mass inside your home, resulting in more comfortable conditions the next day.

3.1 BRICK WALLS: Double brick walls heat up slowly and stay warm for long periods. This is an advantage during short periods of hot weather, but can make your home uncomfortable over extended hot spells. Insulating double brick walls will add to initial costs, but will help to prevent heat transfer to the interior of the home during summer and help to retain heat during winter. Brick veneer walls consist of a single external layer of brickwork, with a lined stud frame inside. These walls have less thermal mass than double brick walls and therefore respond more quickly to temperature changes. Homes with brick veneer walls are better at cooling down during extended periods of hot weather - making conditions more comfortable at night during summer. Brick veneer walls are also easier to insulate. Reverse brick veneer walls have the brickwork inside and lightweight frame and cladding outside. This has the advantage of providing the thermal mass on the inside of your home. With both double brick and brick veneer walls (or any type of wall for that matter), it is important to ventilate your home in summer once the temperature outside becomes cooler than the temperature inside. This will help cool your home down and make conditions more comfortable. Retained night time coolness achieved through ventilation can also keep your home cooler during the day.

3.2 LIGHTWEIGHT WALLS: Weatherboard, fiber cement and other lightweight walls get hot quickly in the sun, but also cool down quickly once shaded and after sunset. During winter, they lose heat far more quickly than brick walls. The thermal performance of lightweight walls will improve significantly with insulation, which is cheaper and easier to install at the building stage.

3.3 FLOORS: Concrete floors store heat from the sun shining through northern windows in winter and return some of that heat during the evening. Laying dark tiles where the low angle

winter sun hits the floor will maximize the absorption of heat to be re-radiated. It is important that this thermal mass is not exposed to direct solar energy during summer, as this can lead to uncomfortably warm internal conditions. Timber floors do not have the high thermal mass of concrete floors. This means that a home with a timber floor will lose far more heat than one with a concrete floor. For homes on stumps which are open at the sides, it is recommended that insulation be installed to the underside of all exposed floorboards. Another solution is to fully enclose the area between the ground and the floor with a solid material like brick, but this will not be as effective as using insulation. An enclosed space under the floor will also require some permanent ventilation to control sub-floor dampness.

3.4 COLOUR OF EXTERNAL BUILDING MATERIALS: As a general rule, light colors tend to reflect the sun's heat while darker colors absorb it. You can take advantage of this fact when selecting the color of your roof and wall materials. In summer, lighter colored materials will help to keep your home cooler by reflecting heat from the sun. However if your home is properly insulated, which is a much more effective method of controlling heat transfer, the effect of external building color on your comfort will be greatly reduced.

4. DRAUGHT PROOFING

Air leaks and draughts can add significantly to your heating and cooling bills by allowing cold air into your home during winter and warm air during summer. You can prevent these unwanted leaks by installing draught excluders on the bottom edge of doors and sealing strips around doors and windows. These are easy to fit and can be purchased from your local hardware store. When draught proofing you should also check for spaces between walls and skirting and block off any unused fireplaces. Note that homes with heaters that burn a fuel inside are required by law to have fixed ventilation for safety reasons.

4.1 VENTILATION: Doors and windows should be positioned to achieve cross ventilation in summer. The best distribution of cooling cross ventilation is achieved by flowing air in through an opening at least as large as the opening through which it leaves. If this has been allowed for in the design of your home, doors and windows opened late on a summer's day will make use of cooling late afternoon and night time breezes to rid your home of heat accumulated during the day.

5. LANDSCAPING

Gardens can provide significant climate modification effects, so designing your garden carefully can improve your home's comfort levels. Deciduous trees or vines which provide shade in summer but allow the winter sun to shine through when their leaves have dropped are an effective, simple option. Deciduous creepers can keep west-facing walls cool on hot summer afternoons. Shrubs or trees to the south can be placed to direct southwesterly sea breezes into and through your home. Plantings to the west and northwest can shield your home from winter storms, but avoid close plantings, which could cause damage. Un shaded paving to the north, east and west of your home should be avoided as it can cause heat to be reflected into windows during summer. Lawns and other ground covers will help reduce this problem. South-facing courtyards with moist cool ferneries will also assist summer cooling.

6. STUDY OF ENERGY EFFICIENT BUILDINGS

6.1 The RETREAT in India built as a model training complex with a built-up area of 3000 sq meters, demonstrates efficient utilization of energy, sustainable and integrated use of both natural resources and clean energy technologies, and efficient waste management systems. It has established that 'zero energy in - zero waste out' buildings could be created and run efficiently in the country. The building has been designed as south facing for the winter heat gains. Deciduous trees have been planted on the southern side of the complex which protects the building from the summer sun, and in winter, by shedding their leaves, the trees brighten up the rooms and provide the required heating. The south side of the structure is partially sunk into the ground to reduce heat gains and losses. Instead of the conventional grid electricity, the complex has a PV-Gasifier hybrid power plant. The biomass gasifier generates 50 kW power by burning twigs, branches, straw and crop residue and a 10 kW roof integrated PV system generates power from solar energy. To avoid conventional air conditioning methods, an earth tunnel system has been constructed that collects atmospheric air, cools it

under the earth, and then pumps it to the residential rooms to maintain a steady temperature. The air conditioning of the conference hall and other common facilities is achieved by using gas-fired ammonia absorption chillers, which consume 75 percent less electrical energy than a conventional system, as well as being cleaner and CFC (chlorofluorocarbon) free. Further, the use of 40mm thick expanded polystyrene insulation on the walls, and vermiculite concrete insulation on the roof, has brought down the space conditioning loads by about 15 percent. The lighting is provided by a combination of innovative day lighting techniques, such as skylights, the use of compact fluorescent lamps, high efficiency fluorescent lamps and lighting controls to reduce consumption. The building design and engineering has led to a reduced peak load of 96 kW, down from a conventional peak load of 280 kW, showing a saving of 184 kW. The efficient lighting system alone has resulted in a lighting load of only 9kW compared to the minimum requirement of 28kW in a similar conventional building. Solar hot water systems integrated into the parapet wall meet the entire hot water requirement for the building. The 'Root Zone' facility recycles wastewater with the help of pragmatism and the recycled water is used for gardening purposes. The annual energy savings by use of the passive design and efficient devices in the building is about 250 000 units of electricity and 2 100 LPG cylinders, valued at about 1.05 million Rupees per year.

6.2 The Mittal Residence; The Dr Yash Mittal residence at the foothills of the Catalina Mountains in Tucson, Arizona is an energy conservative and passive solar building designed by arch. Robert G. Hershberger.

A. Design Development: During the design development phase two major challenges had to be faced; first, since the site is located in the foothills, city view to the southwest became an urgent desire and thus a large panoramic window area was required. There was also a spectacular view up the mountain to the northeast, which was important to the client. Here a band of high windows was desired. Second, the combined effect of the site at a high mountain elevation and the severe cold winter nights, caused by the large temperature swing necessitated the use of additional thermal storage mass to provide heating at night. While the large cooling load is common to most residences in the Tucson area, the heating load, is higher than the norms. This is mainly because of the site location and the large window area (over 50% glass to floor area ratio).

B. Parametric Analysis: To achieve the goal of minimizing the cooling load and eliminating the need for auxiliary heat, various passive improvement strategies were first implemented. These strategies were: increased roof and wall insulation, double glazing, vented roof, shade trees, reduced glass area, overhangs, and high efficiency mechanical systems.

C. Thermal storage subsystem: Because the house-heating load was reduced through energy conservation strategies a carefully sized thermal storage wall subsystem was employed on the southwest facade. Since southwest is also the orientation for city view the thermal mass had to be placed under the collector. This innovative system will not block the main view and will provide most heating requirements through natural convection at night. The thermal mass is located behind and under the solar collector (double glass) and extends 3' 2" above floor level. Its volumetric heat capacity is estimated at 18,432 BTU/°F and consists of a 576 ft³ of grouted double CMU walls (12" thick each) running 24 feet long under the southwest collector. A charging fan running at a speed of 2 cfm/ft³ of mass and controlled by a conventional (non-differential) thermostat is used to charge the thermal mass walls located under the collector during the day. The southwest facing collector is protected by a 4' 3" horizontal overhang to reduce summer heat gain.

D. Diurnal Performance: In a typical diurnal performance the low-angle sun in winter reaches the collector glass during the day. The collected heat --as trapped between the double glass and the mass wall-- is then forced down by the fan to charge the lower mass. At night, two openings (2 ft² each) are opened to allow free convection from the thermal storage into the house while the cold air from the northeast side is pulled by convection through floor vents into ducts in the crawl space and back to the storage. In summer, the collector is opened to the outside to allow ventilation of the mass preventing any built-up heat.

E. Thermal Storage Capacity: The increased thermal mass and the addition of 2" rigid insulation on its outside surfaces have optimized the system performance.

6.3 The Hodges Residence: Designed by Mr. Carl Hodges and located at the east side of Tucson, the Hodges residence is a 2 bedroom, 1527 ft², one story single family detached house. The house is comprised of two main living zones, which are separated by a semi-enclosed inter zone. The west zone is the guesthouse with a kitchen, dining, bedroom and bathroom. This zone is entirely heated and cooled passively through a "Trombe Wall" and a

"Cool Tower". The east zone is the private master bedroom, a room converted into a walk-in closet and dressing area, and a bathroom. This zone is mechanically heated and cooled by a heat pump. The house is constructed with a 4" slab-on-grade and high mass 8" thick common double brick walls insulated from the outside with 1½" Styrofoam rigid insulation and stucco. The roof has 6" insulation covered in some parts with wood shingles and in other parts with asphalt rolls. Most of the windows are single glazed except the bathroom. The house is oriented about 2.4° east of south.

A. Parametric Analysis: A set of conservation strategies was applied to the design. The use of slab-edge insulation, double-glazing, movable shading devices, inside shutters, increased wall insulation, and optimized winter and summer thermostat settings were all strategies selected for the Hodges house.

B. Innovative and Technology Features: The west zone of the Hodges residence uses no mechanical heating or cooling devices as mentioned above. Instead, it uses a Trombe Wall system, which captures solar radiation from the south through its double-pane collector, and distributes the heat to the north via a ceiling duct. The heat then charges the double brick walls during winter days and releases that heat at night when it's mostly needed. Another important cooling device is a "Cool Tower". The owners recently added the tower. The tower is a combination of two methods of making life comfortable in hot, dry climates. Traditional Middle Eastern architecture uses tall towers to funnel passing breezes down to a courtyard. Often ponds or fountains are built in the enclosed area to further cool the air through evaporation. These cooling devices use a fan to disperse a cool water vapor. The cool tower is essentially an evaporative cooler on top of a tower; but it differs in that no forced air is needed to promote evaporation or circulation, and no passing breezes are necessary to generate a cooling draft. The hydro unit that caps the tower structure is a cube with cellulose pads exposed on four vertical sides. Water is conveyed through hoses up the tower to the hydro unit where it saturates the pads. As air comes into contact with the wet pads the water evaporates. The exchange of energy between the air and the water results not only in vaporized water, but also in cooler air. The cool air is now denser than the surrounding air, and it falls down the tower creating a breeze that escapes out of a vent at the bottom. The falling air pulls more warm air through the pads perpetuating the evaporation process. Troughs around the base of the hydro unit catch excess water, and a pump recirculate the runoff water through the pads.

7. CONCLUSION

With increasing degradation of the environment because of increased energy consumption, environment conscious building design has become urgent. The benefits of green design to society in general, and building owners and users in particular, are manifold. The construction of such buildings results in reduced destruction of natural habitats and bio-diversity, reduced air and water pollution, less water consumption, limited waste generation and increased user productivity. The cost differential between passive and conventional systems is hard to determine, as passive elements are an integral part of the building architecture. However, it is believed that passive design could prove to be a cost effective solution and should not cost more than 15 to 20 percent of the total building cost. With the active cooperation of architects, engineers, builders and policy makers, we can save energy.

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**Evaluating the Successful Sustainable Outcome Criteria in the AEC industry
using Analytic Hierarchy Process**

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Abstract

The owners' commitment level to the completion of the sustainable project is imperative for implementing sustainability in the Architecture, Engineering, and Construction industry. Besides the owners, the architects, engineers, and contractors also play a great role in the construction process. It was hypothesized in this paper that the architects, engineers, and contractors believe that the quality of the completed structure is essential for maintaining owners' level of commitment to sustainability. This paper presents the results for the survey on evaluating the successful sustainable outcome criteria so that practical experiences from their point of view on sustainability are obtained. Respondents were asked to answer which attribute has greater influence on the attribute one level higher in the hierarchy and to what extent they prefer. Twenty completed surveys were collected and the analytic hierarchy process (AHP) was applied to analyze the data. The results indicate that the quality of the finished product is the most significant effect on sustainability because the overall performance after completion is the most important factor when designing and building a facility.

Keywords: Owner commitment, sustainability, AEC industry, AHP, Survey

INTRODUCTION

Design and construction of green buildings (also known as sustainable and high performance buildings) is dramatically increasing in recent years. Sustainable design and construction has emerged nationally and internationally as a major paradigm for saving our earth in the future. For the non-construction industry, governmental agencies already made a progress to address the aspects of sustainability to govern issues like land use and soil conservation. Since the concept of sustainability is still broad to define in scope, the built environment still has a difficulty to achieve sustainable goals in reality. The built environment consumes about 40 percent of extracted resources in most industrialized countries, as well as consumes approximately 40 percent of all primary energy (Kibert 1999). According to the U.S. Green Building Council (USGBC 2007), buildings are responsible for 38% of carbon dioxide emissions, 71% of electricity consumption, 39% of energy use, 12% of water consumption, and 40% of non-industrial waste in the U.S. Green building has developed as a practical answer to the environmental and health burdens of the built environment, which causes significantly negative impacts on the natural environment and on human health. USGBC (2007) recommends four challenging areas for the success of green buildings. They include (1) project delivery process and performance evaluation, (2) integrated building systems, (3) buildings' interaction with local environments, and (4) buildings' interaction with occupants. Kibert (1994) emphasizes the principles for sustainable construction, which include the minimization of resource consumption, maximization of resource reuse, usage of renewable or recyclable resources, protection of the natural environment, creation of a healthy, non-toxic environment, and quality in creating the built environment. In addition to the public sectors, private developers have recognized the value and importance of green buildings in terms of costs and public relations. Identifying successful sustainable outcomes in the Architecture, Engineering, and Construction (A/E/C) industry is required to meet the need for the implementation of sustainability.

A recent research found that the most important criterion for achieving sustainable outcomes for construction projects is the owners' commitment to sustainability (Beheiry et al. 2006). Besides the owners, the architects, engineers, and contractors also play a great role in the construction process. Many architects, engineers, and contractors are using largely existing methods, tools, and technologies to design and build facilities by lowering environmental impact, reducing resource consumption, and improving interior environments. With the assumption that a certain level of the owners' commitment to sustainability is maintained, it is necessary to examine the point of view of the architects, engineers, and contractors to consider what kinds of criteria for the successful implementation of sustainability they recognize and to what extent they prefer.

The main objective of this paper is to evaluate major criteria essential for identifying successful sustainable outcomes for the construction projects from the point of view

of the architect, engineer, and contractor. It was hypothesized that the A/E/C believes that the quality of the completed structure is essential for maintaining owners' level of commitment to sustainability. Throughout the information gathering and generation of feasible alternatives, the research team created five evaluation criteria associated with three alternatives supporting each main criterion, which may significantly affect the owners' decision making for the commitment to the sustainable development. In order to achieve the research objective, we established four sub-objectives as follows:

- Compare each of the five main criteria,
- Compare three alternatives supporting only each main criterion directly,
- Examine the overall preference of one alternative over another by aggregating the findings obtained from the survey, and
- Examine the current and future trends in the sustainable development of the construction projects.

This paper begins with the description of the sustainability outcome criteria in the AEC industry, followed by the survey and data section that contains the development of a structured questionnaire and the implementation of the survey. Data collected from the survey are analyzed using Analytic Hierarchy Process (AHP) technique. The paper then presents the analysis results and summarizes the research findings and limitations.

SUSTAINABILITY OUTCOME CRITERIA IN THE AEC INDUSTRY

Sustainability can be defined as creating designs and building a facility that seeks to balance the short-term goals of a project with the long-term goals of efficient operating systems that protect the environment and nature's resources (Levy 2007). This section describes the sustainability outcome criteria that will have a significant impact on the decision making process of construction project owners in order to leverage the level of the commitment to the sustainable development in their potential project. Successful sustainable outcomes considered in this paper are broken into five criteria associated with three alternatives that directly support the criterion immediately above them. Figure 1 shows the hierarchy for successful sustainable outcomes for the survey. The evaluation criteria in Level 1 to implement the sustainable development successfully include the appropriate selection of delivery method (SR-1), innovation and creativity (SR-2), health and safety (SR-3), materials and resources (SR-4), and the quality of the finished product (SR-5).

The appropriate selection of delivery method (SR-1) includes design-bid-build (A), construction management for fee/at risk (B), and design-build methods (C). A project delivery method can be defined as a contractual arrangement that exists between the primary stakeholders such as the owner, the designer, and the constructor (Kelleher 2005). In this paper, we considered three delivery methods such as design-bid-build project delivery, construction management for fee/at risk, and design-build methods as

the feasible alternatives for supporting the appropriate selection criterion to the successful sustainable development of the construction projects.

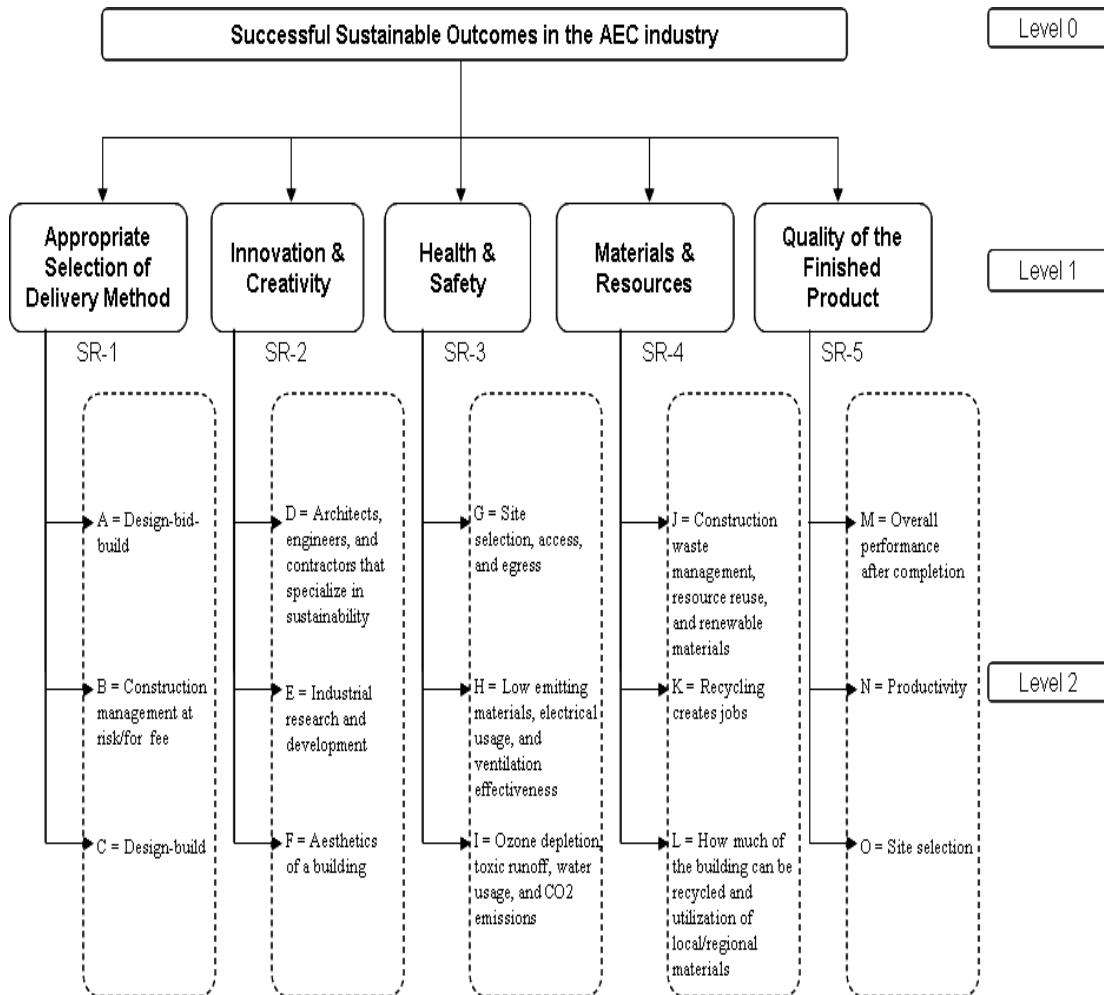


Figure 1. Hierarchy for successful sustainable outcomes

Innovation and creativity (SR-2) has three attributes, which are architects, engineers, and contractors that specialized in sustainability (D), industrial research and development (E), and aesthetic of a building (F). Architects, engineers, and contractors that specialize in sustainability indicate private firms that solely base their work around sustainable structures. Industrial research and development (R&D) include all materials producers and owners who fund research for sustainable structures. The aesthetics of a building is how the building looks or how it pleases the eye.

Health and safety (SR-3) consists of site selection, access, and egress (G), low emitting materials, electrical usage, and ventilation effectiveness (H), and ozone depletion, toxic runoff, water usage, and CO2 emissions (I). Site selection under the health and safety criterion includes choosing a job site that is close to local resources, or a refurbished plot of land, e.g. a building built on clean fill. Building access and

egress include how accessible a building is to the public, egress would be how easy it is to exit a building in the case of an emergency. Low emitting materials include materials that do not emit much toxic particles into the environment. Electrical usage includes using light fixtures and HVAC units that are extremely efficient. Ventilation effectiveness includes how easy a building can be ventilated without much help from the HVAC units. Ozone depletion includes buildings that do not pose a threat to the Earth's Ozone layer. Toxic runoff includes a building's capability of returning all of its waste back to nature without harming it. Water usage includes buildings that are extremely efficient at using water, or they may collect and use rain water. CO₂ emission includes a building's ability not to emit much CO₂ into the atmosphere.

Materials and resources (SR-4) include construction waste management, resource reuse, and renewable materials (J), recycling creates jobs (K), and how much of the building can be recycled and utilization of local/regional materials (L). Construction waste management includes the contractor's ability to successfully manage their waste in a sufficient way as to not harm the environment. Resource reuse includes a building that uses recycled materials during construction. Renewable materials may include materials that can renew themselves such as wood. Recycling can create jobs because if a building really supports recycling all of its waste, it will affect the employment within the community. How much of the building can be recycled includes building a building out of recyclable materials. The utilization of local and/or regional materials includes obtaining materials for construction that are close by, thus cutting down on shipping costs and the over use of fossil fuels.

Finally, the quality of the finished product (SR-5) has three attributes of overall performance after completion (M), productivity (N), and site selection (O). The overall performance after completion of a building includes how overall efficient a building is after it is completed. Productivity includes how much more productive the people working inside of the sustainable structure are compared to a regular building. Site selection in this context is different from the site selection under the health and safety (SR-3) in that it includes the location of a job site that has good underground conditions and working conditions in terms of the construction operation. The following section describes how a structured questionnaire was developed and employed for the survey.

SURVEY AND DATA

In order to achieve the objectives of this research, a structured survey form was developed using the hierarchy for successful sustainable outcomes shown in Figure 1. The survey form was designed in three sections, which consist of five main criteria comparison section, 15 alternatives comparison section, and three questions section. The last three questions section aimed to enable us to acquire the knowledge and experiences of the experts on the current and future trends for the sustainability in the AEC industry. After validating the survey form with a professional architect, its final

version was issued. The structured survey form was especially designed to conduct a survey via E-mail and fax communication.

Data were collected on the final version of the survey form to compare the relative importance of five main criteria as well as three alternatives supporting each of the five criteria. Table 1 shows a sample pair-wise comparison questionnaire. Participants were asked to compare which attribute is more important or has greater influence on the attribute one level higher in the hierarchy and to what extent they prefer.

Table 1. Sample Pair-wise Comparison Questionnaire

Compare the relative IMPORTANCE with respect to "Successful Sustainable Outcomes in the AEC industry"																			
1 = EQUAL		3 = MODERATE					5 = STRONG					7 = VERY STRONG					9 = EXTREME		
1	SR-1	9	8	7	6	5	4	3	2	1	2	3	4	5	6	7	8	9	SR-2
2	SR-1	9	8	7	6	5	4	3	2	1	2	3	4	5	6	7	8	9	SR-3
3	SR-1	9	8	7	6	5	4	3	2	1	2	3	4	5	6	7	8	9	SR-4
4	SR-1	9	8	7	6	5	4	3	2	1	2	3	4	5	6	7	8	9	SR-5
5	SR-2	9	8	7	6	5	4	3	2	1	2	3	4	5	6	7	8	9	SR-3
6	SR-2	9	8	7	6	5	4	3	2	1	2	3	4	5	6	7	8	9	SR-4
7	SR-2	9	8	7	6	5	4	3	2	1	2	3	4	5	6	7	8	9	SR-5
8	SR-3	9	8	7	6	5	4	3	2	1	2	3	4	5	6	7	8	9	SR-4
9	SR-3	9	8	7	6	5	4	3	2	1	2	3	4	5	6	7	8	9	SR-5
10	SR-4	9	8	7	6	5	4	3	2	1	2	3	4	5	6	7	8	9	SR-5
Abbreviation		Definition																	
SR-1		Appropriate Selection of Delivery Method																	
SR-2		Innovation and Creativity																	
SR-3		Health and Safety																	
SR-4		Materials and Resources																	
SR-5		Quality of the Finished Product																	

The research team conducted a survey on the successful sustainable outcomes with professional experts in the AEC (architecture, engineering, and construction) industry. This research was limited to both “Engineering News Record; the top 50 companies” from each of Design-build, Construction Management for Fee, and Construction Management at Risk companies and general construction contractors. These companies are more likely capable of conducting sustainable practices because they are willing to execute larger construction projects.

Lists of companies were obtained from Engineering News Release webpage. The top 50 companies from each of Design-build, Construction Manger for Fee, and Construction Manager at risk were extracted for the survey as a starting point. With the aid of the internet, phone numbers were found to correspond with the 150 companies and compiled into three phone contact lists. The lists were split between each research team member. The survey was completed either by phone, E-mail, fax or by personal contact. The research team made phone calls to find the appropriate

personnel who pursues his/her careers in the area of sustainable development in their companies in addition to the several years of working experiences in the AEC industry, and to obtain the completed questionnaires back from the respondents via E-mail or fax. Twenty completed surveys were collected during June 25 – July 23, 2007 from the AEC companies in the U.S.A. All pair-wise comparison data obtained from 20 participants were analyzed using the Analytical Hierarchy Process (AHP).

ANALYTIC HIERARCHY PROCESS

The research team used the analytic hierarchy process (AHP), which is one of multi criteria decision making methods and originally developed by Satty (1980), to analyze the data throughout six steps.

Step 1: Hierarchy Construction

As the first phase of the AHP-based criteria evaluation, the research team broke down the overall goal of the project into its constituent parts. The research team started the process by determining five major criteria (Level 1) for identifying successful sustainable outcomes in the AEC industry, which is the highest level (Level 0). Associated with each criterion, three attributes were decided to only support the criterion immediately above them, which constitutes the common basis for the pair-wise comparisons, as previously shown in Figure 1. The goal of initializing the hierarchy construction was to set up all the levels according to how specific the subject was. The initialization was accomplished so that more specific items become less important than the criteria further up in the hierarchy (Grandzol 2005; Teknomo 2006).

Step 2: Pair-wise Comparisons

Relative weights of the attributes were determined by comparing them in pairs using the interrelationships between attributes. Twenty reciprocal matrices using the twenty respondent data obtained for the leading question No. 1, which is “What attribute has a greater influence on Successful Sustainable Outcomes, and to what extent the attribute in row i or the attribute in column j ?,” in order to compute the relative importance. Figure 2 shows one of twenty reciprocal matrices along with the AHP model implementation procedures.

One hundred reciprocal matrices (20 respondents x 5 comparisons in Level 2) were also developed in Step 2. One of the questions in the comparisons of No. 2 is “For successful sustainable outcomes, which attribute has a greater influence on appropriate selection of delivery method, and to what extent the attribute in row i or the attribute in column j ?.” Figure 3 shows one of the examples that captured screen of pair-wise comparison for Level 2.

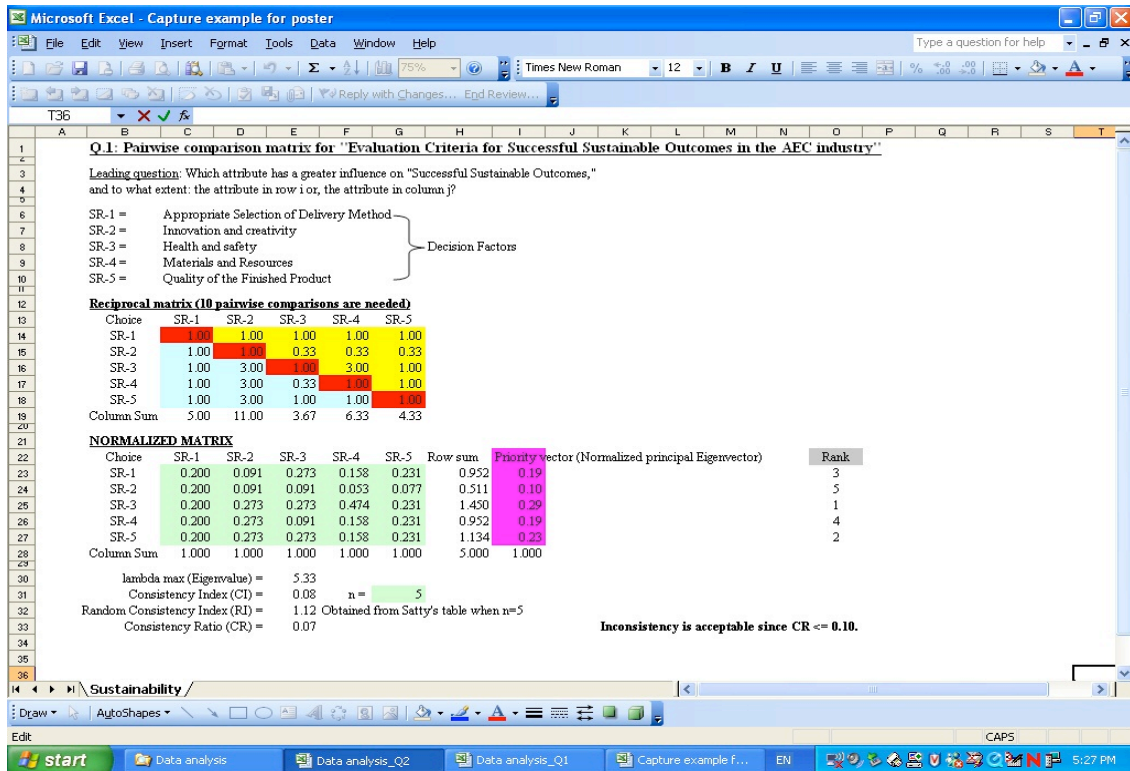


Figure 2. Model implementation: screen capture of example pair-wise comparison for Level 1 (Q1)

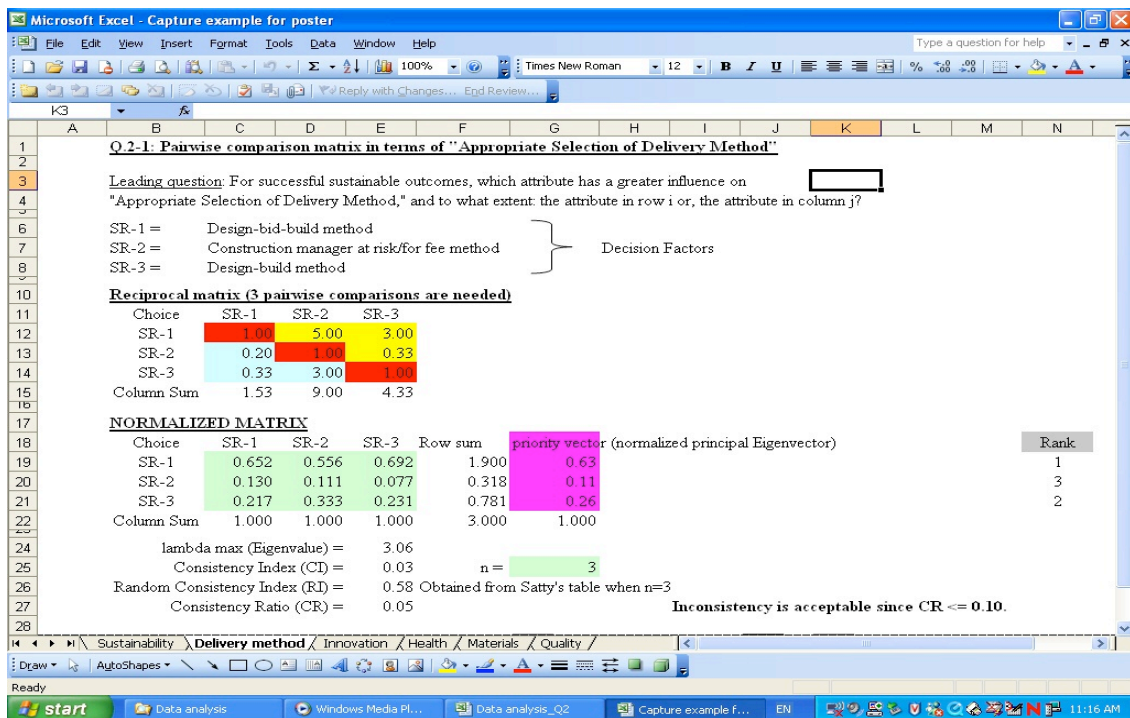


Figure 3. Model implementation: screen capture of example pair-wise comparison for Level 2 (Q2-1)

Step 3: Relative-Weight Calculation

The average of normalized columns (ANC) method was applied to compute the eigenvector of the decision matrix (Satty 1980; Shapira and Goldenberg 2005; Teknomo 2006). Figures 2 and 3 also illustrate the ANC calculation process in detail for pair-wise comparisons of Levels 1 and 2, respectively.

Step 4: Aggregation of Relative Weights

The overall scores of each alternative, indicating the preference of one alternative over another, were calculated, in addition to the scores for the pair-wise comparisons at each level.

Step 5: Consistency Ratio

The consistency ratio (CR) measures, the last element of the AHP, were computed for each reciprocal matrix. The CR, which must be less than or equal to 0.10, controls the consistency of pair-wise comparisons. The CR was calculated from the consistency index (CI) divided by random consistency index (RI). Table 2 shows the random consistency index that is obtained from Satty’s table (1980).

Table 2. Random Consistency Index (RI)

n	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
RI	0.00	0.00	0.58	0.90	1.12	1.24	1.32	1.41	1.45	1.49	1.51	1.48	1.56	1.57	1.59

Step 6: Determination of Ranking Criteria and Attributes

Five criteria were ranked by allocating the weights of five to one for the first rank and the fifth rank and then averaged all scores to determine the overall rank for five criteria, three attributes under each criterion, and the overall preference of combining all of the scores.

ANALYSIS RESULTS

Application of the AHP methodology, which includes (1) constructing hierarchies, (2) establishing priorities, and (3) verifying logical consistency, brought four results that correspond to each sub-objective as the findings of this research effort. Data collected from this research were only used to present the results. Figure 4 shows the final rating of the five evaluation criteria in Level 1. The result indicates that the quality of the finished product (SR-5, 26.67%) is the most important criterion among five main criteria for successful sustainable outcomes, followed by health and safety (SR-3, 22.22%), and then materials and resources (SR-4, 21.67%). Figure 5 shows the final rating of the three attributes that only supports each of five evaluation criteria in Level 2. The result explains that the design-build delivery method (A, 36.81%), AEC that specialize in sustainability (D, 46.53%), Ozone depletion, toxic runoff, water usage and CO2 emission (I, 36.81%), Construction waste management, resource reuse, and renewable materials (J, 42.36%), and Overall performance after completion (M, 42.36%) were found to be more important than other attributes under each criteria in level 1.

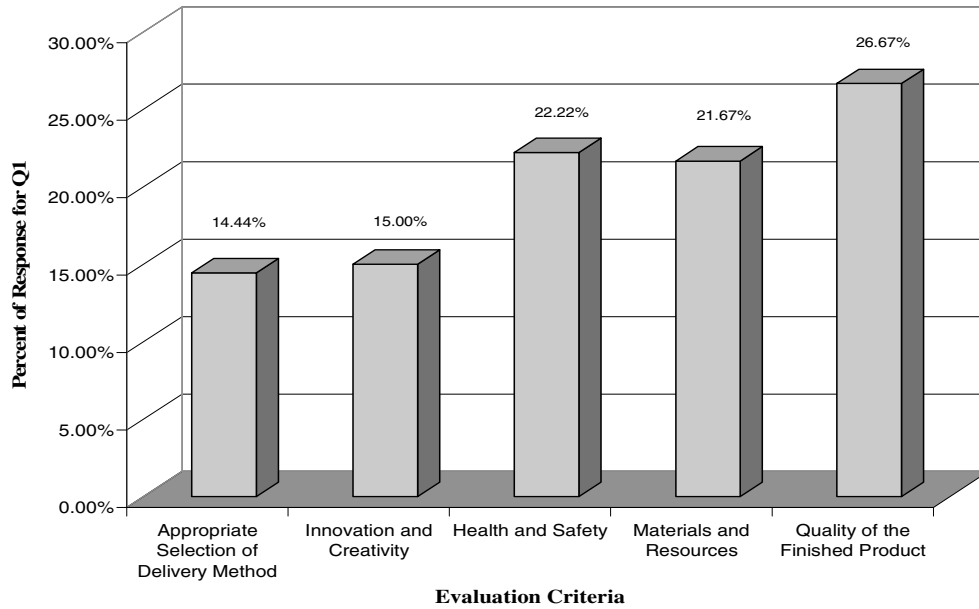


Figure 4. Final ratings of evaluation criteria on Level 1

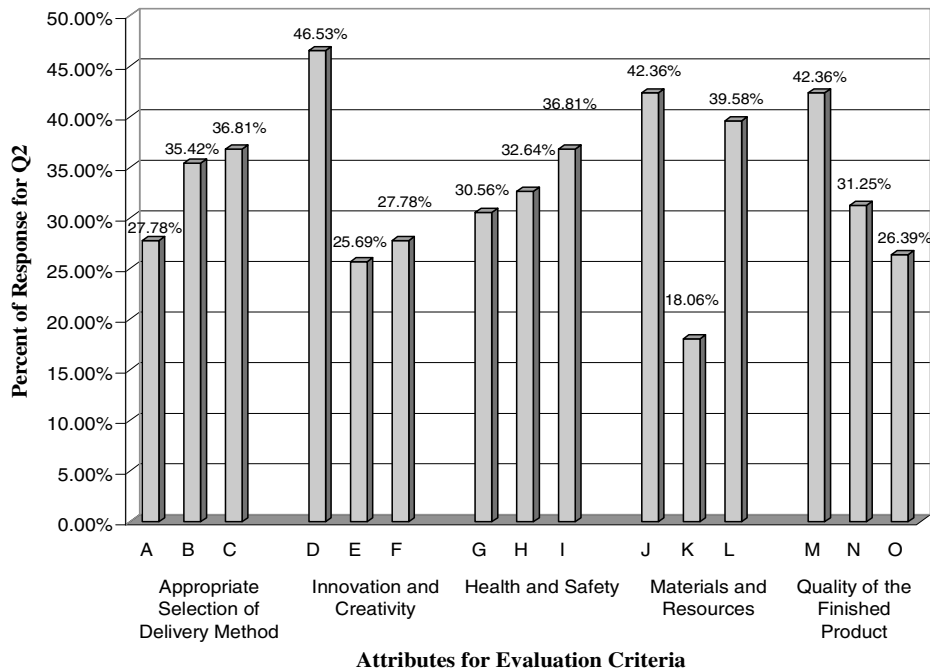


Figure 5. Final ratings of attributes for each evaluation criteria on Level 2

Both Table 3 and Figure 6 show the overall preference of one alternative over another. Aggregating the relative importance using the first and second results indicates that the overall performance after completion, which directly supports the quality of the final product criterion, is the highest preference ($0.267 \times 0.424 = 11.3\%$).

Table 3. Preference Structure with Priorities *

Level 0	Successful Sustainable Outcomes in the Architect/Engineering/Construction Industry																			
Level 1	Appropriate Selection of Delivery Method			Innovation and Creativity			Health and Safety			Materials and Resources			Quality of the Finished Product							
%	0.144			0.150			0.222			0.217			0.267			1				
Level 2	Design-Bid-Build	Construction Manager at Risk/ for Fee	Design-Build	Architects, engineers, and contractors that specialize in sustainability	Industrial research and development	Aesthetics of a building	Site selection, access, and egress	Low emitting materials, electrical usage, and ventilation effectiveness	Ozone depletion, toxic runoff, water usage, and CO2 emisson	Construction waste management, resource resue, and renewable materials	Recycling creates jobs	How much of the building can be recycled and utilization of local/regional materials	Overall performance after completion	Productivity	Site selection					
Index	A	B	C	D	E	F	G	H	I	J	K	L	M	N	O					
%	0.278	0.354	0.368	1.00	0.465	0.257	0.278	1.00	0.306	0.326	0.368	1.00	0.424	0.181	0.396	1.00	0.424	0.313	0.264	1.00
Overall	0.040	0.051	0.053	0.070	0.039	0.042	0.068	0.073	0.082	0.092	0.039	0.086	0.113	0.083	0.070	0.100	0.113	0.083	0.070	0.100

* Level 1 criteria sum to 1; Level 2 attributes sum to 1 within each level 1 criterion; Overall preferences sum to 1

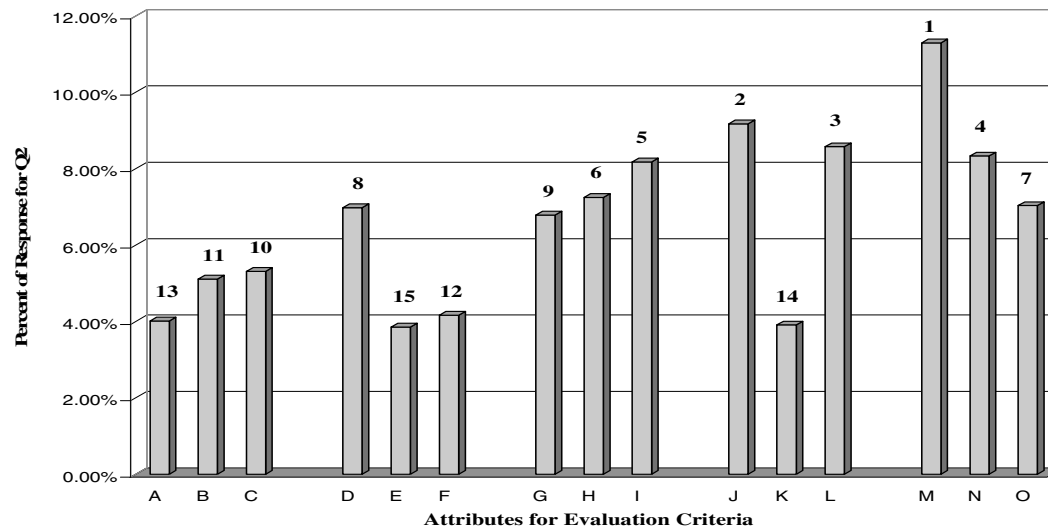


Figure 6. Final ratings on overall preferences

Most of respondents recognized sustainability as meeting today's needs without compromising the ability of future generations to meet their needs. Unlike the public sector, the private sector is willing to spend much less amount of money to meet green building standards. With a benefit from energy efficiency over the life cycle of a facility, the construction cost for green buildings can be equal to or even less than conventional construction.

CONCLUSIONS AND LIMITATIONS

This research project dealt with the evaluation criteria necessary for identifying successful sustainable outcomes for the AEC industry based on the viewpoint of the architects, engineers, and contractors. From the analysis of data obtained in this study, AEC recognized that the quality of the finished product is the most significant effect on sustainability because the overall performance after completion is the most important factor when designing and/or building a facility. The research findings also provides an alternative to complex judgments by categorizing priorities among criteria and also help the owners to make a decision on commitment to sustainability by applying the major criteria into their future construction project.

Modification and redistribution of the current survey form is required because some of data showed that the consistent ratio (CR) is not acceptable due to either the lack of the thorough understanding and knowledge for the AHP survey of respondents or the lack of the sufficient materials for decision makers, even though AHP allows some small inconsistency in human judgment. Collecting more data to support the research findings will improve the consistency and accuracy of the survey results.

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ACKNOWLEDGMENT

This research was supported by 2007 Summer Research Institute from the Missouri Western State University. The authors would like to thank all of the respondents for their cooperation and support in carrying out this study.

Greening of Healthcare Facilities: Case Studies of Children's Hospitals

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Abstract:

Healthcare is one of the most significant built environment markets today with over 120,000 buildings in the United States. Moreover, in the next 15 years, an estimated \$300 billion will be spent on hospital construction across the United States; therefore, there is a great opportunity for research and improvements to be made in this area. The built environment has a great impact on healthcare, particularly on the health, safety and well-being of patients and staff. Green healthcare offers benefit of improved clinical outcomes, reducing operating costs, energy consumption and water use. Children's hospitals are the most specialized centers for care, providing care for children with complex and rare conditions and they are therefore amongst the most complex types of facilities to design, construct and operate. This paper outlines the findings from three children's hospitals with different levels of sustainability in regards to the project delivery process, greening strategies and lean principles. This paper will provide a thorough understanding of the building delivery process in green children's hospitals, starting from programming, through design, construction, operations and maintenance. Using the Dell Children's Hospital of Austin, the Children's Hospital of Pittsburgh and the Hershey Medical Center Children's Hospital, this paper will show how coupling lean and green can make healthcare facilities not only efficient, but healthful places for treatment.

Keywords:

Sustainability; healthcare design and construction; process; children's hospitals

1. Introduction

The sheer size and continued growth of the healthcare industry provides an important opportunity to dramatically impact how the built environment affects human health and well-being. In the U.S., healthcare is a \$1.4 trillion industry, accounting for 13.2 % of the entire U.S. gross domestic product (GDP), with projected growth to \$2.8 trillion, or 17 % of GDP by 2010 (Frampton 2003), being the world's largest. Healthcare is one of the most significant built environment markets today with over 120,000 buildings in the United States. Moreover, in the next 15 years, an estimated \$300 billion will be spent on hospital construction across the United States (Ulrich 2004).

The built environment has a great impact on healthcare, particularly on the health, safety and well-being of patients and staff. Paramount, of course, is that healthcare facilities are places for treating and healing humans. Unfortunately, however, healthcare facilities are also responsible for inducing health problems: Hospital-acquired infections are one of the leading causes of death in the U.S., killing more people than AIDS, cancer or automobile accidents

(Institute of Medicine 2001). These hospital-acquired infections are mostly due to environmental factors. The design and construction of a hospital environment can have detrimental effects on patients and staff. For example, research has shown poor ventilation and indoor air quality are major causes of nosocomial infections. Improper lighting is correlated with patient depression and medication errors. Excessive noise upsets patients and causes increased stress and lack of sleep.

A promising area of research and action to combat the stressors that affect patient and worker health and well-being has been the use of “green” or “environmental” strategies. Green healthcare also offers benefit of reducing operating costs, energy consumption and water use. These are important given that healthcare facilities use complex equipment to run without cross-contamination and transfer of harmful bacteria that use large amounts of energy as a consequence (Phelps 2005). Hospitals have very high energy uses and energy bills due to the fact that some of their facilities are run 24 hours a day. In addition, the healthcare industry is also one of the most significant consumers of resources and producer of waste, and, therefore, has a large ecological footprint (Frampton 2003). For these reasons, it is also important that healthcare facilities are constructed, and operated appropriately in order to demonstrate that they are a source of health for our communities.

Children’s hospitals are the most specialized centers for care, providing care for children with complex and rare conditions. Therefore, they are amongst the most complex types of facilities to design and construct amongst complex facilities. Moreover, children’s hospitals serve as research centers and promote health protection by addressing issues such as child abuse, child obesity and injury prevention.

2. Objective

Research has shown that there are significant benefits in incorporating sustainable and lean principles in the construction industry. At the moment there are major challenges to incorporating these strategies in the healthcare building industry, more so than the commercial and residential building industry. Children’s hospitals are a type of healthcare facilities where the consequences of poor building design, construction and operations have the potential to affect the lives and health of the children and staff. In short, they are the most complex facilities to procure, design and construct. To this end, this paper seeks to provide an understanding of the building delivery process in green children’s hospitals, starting from programming, through design, construction, operations and maintenance. An emphasis is placed on how the delivery process, the stakeholders present in each project and also the project environment affect the final product. Understanding the delivery process is the first and most important step in facilitating the construction of more green facilities, and in reducing the challenges that come with it at the moment. Process modeling is used as a crucial element in understanding this delivery process.

3. Background

Unlike regular hospitals, in children’s hospitals planners and designers are challenged to accommodate adults and also children of all ages. Children’s hospitals are one of the most challenging types of buildings to design and construct. Hospitals must be calming environments for the children and their families in order to aid the healing process.

Children are a unique end-user, especially in an environment such as healthcare, which supports their healing. Designers and constructors have to create children’s hospitals that are playful and provide a supportive environment for the children, while performing technically and functionally.

Given the extra complexities and needs in children’s hospitals, a research focus on the design and construction of children’s hospitals is likely to have a major impact on the delivery of all healthcare facilities. The patient-friendly successes in children’s hospitals will influence all other healthcare facilities, and therefore set the standard for the next generation of hospitals and buildings in general.

4. Research Methodology

To investigate this problem, this research will use detailed case study research to investigate the differences that exist in the building delivery processes for each of a LEED Platinum, the Dell Children’s Hospital of Austin ant two LEED certified, Children’s Hospital of Pittsburgh and Hershey Children’s Hospital. The case studies provide a wide range of data, and the comparison between two moderate and a highly green project will be beneficial for the design and construction industry.

4.1 Case Studies

4.1.1 The Dell Children’s Hospital of Austin

The Dell Children’s Medical Center of Central Texas is located on 32 acres of the old Robert Mueller Municipal Airport. It is four-stories, containing approximately 480,000 square feet. The hospital also has a Healing Garden, totaling 3 acres, that is located on the south side. The previous Children’s Hospital of Austin located downtown Austin could not be extended; therefore the new facility took into consideration the planning for future generations, creating an environment that can grow through time. (Dell 2007). The project was fast-track, being on an aggressive 27 month schedule, and it opened in June 2007. The hospital is on track for being LEED Platinum, which will be the first LEED Platinum hospital in the nation. The children’s hospital uses sustainable building practices as seen in Teble 1 (Dell 2007):

Table 1: Sustainable Building Practices at Dell

Sustainable site planning	The site is part of the City of Austin’s Smart Growth Initiative
Water Conservation	The facility contains a rainwater collection system and is xeriscaped for water efficiency
Energy Efficiency& Energy Plant on site	Heat recovery systems and high efficiency equipment Seton is building a District Energy Plant on site which provides power and chilled water and steam to the hospital and the surrounding areas.
Conservation of Materials and Resources	Use of Recycled Materials and low VOC Minimize construction waste
Indoor Environmental Quality	Optimize natural daylight, eliminate airborne pollutants

The hospital is located in a brownfield development, adding to the list of LEED points. The runway of the Muller airport was demolished and recycled. 35,000 tons of asphalt were used as a base for the parking lots. Moreover, 41,000 cubic yards of high volume fly-ash concrete were used for the foundation and walls which added to the project’s LEED points. About 75% of all waste is being recycled. The roof of the facility is TPO single-membrane with standing-seam metal in a few areas. The roof reflects sunlight instead of absorbing radiant energy. Other LEED points achieved in the sustainable site section include credit for urban redevelopment and a rain and ground-water collection system for irrigation. Additional features include carbon dioxide monitoring and sealing the ends of the ductwork during construction. Water-efficiency points were earned by using low-flow toilets and fixtures and native plants in the landscaping. The construction cost for the facility is estimated to be around \$110,000,000, while the 35,500 combined cooling-heating power plant is estimated to have a construction cost of approximately \$18,000,000.

The decision to go green and pursue LEED Platinum has been aligned with the corporate mission from the start of the project. Austin’s Green Initiative is another reason for going green. The owner also looked in depth at the life-cycle cost analysis and it was discovered that the facility would have a 5.9 year payback. Lastly, one of the factors for pursuing green was to “improve the quality of life for all”.

4.1.2 Children’s Hospital of Pittsburgh

The New Children’s Hospital of Pittsburgh is located in Pittsburgh’s Lawrenceville neighborhood on 10 acres -1.5 million square feet of usable space. Te actual hospital is 900,000 square feet with 262 beds. The cost of the hospital is approximated at \$555million. The hospital is under construction on the urban campus of the former hospital, which will benefit the new hospital by using some of the assets of the previous hospital.(Pittsburgh 2007)

The hospital has been designed with input from physicians, nurses and families in order to inspire transformation to all who pass the doors. The hospital has been designed with the children in mind from the very beginning and it is committed to family centered care. The hospital is on track to be LEED certified. The hospital is grounded on five principles shown below in Figure 1:

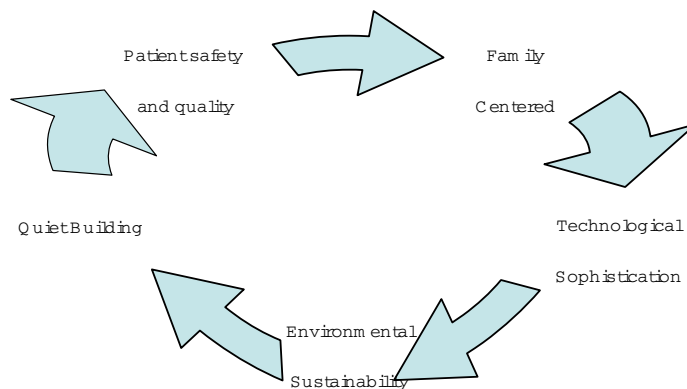


Figure 1: Grounding principles at the Children’s Hospital of Pittsburgh

4.1.3 Penn State Hershey Children's Hospital

Penn State Children's Hospital is part of the Penn State Medical Center, the only medical school and university hospital in Pennsylvania located outside the urban areas of Philadelphia and Pittsburgh. The present Penn State Children's Hospital is the only children's hospital in central Pennsylvania and has the region's only LEVEL III, state-of-the-art neonatal intensive care unit (NICU). The hospital is well-known for the following specialties: neonatal care, pediatric oncology, pediatric cardiology, pediatric surgery and pediatric trauma.

The new Children's Hospital will be approximately 323,500 square-feet. The hospital will have a total cost well in excess of approximately \$270 million dollars and is seeking to raise \$65 million in philanthropy.(Hershey 2007). The proposed new seven-story Children's Hospital and adjacent Cancer Institute (which is currently under construction) will make up the new face and main entrance to the entire hospital. The conceptual design of the hospital was developed through business case studies, interviews with various user groups, visits to other children's. All new construction Penn State facilities are mandated to be LEED certified and this will be one of the.

4.2 Data Collected

A comparative case study approach is used to compare certain features of the hospitals such as the LEED level achieved and the reasoning behind going after the specific points, incorporation of a "charette" system, sequence and timeline of each step in the delivery process, owner and other stakeholders involvement, costs associated with each step in the delivery process and percentage of total cost, life-cycle cost analysis if available, an evaluation of the green design feature for each hospital, integration of technical systems, satisfaction of users with green features.

Data is collected through the following methods:

1. A series of meetings, phone and email **interviews** with the stakeholders in each project. Most of the data will be collected using this method. Specific questions will be asked regarding LEED points achieved and the reasoning behind each, sequence of processes and time spent for each, lean principles incorporated.
2. **Project materials:** design documents, meeting minutes, bidding documents obtained from the stakeholders. These materials will be primarily used to obtain an overview of the case studies and to follow their development. They will supplement the interview data collection.

After an extensive literature review and recommendations from industry representatives and academic researchers, the issues identified in Table 1 are selected to be most important in the delivery of a successful green, LEED certified building. The stakeholders for each project, owner's representative, architecture firm and construction manager, were interviewed based on the issues below in Table 2:

Table 2: Data Collection Tool

Broad key green process	Specific attributes	Data to be collected
<i>Transparent delivery process in relation to green outcomes and an integrated approach</i>	Integrate sustainable project objectives well with other delivery aspects during programming, design and construction	Were all project members aware of the green outcomes? Good communication between stakeholders?
	Sustainable attributes are not singles out as additional requirements	Was there any rework or delays due to poor understanding of green strategies?
	Thoroughly communicate the project's sustainable attributes to each project member	Were eco-charettes conducted? How often and who participated? Were all workers utilized during the project as the sustainability issues were worked through?
<i>Early adoption of green</i>	This enables a more clear understanding of project scope requirements and project needs	When was the notion of green first introduced? At what point in the delivery
	Saves project rework due to accurate project bids and costs	Who proposed the notion of green? (owner, architect, design-builder)
	Saves time for incorporating sustainable objectives later in the project	Was the team trained on these sustainable objectives?
	A sustainability filter can be applied to all decisions	Were green objectives discussed in relation to overall project goals and objectives?
<i>Business case for green initiatives</i>	Project budget aligned with environmental project goals	Was there a business case for going green performed
	Life-cycle cost analysis data to justify operational savings	Is life-cycle analysis data available?
	Payback period for sustainable objectives	What is the payback period for the sustainable aspects and LEED certification?
<i>Owner commitment</i>	Increased commitment from the owner side leads to better project planning and to better cost and schedule performance	Was the owner the green driver of the project? Did the owner introduce the concept of green? Was the owner in charge of educating the rest of the team members?
<i>Architect and CM commitment to green and a consistent approach</i>	Better delivery in regards to green aspects if teams are committed to sustainability.	Are these teams committed to sustainability as one of their core values regardless of owner commitment?
<i>Early team selection/ team experience</i>	bringing the teams together early engages critical process integration and allows system and environmental knowledge to evolve as design begins	Individual experience of team members with healthcare facilities? With green buildings? Previous team experience as a unit? Did the project have a sustainability consultant in the beginning?
<i>LEED certification</i>	level of certification and break-out of points	Why did you go after the specific points? Which specific points required the most effort?
<i>Energy Modeling</i>	Energy modeling helps optimize the building design and allows the design team to prioritize investments in the strategies that will have the greatest effect on the building's energy use	Who was in charge of energy modeling? What were the costs associated with it? Effort and rework? At what stage in the design were energy simulations used?

4.3 Process Maps

One type of analysis in this research is **process mapping** analysis. After the first wave of data collection was completed, the data was synthesized and process models were created for each case study. The critical processes and events for each children’s hospital are modeled and compared. Process modeling is the critical step in analyzing and understanding the delivery process. A lot of green projects have rework, changes and overproduction as a result of not using the best delivery processes. The Lean and Green process modeling protocol will be used to model each step in the delivery process and convey it in a simple and effective way. Comparisons between the four case studies will be more effective by making everything more transparent through the process models. By using this protocol, wasteful and unnecessary processes can be identified. The timing and sequence of the different activities will be identified in each process model and also which stakeholders were involved in what phases of the projects. To prepare the maps the following protocol will be used (Klotz 2007):

- Level 1: a big picture map to understand the overall delivery process and organization. Interviews with an employee, typically a high-level executive, will be completed in two stages. First stage is to record the main events and processes and a follow-up interview is used to clarify that the draft map represents the interviewee understanding.
- Level 2: focus on each individual process, for example programming stage or design, and interview members in each project who understand best the applicable process.

Microsoft Visio has been used to create the maps. An example from the Hershey Process Map is presented below in Figure 2.

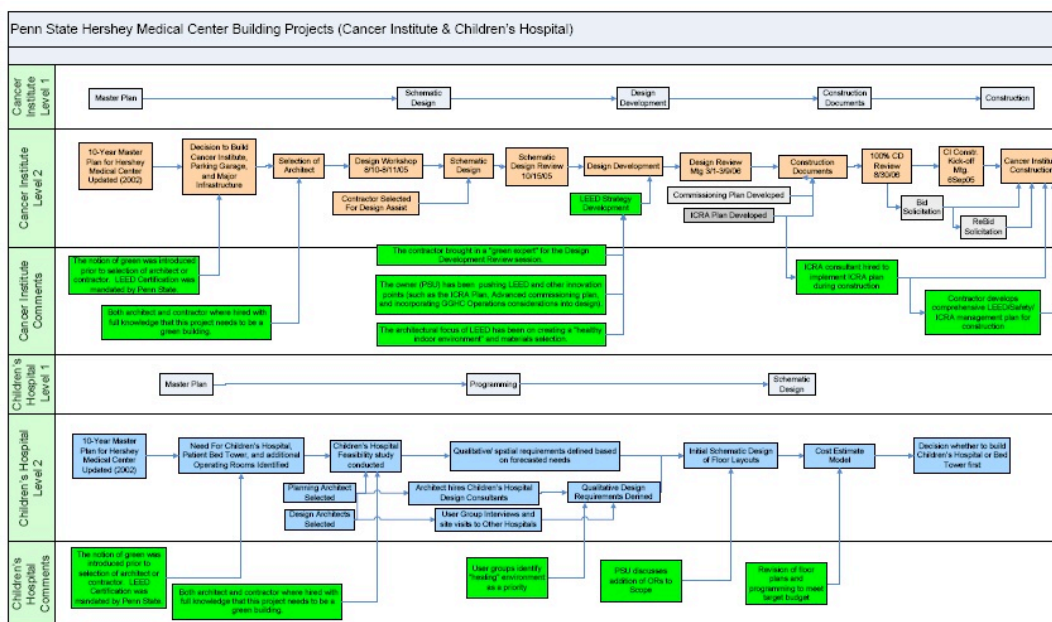


Figure 2: The Hershey Process Map

The delivery processes for green buildings are different than those for conventional buildings. For example, green projects tend to use a stronger interdisciplinary and integrated approach, complex modeling and analysis, untraditional materials and systems that require more planning in the early stages of the project delivery process. Green buildings are often perceived as having a higher first cost due to the complexity of materials, systems and building envelopes, although their life-cycle cost is lower (Klotz 2007)

The process maps for the three children’s hospitals show that they all had a conventional delivery process starting with a master plan followed by programming, schematic design, design development, construction documentation, construction and occupancy and maintenance. The Dell Children’s Hospital of Austin has been a design-bid-build project from the start, while the Children’s Hospital of Pittsburg was also design-bid-build, but it took a fast-track approach in the later phases of the project. The Hershey Children’s Hospital is in the schematic design phase right now, although the master plan was developed in 2002. The delays due to financing in the case of the Hershey project and site selection for the Children’s Hospital of Pittsburgh are examples of situations where specific phases in the delivery process could take longer than expected. The Dell Children’s Hospital followed their strategic plan they set in the beginning and opened in July 2007, while the other hospitals are set to open in 2009 and 2010 specifically. This shows that the commitment for a LEED Platinum project to be completed on time, and have a commitment to green features is higher than a LEED certified project.

. Process modeling is the crucial step in understanding green building delivery processes. A lot of green building processes have a lot of wasteful rework, delays, changes and overproduction due to inappropriate delivery processes. Process waste can undermine sustainable outcomes and limit the business case for sustainability (Lapinki 2005). If process waste is decreased in the delivery of green buildings, then sustainable strategies could be used without the higher first costs. Process modeling is used in this research as an integrative process to unify single processes from each organization into a larger process where the key participants and events are seen.

4.4 Data Analysis

Data analysis consists of categorizing the data collected, examining, tabulating, and recombining the quantitative and qualitative data. The different parts of the delivery process and issues, such as owner commitment, LEED points achieved, will be analyzed and compared between the case studies, and they are considered individual embedded units of analysis. A criterion for analyzing the findings is to look at the patterns, a technique called **pattern-matching analysis**.

The data is analyzed by using different arrays, making a matrix of categories and placing evidence within each category, examining the frequency of certain aspects and events, charettes as an example. The issues in the data collection tool are summarized and analyzed below in Table 3 with discussions for each project.

Table 3: Analysis of Results

<ul style="list-style-type: none"> • <i>Transparent delivery process in relation to green outcomes and an integrated approach</i>
<p>The Dell Children’s Hospital of Austin had all project members involved in the green outcomes aspect from the beginning. The project had monthly eco-charettes in the beginning and quarterly after design was completed where the architecture team, landscape architecture engineers, owner representatives, construction management team and the subcontractors were involved. The Hershey Children’s Hospital and the Due to the LEED Platinum</p>

certification, the project needed a more integrated approach than the two LEED certified projects examined.
<ul style="list-style-type: none"> • <i>Early adoption of green</i>
The notion of green was introduced very early in the programming phases at the Dell Children’s Hospital and similarly at Hershey it is a campus-wide mandate for the Penn State campus. At the Children’s Hospital of Pittsburgh the adoption of green happened in the design phases of the project. An early adoption of green enabled a more clear understanding of the project goals and needs at Dell, and it saved project rework due to accurate project bids. Although LEED certification at the Hershey Children’s Hospital has been mandated from the beginning it has not been a priority for the team yet.
<ul style="list-style-type: none"> • <i>Business case for green initiatives</i>
A business case for infection control was made at the Hershey Children’s Hospital, but not for green in general. The Penn State LEED mandate based assumption that LEED aligns with business case to reduce life-cycle costs. At the Children Hospital of Pittsburgh, sustainability was part of the hospital’s overall vision. The Dell Children’s Hospital of Austin focused on a business case for each initiative from the very beginning of programming. The owner has been the proponent of green on all the projects.
<ul style="list-style-type: none"> • <i>Owner Commitment</i>
For the Dell Children’s Hospital of Austin the owner’s commitment was the most important aspect in the delivery of the LEED Platinum building. The owner never wanted to sacrifice LEED Platinum. They never said it was too difficult or back down from it. Similarly to this, the owner was committed to green as part of the overall vision but not to that extent. In the case of the Hershey Medical Center, the LEED certification is a Penn State mandate.
<ul style="list-style-type: none"> • <i>Architect and contractors commitment to green</i>
Most of the teams are starting their involvement in LEED certified projects and are relatively new to the delivery of green building. The architect for the Dell Children’s Hospital is greening their specifications and promote themselves as green experts. They hope to achieve LEED certification in every project without the owner asking for it.
<ul style="list-style-type: none"> • <i>Early team selection/ team experience</i>
At the Children’s Hospital of Pittsburgh the architectural firm was experienced in green projects, but some individuals had limited experience. At the Hershey Children’s Hospital the project teams had previous experience with LEED, but similarly individuals had limited experience. The owners were not experienced with LEED. At the Dell Children’s Hospital none of the teams had previous experience with LEED, but they were strongly committed to pursuing the LEED Platinum certification and their commitment played an important role in the success of the project. The Dell Children’s Hospital had an experienced sustainability consultant throughout the project.
<ul style="list-style-type: none"> • <i>LEED certification</i>
LEED Certification in the Children’s Hospital of Pittsburgh case study was one of the several project team goals. However, LEED did not guide the design. Penn State has made an institutional commitment to decrease the lifecycle energy and maintenance cost of the facility. As a result, the University has mandated that all new construction meet LEED Certified levels of sustainability. At Dell, the LEED Platinum has guided the design from the beginning and along with patient care has been the top priority for the hospital. All the case studies have focused heavily on the indoor air quality aspect of LEED, due to the nature of the facilities.
<ul style="list-style-type: none"> • <i>Energy Modeling</i>
Both the Children’s Hospital of Pittsburgh and the Hershey Children’s Hospital had the energy modeling performed by the architecture teams. Regarding the LEED points achieved, the Energy and Atmosphere section was not their main focus. In contrast, the Dell Children’s Hospital spent an extensive amount of time and effort on the energy modeling; the energy modeling was performed by an engineering firm. The energy modeling is probably the most important aspect in the achievement of LEED Platinum. Austin energy played an important role in the design and construction of a combined heat and power plant on site which provides all the energy for the hospitals and a few of the surrounding facilities.

5. Conclusions

Increasing the implementation of successful green design and construction processes for healthcare facilities can improve the health of the patients, in this case study the children, increase the productivity of doctors and nurses, and reduce the life cycle cost of the facilities. The analysis of the design and construction process in the three children’s hospitals examined identified several key areas and issues that project teams should focus on to facilitate the implementation of green design in hospitals from both a practical and theoretical perspective. Having a committed leadership that believes from the start in the value of sustainability, adopting green issues from the very beginning, using extensive

energy modeling for increased efficiency, transparency and an integrated approach to the project and most importantly an owner that is very committed to LEED will yield successful results in these projects. Team experience on previous LEED projects also plays a very important role, but as shown in the Dell LEED Platinum project the commitment could be in some cases more important than the experience. Energy modeling and the use of a combined heat and power plant has played a very important role in reducing energy use and achieving LEED Platinum. In the case studies examined, achieving the LEED certification was aligned with good hospital design practice, the owner's aspirations, and other project goals, such as providing an indoor environment conducive to patient recovery. Finally, the LEED green building rating system has provided substantial assistance to the project team in identifying green design features and performance goals. For the Children's Hospital of Pittsburgh and the Hershey Children's Hospital LEED was not fully integrated with the design and construction process, and therefore LEED has remained independent and somewhat secondary to hospital's design development and construction process. In contrast to that, LEED has been integrated through out the design and construction process and it has been a goal and a priority from the start. Completing a LEED Platinum healthcare facility successfully requires a well integrated design and construction approach, buy-in from every team, owner commitment from the very beginning and a sustainability consultant who can guide the teams through the process and provide expertise on the documentation process.

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In-plane and Out-of-plane Load Testing and Evaluation of Sustainable Masonry Walls

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Abstract

This paper presents the results of a pilot study at Penn State University to compare the structural behavior of three types of masonry that have some sustainable attributes. The three masonry types chosen consisted of concrete masonry unit (CMU), autoclaved aerated concrete (AAC), and adobe. Design professionals are interested in the relative performance of sustainable wall systems and this pilot study was motivated because of the lack of such a comparative study. In the study reported in this paper, 4 ft x 4 ft wall specimens were tested in shear and flexure under dry and wet conditions. A specially made spray rack was used to simulate rain condition on wall specimens before structural tests for wet condition. Comparison of the test results and observations made during the tests are discussed.

1. Introduction

Desirable attributes of sustainable construction include high thermal storage capacity and high thermal insulation capacity of wall systems. Various types of masonry have shown to possess such attributes. Walls with high thermal storage capacity are sometimes referred to by architects as “mass walls” or “high thermal mass” (HTM) walls, which means the larger the mass of the wall material, the larger the amount of solar heat it can store and the longer it takes for the material to release heat (after sunset). Aside from sustainable attributes, the use of masonry in residential construction is getting more attention in light of the devastation following hurricane Katrina. Furthermore, new masonry materials are also being recognized to provide an alternative to wood-frame construction that is inherently fire resistant and may reduce the necessity for layers of materials and the need for multiple subcontractors on the wall construction.

Although the use of masonry as the load-bearing exterior walls is common for low-rise buildings, today’s use of load-bearing masonry walls in residential construction may stem from energy efficiency and sustainability considerations rather than the need for a gravity load-bearing system. Nonetheless, the structural behavior of the types of masonry walls that may be considered suitable for sustainable design is a primary concern, given the emerging new masonry materials and in some cases, the use of indigenous materials. In particular, because HTM walls are usually thick and architectural features of sustainable design such as wall setbacks with respect to the roof or re-entrant corners for shading

introduces more complicated structural response under lateral loads, additional experimental studies seem to be necessary. Along with the added structural complications under dry conditions, HTM walls that are subjected to rain or moisture are also of concern because of the possible effects water and moisture can have on thermal efficiency and structural capacity.

Given this interest in increased use of masonry, a pilot study was undertaken at Penn State University to compare the structural behavior of three types of masonry that have some of the desirable sustainable attributes. The three masonry types chosen were conventional material: concrete masonry unit (CMU), new material: autoclaved aerated concrete (AAC), and indigenous material: adobe. Although there are interests in exploring and use of new materials, some architects have rediscovered the desirable sustainable properties of adobe as well. Design professionals are interested in the relative performance of sustainable wall systems and this pilot study was motivated because of the lack of such a comparative study. In subsequent sections, a brief background and literature review, the experimental program, and the test results are discussed.

2. Background and Review of the Principles of Sustainable Design Relevant to Wall Systems

The use of HTM walls in sustainable design can reduce energy cost, particularly in climates with high temperature swing over a 24-hour period (Bolin, 2005; Bunz et al., 2006; Van Geem, 2006; Goodhew and Griffith, 2005, The Natural Home, 2008). Although HTM materials such as concrete or clay brick are favored for passive solar energy use in sustainable design construction, other (new) masonry materials such as AAC with lower HTM property but higher insulation property also qualify as sustainable material with respect to several attributes.

AAC is a factory-produced lightweight precast concrete product made by mixing portland cement, lime, silica sand or fly ash, water, and aluminum powder, and then pouring the slurry into a large mold (Memari and Chusid, 2005). While still in a semi-cured state, the demolded concrete is wire-cut to the configurations and sizes of desired elements such as blocks, beams, slabs, or panels. The members are then placed in a pressurized autoclave for steam curing. The resulting precast units weigh between 25 to 50 pcf with compressive strengths varying between 300 to 900 psi. The insulation property of AAC is approximately an R-value of 1.25 per inch. AAC blocks are laid with thin-bed mortar joints.

Adobe brick is made from a mix of soil (clay, sand, and aggregates), water and straw (for strength) and is molded and sun dried. Adobe and its construction have some useful properties including thermal insulation and thermal energy storage capacity (O'Connor, 1999; Newcomb, 2001; Reeve and Reck, 2001; and Witynski and Carr, 2002). Older type adobe construction is, however, associated with many disadvantages such as large weight, low strength, and brittleness that make traditional adobe construction extremely vulnerable to damage in earthquakes. With renewed interest in the use of adobe in some sustainable designs, modern adobe brick and mortar may include some stabilizer cement.

Very little experimental information is available on such stabilized adobe, and because of the recent interest in its use, this research considered its comparison with CMU and AAC.

The study of behavior of moist and wet walls in this research is important because of the possibility of moisture entrapment in the wall. In general, some masonry walls may have exterior finish such as plaster or cement. Rain water can infiltrate the wall thickness through cracks in the finish surface and get trapped. In such cases, the thermal efficiency of the wall could be compromised and the lateral load capacity of the wall affected, depending on the type of the masonry and mortar. Furthermore, any entrapped moisture may initiate undesirable mold growth in the walls.

Another aspect of the interest to study the masonry walls under wet conditions is to develop a better understanding of the way rain water penetrates through the wall. It is not known for example whether the water leakage would mostly take place through the masonry blocks/bricks or through the mortar. Furthermore, the rate of wetness needs to be determined to yield a measure of the resistance of the wall to water leakage (Choi, 1998). Still another reason for the need to study water effect on walls is the fact that any moisture trapped may adversely affect the performance of the building in the long term. For example, water entrapment can lead to corrosion of reinforcement, metal anchors, or other metal attachments to the building. According to Galitz and Whitlock (1998), "Improper handling of rainfall incident upon building surfaces causes innumerable problems, both aesthetic and structural, e.g., efflorescence, mildew, corrosion, and freeze/thaw damage." It is therefore, necessary to better understand the mechanism of water penetration through masonry walls.

Furthermore, according to Straube and Burnett (1998), "Despite the importance of driving rain to building performance, very little is known about the magnitude, duration, and frequency of driving rain deposition on buildings." According to Straube and Burnett (1998), mass walls (e.g., masonry) absorb and store some of the rain water that wets the exterior wall surface. A wall that functions properly, will lose the stored moisture through evaporation drying before any wetness reaches the interior surface of the wall. However, this study has shown that the wetness will reach the other side of the walls; only the amount of moisture will vary depending on the type of masonry used.

3. Experimental Program

The objective of the pilot study reported here was to compare the lateral in-plane and out-of-plane behavior of three types of masonry walls under dry and wet conditions. The test matrix discussed here consists of three 4 ft x 4 ft specimens of each of the three chosen masonry types (CMU, AAC, and adobe) tested under in-plane shear loading and two conditions of dry and wet specimens. Three specimens of each masonry type were tested under out-of-plane flexure loading and under dry and wet conditions. On this basis, a total of 36 specimens were tested in this study.

The specimens were constructed by professional masons in the laboratory where they were stored for curing. The base case material for this study was conventional CMU

blocks with a nominal block size of 8 x 8 x 16 in. The mortar joints were 3/8 in. thick and were of medium strength type S mortar. The AAC blocks had dimensions of 8 x 8 x 24 in. and thin bed mortar was used that resulted in 1/16 in. thick joints. The adobe bricks were 4 x 8 x 14 in. and a thick cement stabilized mortar was used that yielded 3/4 in. thick joints.

4. Description of Test Setups (Load Testing and Water Spray Testing)

The out-of-plane flexural tests, as shown in Figure 1, consisted of a setup with loading applied at mid-height, while the in-plane tests also shown in Figure 1 were carried out in the form of a concentrated load at top corner of the specimen. A tie-rod hold-down mechanism was used on the loading jack side and a sliding stop mechanism on the bottom far side. The static monotonic loads were applied using ENERPAC loading jacks. Measurements were taken using load cell and displacement transducers.

The moisture penetration test, also shown in Figure 1, used a specially constructed spray rack to simulate rain conditions on the wall specimens for 2 hours before structural tests under wet conditions were performed. Various methods of applying water spray on wall specimens to simulate rain conditions have been studied by (Galitz and Whitlock, 1998). Such methods include water test chamber, spray rack, and calibrated nozzle testing. ASTM E514, C1403 and C1601 (ASTM 2006) discuss the standard methods of testing moisture penetration of masonry under wind driven rain, field testing of moisture penetration and water absorption of masonry mortars.

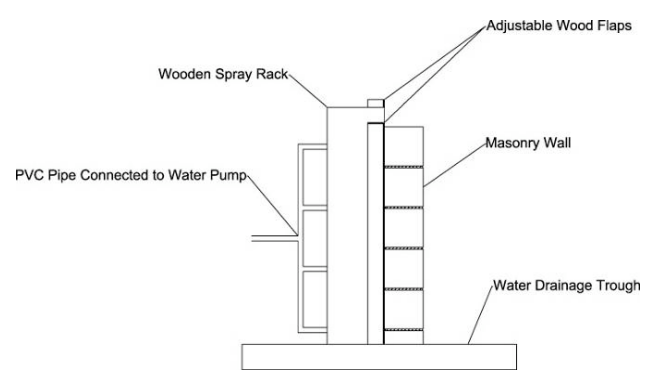
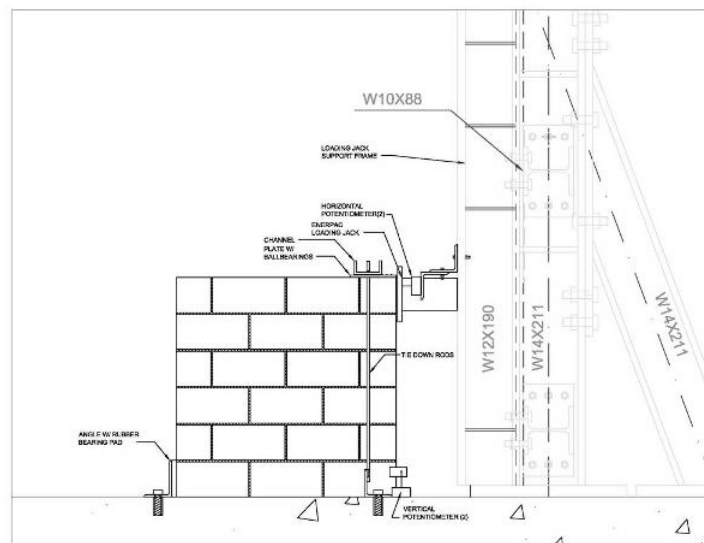
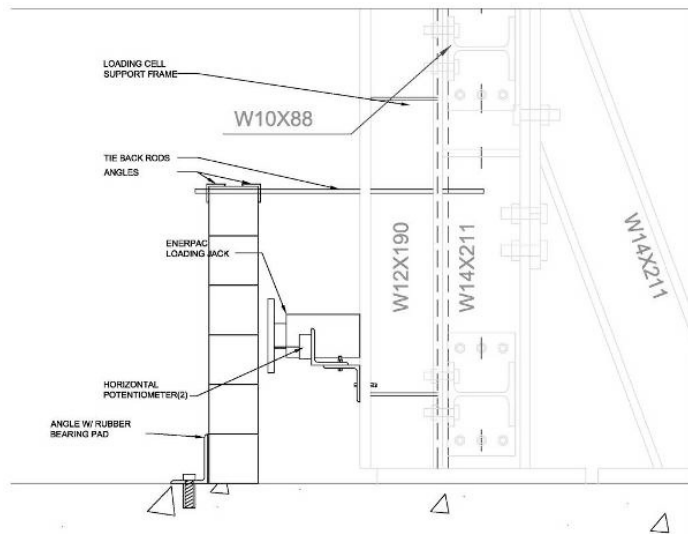


Figure 1. Setups for out-of-plane flexure load test, in-plane shear load test, and water penetration test

The wall specimens were subjected to a constant flow of water to the exterior through the specially constructed spray rack. The wall was inspected and the water penetration to the interior side was documented in increments of 10 minutes for 2 hours. Immediately after the 2-hour moisture penetration test, the walls were then tested for in-plane or out-of-plane structural capacity while still wet.



Figure 2. Photographs of the wall specimens at the end of water penetration tests

5. Discussion of the Water Penetration Test Results

During the water penetration test, pictures were taken every 10 minutes for 2 hours and were compared to the 3 similar wall specimens along with comparison of the AAC, CMU and adobe wall specimens. Taking the average saturation levels of the different masonry materials from the photographs, the CMU walls showed much more water penetration than the Adobe and the AAC. The Adobe walls had a considerable amount of water penetration while the AAC walls stayed somewhat dry in comparison. This test also showed the water traveled through the mortar joints significantly faster than through the blocks in all 3 wall materials. The CMU walls, because of the blocks not being solid, showed more water penetration through the block than the Adobe and AAC walls. Figure 2 shows photographs of the 3 most saturated wall specimens after the 2 hour spray period.

Taking the average saturation levels of the different masonry materials, the CMU walls showed much more water penetration than the Adobe and the AAC. The Adobe walls had a considerable amount of water penetration while the AAC walls stayed somewhat dry in comparison.

6. Discussion of the Load Testing of Wet and Dry Wall Specimens

The test results are presented here in the form of load-displacement curves for shear and flexural tests of CMU, AAC, and adobe masonry wall specimens. Comparison of the three dry and three wet responses of different wall systems will be made based on such diagrams as well as photographs taken from specimens before and after the tests. Figures 3-5 show the load-displacement plots for shear and flexural tests of CMU, AAC, and adobe specimens under dry and wet conditions. Approximate average value of peak load resulting from shear tests for the CMU, AAC, and adobe specimens are, respectively, 10,000 lbs, 19,500 lbs, and 9,500 lbs for the dry tests and 9,000 lbs, 11,000 lbs, 6,400 lbs for the wet tests. The respective displacements under the peak loads are 0.6 in., 1.80 in.,

and 1.0 in for the dry tests and .70 in, .70 in, .75 in for the wet tests. The boundary conditions for all wall specimens were the same.

The dry wall test results show that AAC walls had approximately 95% higher strength capacity while adobe walls had approximately 5% lower strength capacity compared to CMU specimens. One reason for the relatively larger AAC shear capacity is that AAC masonry is solid, while the CMU blocks were hollow

and not grouted. Although adobe is generally thought of to be a weak material, the test results show that the cement stabilized adobe can have shear capacity comparable to that of ungrouted hollow CMU block walls. For the wet wall test results, AAC had approximately 22% higher while the adobe walls had 28% lower strength capacity compared to CMU

specimens. Comparing the dry test results to the wet test results, the CMU, AAC and adobe decrease in strength capacity are, respectively, 10%, 44% and 33%. Figure 6 show the photographs of the specimens tested under flexure both before and after the shear test. Although there were variations in the failure planes of AAC specimen, the failure mode of CMU and adobe specimens were consistent.

Approximate average value of peak load resulting from the flexural tests for the CMU, AAC, and adobe specimens are, respectively, 700 lbs, 3000 lbs, and 350 lbs for the dry tests and 1100 lbs, 2000 lbs, 300 lbs for the wet tests. The respective displacements under the peak loads are 0.69 in., 0.24 in., and 0.07 in for the dry tests and 0.41 in, 0.71 in, 0.12 in for the wet tests. The boundary conditions for all wall specimens were the same.

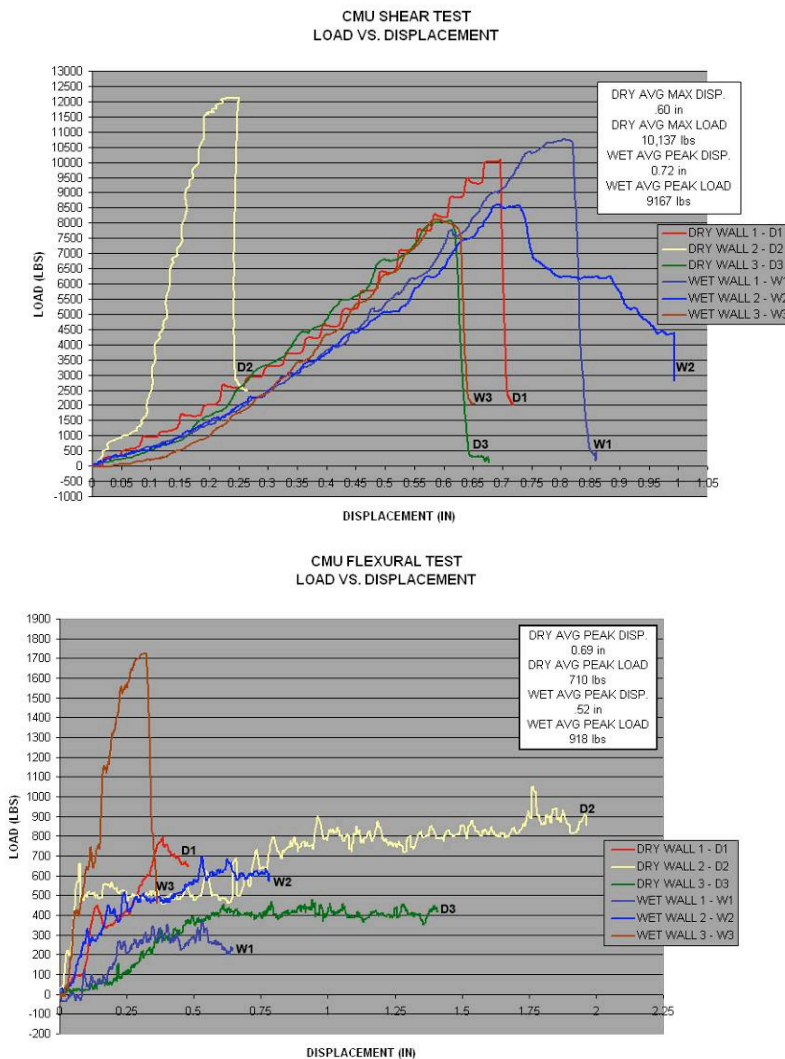
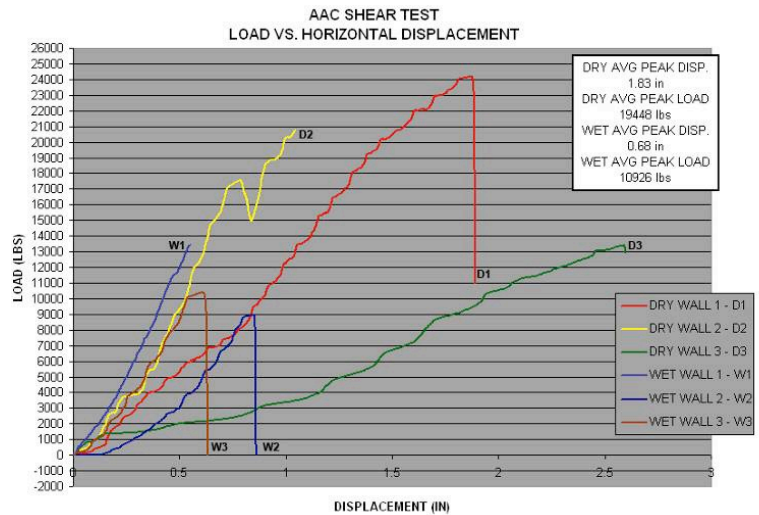


Figure 3. CMU specimen shear & flexural test results

These plots show that the adobe specimens had approximately 50% less flexural capacity in the dry wall tests than that of the CMU and the AAC specimens had 328% more flexural capacity than CMU. In the wet wall tests, both the AAC specimens and the adobe specimens have a reduced flexural capacity while the CMU average flexural capacity increased. The reason could be that the dry and wet wall specimen # 3 flexural capacities were somewhat lower and higher, respectively, compared to the other two wall specimens. The average



displacements of the dry and wet wall specimens seem to be a bit irregular as the CMU specimens decreased peak displacement, while the flexural capacity increased. However, both AAC and adobe specimens increased in peak displacement, while the flexural capacities decreased. Figure 6 show the photographs of the specimens tested under flexure both before and after the flexural test. They consistently show failure plane through the bed joint, except for AAC specimens where failure is also through the masonry itself.

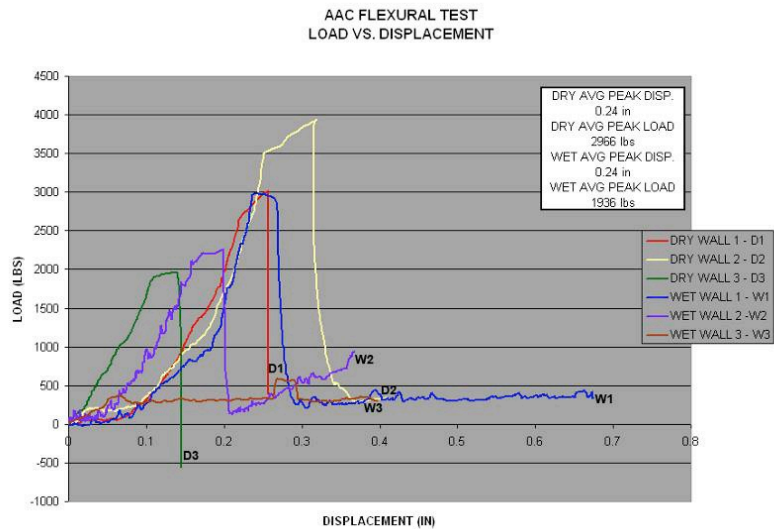


Figure 4. AAC specimen shear & flexural test results

7. Concluding Remarks

Although limited in scope, number of specimens, and the choice of test setup, this pilot study has shown results that would be of interest to architects and structural engineers. The results have shown that walls made with the relatively new AAC masonry can have larger shear and flexural capacity compared to the conventional CMU in both dry and wet conditions. Considering that the density of AAC is 1/3 to 1/5 that of concrete, the

strength to weight ratio for these walls will be quite attractive for seismic design. Another result of the study is the high strength that the cement stabilized adobe brick is showing in the dry shear and flexure tests. This study has also shown that subjecting these walls to water and moisture to simulate rain, the structural capacity is decreased in both shear and flexural capacity in all cases except the CMU flexural test, which increased slightly. The results show that water penetration into a structural sustainable wall could have adverse effects on a structure if not properly dried out and water gets trapped in the wall system. Although the test results presented here clearly show some favorable attributes for AAC and adobe, further tests with full size wall specimens and with more realistic

boundary conditions are necessary before any solid conclusions can be drawn.

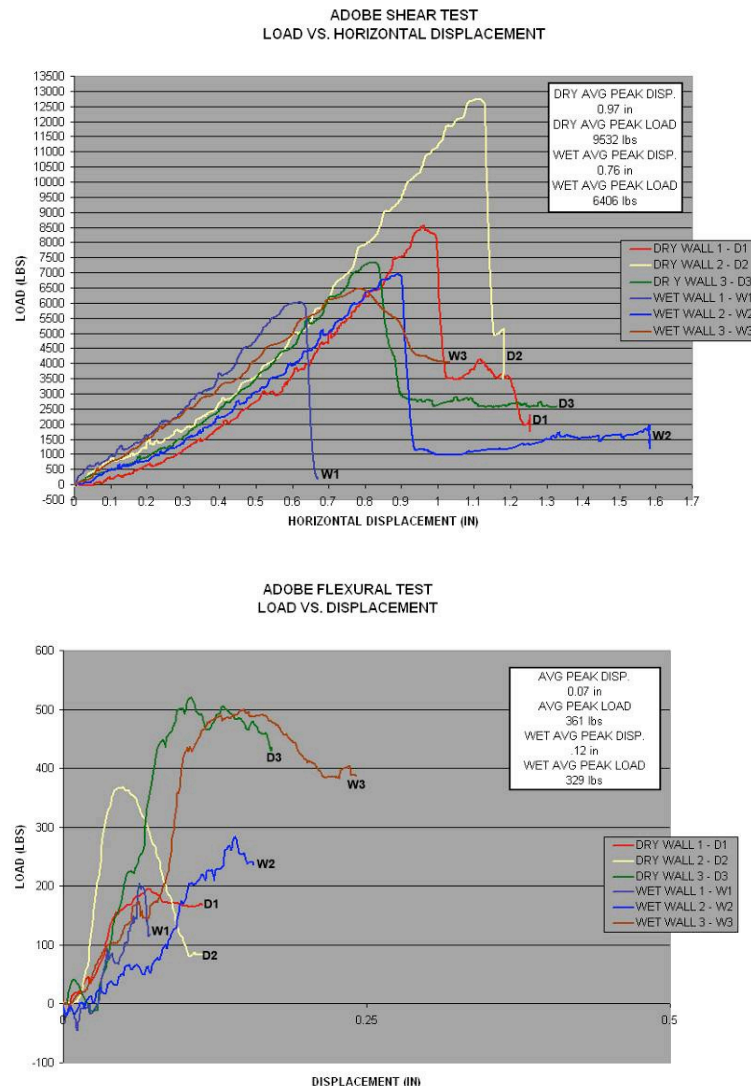


Figure 5. Adobe specimen shear & flexural test results

8. Acknowledgements

This research was partially supported by a grant from the Ramond A. Bowers Program for Excellence in Design and Construction of the Built Environment at Penn State University. The International Masonry Institute provided the CMU and AAC material and its professional staff constructed the specimens. All the support provided to this research is gratefully acknowledged. The assistance of Paul Kremer, Research Associate, and the following students in carrying out the experiments is acknowledged: Joseph Ridgway, Philip Riegel, Daniel Clark, and Michael Hopple. Any opinion expressed in this paper is that of the authors and does not necessarily reflect that of the sponsors.

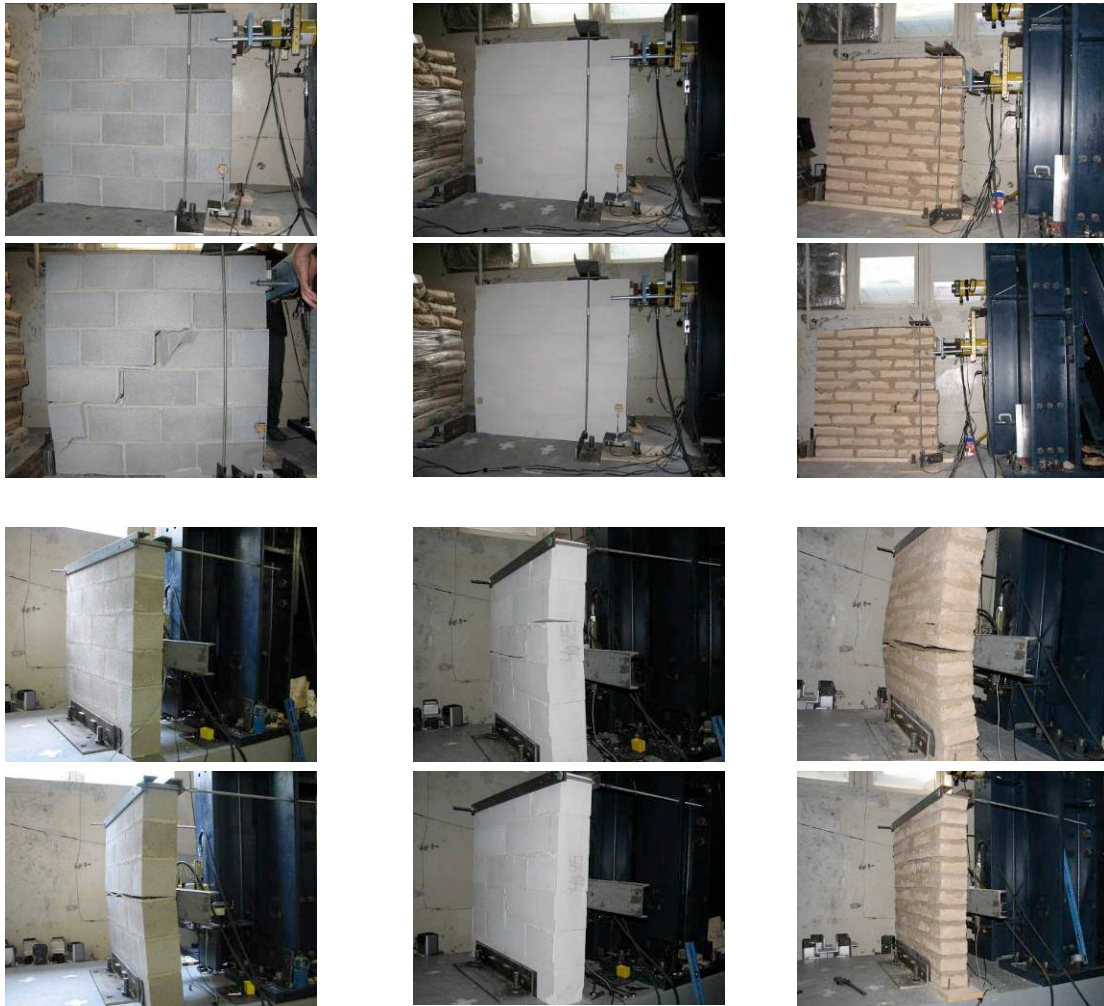


Figure 6. Photographs of shear and flexure test specimens before and after the tests

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Safety Assessment of Construction Site Layout Using Geographic Information System

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Abstract: Construction site layouts involve identifying the positions of temporary facilities on site, and play a major role in the safety and productivity of operations, particularly when site space is restricted. This paper presents a new method of safety assessment of construction site using geographic information system (GIS). First, safety issues on construction sites are discussed and the factors that contribute to unsafe sites are outlined. A procedure for dynamic safety assessment of site layout has been developed in a GIS to obtain proper safety zones around the construction temporary facilities. Safety-related temporary facilities and potential places for locating temporary facilities were established. Furthermore, optimum placement of temporary facilities on the site was considered. An application example is analyzed to demonstrate the application of the model in assessment of construction site layouts based on safety consideration. This shows the capability of GIS combination for proper site layout which can lead to significant improvement in the safety and cost performance construction projects.

Keywords: Safety, Construction Site Layout, Temporary Facilities, Geographic Information System

Introduction:

Safety hazards at the construction site can be one of the main causes of unanticipated additional costs. Many construction companies around the world are implementing safety management systems to reduce injuries, and to provide a safe work environment in their construction sites (Choudhry et al 2008). The annual number of fatalities in the construction industry in 2003 exceeded the number of combat deaths during the first 18 months of armed conflict in Iraq (period 2003-2004) (Halpin 2005).

Safety of construction projects may be affected by various factors such as types and scale of projects, construction methods, safety management procedure, and site conditions (Seo and Choi 2008). Among them is the construction site layout in relation to safety. Construction site layouts involve identifying the positions of temporary facilities (TFs) including security fences, access roads, storage areas of material and equipment, stockpiles of excavation, site offices, fabrication shops, and batch plants on site. (Yeh 1995; Hegazy and Elbeltagi 1999). In addition to reducing the cost of

materials handling and minimizing travel times of resources on site, a proper site layout can lead to significant improvement in the construction safety and quality (Tommelein et al. 1992; Anumba and Bishop 1997).

Health and safety issues were often ignored in most previous studies on construction site layout (Anumba and Bishop 1997). Among the few researchers who considered safety issues during construction site layout planning are Anumba and Bishop (1997), Elbeltagi et al (2004), and El-Rayes and Khalafallah (2005). Anumba and Bishop (1997) examined the need to integrate safety into site layout and organization, and provided guidelines for the required integration. Elbeltagi et al (2004) defined proper safety zones and facilities closeness weights based on safety considerations to provide a dynamic model for site layout planning. El-Rayes and Khalafallah (2005), on the other hand, developed a site layout planning model to improve safety while seeking to minimize the travel cost of resources on construction site. The model considered three major TFs as safety criteria to provide optimal trade-offs between safety and cost in construction site layouts.

In order to improve safety on construction site, safety assessment should be considered in selecting models for practical applications. The paper presents a procedure that indicates the level of hazard within a construction site. As all construction projects are unique in nature, it can be difficult for construction managers to anticipate all the hazard areas existing on the site. By viewing the geographic information system (GIS) model, they can detect areas where accidents may occur and execute prevention measures. But more importantly, by viewing the time and location of the work through the GIS model, construction managers can perceive how layout of TFs may create hazardous situations. Khattak and Shamayleh (2005) used GIS for safety assessment of highway stopping and passing sight distances. Built upon the capability and promise of GIS, three-dimensional models of the various highway subsections were obtained.

This paper is organized as follows. The next section provides an overview of the major TFs that affect construction site safety. The following two sections outline two important aspects of safety assessment, namely: defining dynamic nature of major measures and formulating the impacts of TFs overlapping on safety. This is then followed by an application example to demonstrate the application of the model in assessment of construction site layouts based on safety consideration. Finally, the last section includes the conclusions and proposed directions of future research.

Temporary Facilities in Construction Site Layout

As an initial part of this work, TFs that affect construction site safety and their impact were identified and quantified. Based on the literature review (Jaselskis and Suazo 1994; Anumba and Bishop 1997; Hinze and Russel 1995; Hinze et al 1998; Sawacha et al 1999; Elbeltagi et al 2004; El-Rayes and Khalafallah 2005; Carter and Smith 2006) and OSHA publications (OSHA 1998; OSHA 2002a-d; OSHA 2003a-b), the major measures affecting the construction safety were identified as follows:

- Safety of crane operation. Significant and serious injuries may occur if cranes are not used properly. Often these injuries occur due to falling objects and crane accidents.
- Hazardous material. Hazardous material and equipment are often utilized and located on construction sites, exposing construction workers and engineers to safety risks.
- Travel Routes Intersections. Frequent movements of resources on site often create safety hazards when the travel routes of these resources intersect. The intersection areas between routes create safety hazards due to the high potential of accidents and collisions.
- Scaffolding. When scaffolds are not erected or used properly, fall hazards can occur. About 2.3 million construction workers frequently work on scaffolds. Protecting these workers from scaffold-related accidents would prevent an estimated 4,500 injuries and 50 fatalities each year.
- Falls and falling objects. Falling objects from facilities under construction is the most frequent cause of accident in construction site. Falls are routinely the most common cause of workplace injuries in the United States.
- Heavy equipment in excavations. Even though heavy equipment is used to excavation and trenching, workers are required to be inside the trench to guide the excavations. For this reason excavation and trenching are among the most hazardous construction operations.

The following sections explain the method for quantifying the impact of above-mentioned measures on construction safety. The method for the first three measures (i.e. crane, hazardous material, and travel routes intersections) was introduced by El-Rayes and Khalafallah (2005), and for the last three measures (i.e. scaffolding, falls, and excavations) was developed by the authors.

Safety of Crane Operations

The degree of safety for operating cranes in construction site can be quantified by a performance metric named crane safety criterion (CSC). This criterion can be calculated using Eqs. (1) and (2)

$$Crane\ safety\ criterion\ (CSC) = \sum_{k=1}^K \sum_{z=1}^3 CS_k^z \quad (1)$$

$$CS_k^z \begin{cases} CS_k^1 = \frac{\theta_f}{360^0} (A_{z1} + A_{z2^*}) & (zone1) \\ CS_k^{2^*} = \frac{\theta_u}{\theta_f} \frac{CS_k^1}{m} & (zone\ 2^*) \\ CS_{ik}^2 = CS_k^{2^*} \frac{(H - J)}{2} + CS_k^{2^*} & (zone\ 2) \\ CS_{ik}^3 = 0 & (zone\ 3) \end{cases} \quad (2)$$

where CS_k^z = the area of safety performance due to crane k in Zone z ; A_{z1} = total area of zone 1; A_{z2} = total area of zone 2; K = total number of cranes on site; J = length of the crane jib; m = ratio between the risk of falling objects from crane and the risk of crane collapse; H = reach of the crane; θ_i = angle measured clockwise from the north direction; θ_f = set of crane operating angles; and θ_u = set of unfeasible crane operating angles.

According to this criterion, safety zones around cranes classified into four zones, as shown is Fig. 1. Zone 1 is the area that covers the crane operating angles ($d_i < J$ and $\theta_i \in \theta_f$), and it represents the highest risk zone due to its vulnerability to falling objects from the crane during its operations. Zone 2 is located between Zones 1 and 3 ($J \leq d_i \leq H + J$), and it represents an intermediate level of risk due to its minor vulnerability to low probability crane accidents such as the tilting and/or collapse of the crane. Zone 2* is the area representing the range of unfeasible crane operating angles ($d_i < J$ and $\theta_i \in \theta_u$) and can only be exposed to the risk of crane collapse rather than falling objects. Zone 3 lies outside the crane risk areas ($d_i > H + J$), and therefore its safety is unaffected by crane operations.

Hazardous Material

Hazardous materials include (1) explosives and blasting devices; (2) flammable material such as fuel; (3) toxic substances such as materials including 13 carcinogens identified by OSHA (OSHA2003b); and (4) sources of harmful radiation and high electric voltage. These hazardous materials need to be properly stored and adequately separated to minimize the risk of accidents on site.

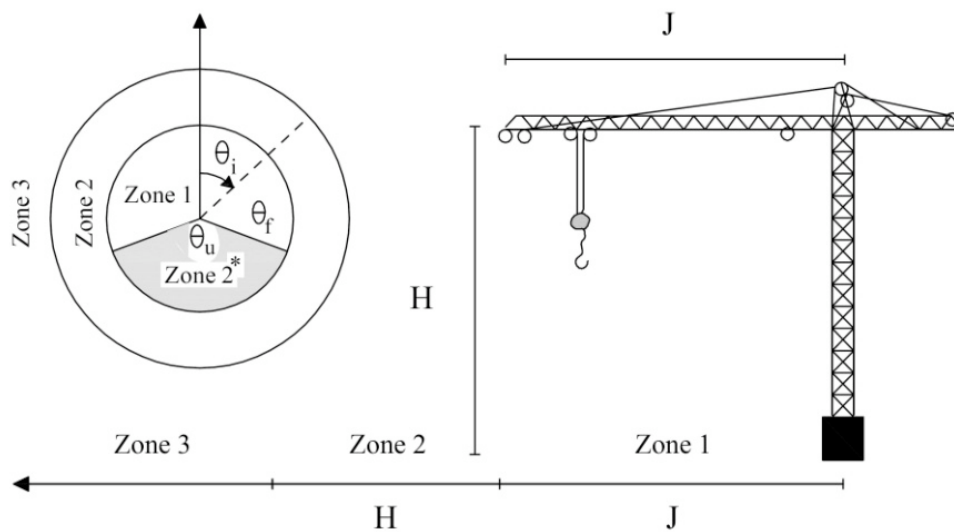


Fig. 1. Safety zones around cranes

In order to improve safety on construction sites, managers need to identify proper storage locations for all hazardous material on site. To this end, they should determine the specific combinations of material and/or TFs (i.e. hazardous material and related TFs) that can create hazardous conditions on site (e.g., explosives and blasting devices). Again, the developed performance metric named hazards safety criterion (HSC) was used for quantifying the degree of safety for hazardous material. As shown in Eq. (3), if the hazardous material and related TFs located far from each other, HSC equals to zero. The geometric relationship between TFs like “far from” could be replaced by a distance that which indicates recommended distance between hazardous materials and related TFs.

$$Hazard\ safety\ criterion\ (HSC) = \begin{cases} 0 & d_{ij} > d_r \\ \frac{(d_r - d_{ij})}{d_r} H_i & d_{ij} \leq d_r \end{cases} \quad (3)$$

where d_{ij} =distance between hazardous material i and related TF j ; d_r =recommended distance between hazardous material i and related TF j ; and H_i =the area of storage for hazardous material.

Travel Routes Intersections

Frequent movements of resources (e.g., labor, material, and equipment) on site often create safety hazards and accidents when the travel routes of these resources intersect (El-Rayes and Khalafallah 2005). The present model assist managers in identifying impact of travel routes on safety by incorporating a newly developed performance metric named intersection safety criterion (ISC). As shown in Eq. (4), ISC demonstrates the amount of the intersections area according to their relative weight/significant. Safety on construction site can be improved by reducing the IAC. For example, in Fig. 2, Layout II provides an improved level of safety that that associated with Layout I.

$$Inter\ section\ safety\ criterion\ (ISC) = \sum_{i=1}^N I_i \times A_i \quad (4)$$

where I_i =relative weight of intersection area i that ranges from 0 to 100%; A_i =the area of intersection I; and N =total number of intersection between travel routes.

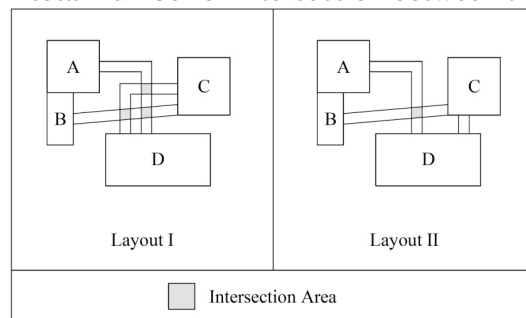


Fig. 2. Intersections among heavily traveled routes of resources

Scaffolding

In the National Census of Fatal Occupational Injuries (CFOI) in 2005, an average of 88 fatalities occurred in the years 2000-2004 from scaffolds, staging. Because the scaffolds are located next to the building boundaries, their surrounding areas take into account in scaffold safety criterion (SSC). Therefore, the length of safety hazard area due to the scaffold is equal to the length of building boundary (e.g. exterior wall) and its width is equal to the width of the scaffold.

Falls and falling objects

In the construction industry, falls are the leading cause of worker fatalities. Each year, on average, between 150 and 200 workers are killed and more than 100,000 are injured as a result of falls at construction sites (OSHA 1998). Based on OSHA (OSHA 1998) all of the locations with 6 feet (1.8 meters) or above in height are the potential of fall hazards. Therefore, all of the open areas such as building ducts, roofs and leading edges recognized as falls and falling objects area. Because falling objects from scaffolds and cranes were considered separately, they did not take into account in this step. The level of safety due to falls and falling objects can be obtained from falling safety criterion (FSC). For example, FSC in Fig. 3 is the sum of the shaded areas next to leading edges. The width of the shaded area representing boundary of hazardous area due to falling objects. It is assumed that the potential distance from leading edge that affected by falling objects is 10 feet (3 meters).

Excavations

Trenching and excavation work presents serious hazards to all workers involved. Cave-ins pose the greatest risk and are much more likely than other excavation-related accidents to result in worker fatalities (OSHA 2002c). In this paper, the new performance criterion named excavation safety criterion (ESC) was introduced to determine the level of safety for excavations. This criterion is equal to area of all open excavations made in the earth's surface, including trenches.

As seen in the major measures above, they may not exist at all stages of the project life cycle. Accurate assessment of safety on construction site is heavily dependent upon the dynamic nature of the method. For this reason, next section describes the incorporation of time into safety assessment.

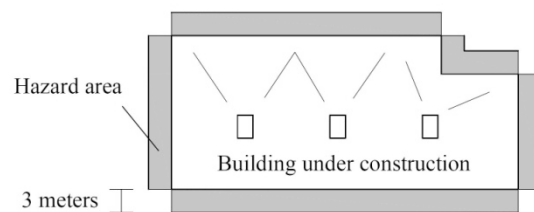


Fig. 3. Hazardous area due to falls and falling objects

Dynamic Safety Assessment

In this study, dynamic safety assessment can be considered as project resources along project duration. This process of defining safety-related TFs is illustrated in Fig. 4. In this figure, as list of TFs (crane, storage of hazardous material, travel routes, scaffoldings, falls area, and excavation equipments) is considered as resources assigned to activities. For example, the “scaffold” on the north side of building is assigned to activity 4. This activity starts on April 10 and completes on July 5. Therefore, the impact of the north side scaffold on construction site safety considered only for its duration. In other words, the proposed safety assessment of construction site layout is time-dependent and has a dynamic nature.

Conflicts between Hazard areas

The impact of individual major TFs on construction site safety mentioned in the previous section. In reality, it is not so, because the more times two or more measures are overlapping, and therefore conflicts between them increase the probability of a hazard's occurrence. To quantify the impact of conflicts with overlapping area on safety, this section incorporates a newly developed criterion named overlapping safety criterion (OSC). As shown in Eq. (5), this criterion is designed to quantify the degree of hazard on overlapping area as a function of (1) the overlapping area (O_{ij}) between TF i and j ; (2) the relative weights (w_i and w_j) of TFs safety criterion; and (3) the sensitivity (V_{ij}) of each pair of TFs to increase the probability of a hazard's occurrence. The sensitivity of TFs can be specified by construction managers by selecting from three categories of low, medium, and high sensitivity. A low category represents facilities that pose little or no risk if they exist in the same location with other TFs. Conversely, a high category represents high risk facilities that their conflicts could cause accidents for project.

$$Overlapping\ safety\ criterion\ (OSC) = \sum_{n=1}^N V_{ij} \times (w_i + w_j) \times O_{ij} \quad (5)$$

where $V_{ij} = 0\%$ for low, 50% for medium, and 100% for high sensitivity; O_{ij} = overlapping area between facility i and j ; w_i = relative weight if facility i ; w_j = relative weight if facility j ; and N = total number of overlapping area.

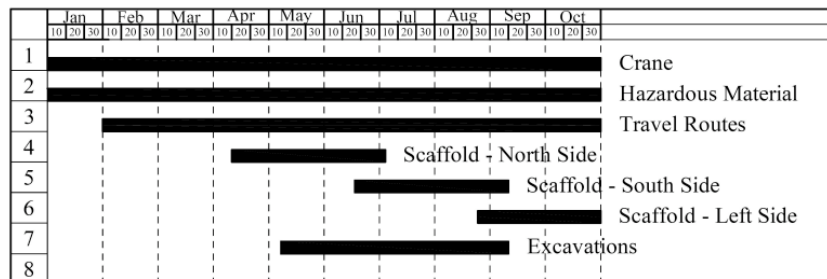


Fig. 4. Major temporary facilities on safety at different schedule intervals

Geographic Information Systems for Safety Assessment

As indicated in previous sections, the procedure established is centered on area of each major TFs that affected construction site safety. It was also include spatial relationships. On the other hand, geographic information system (GIS) software is unique in its ability to capture, store, and manage spatially referenced data. Simply used as a spatial database, GIS assists in modeling applications through handling a special form of data that would otherwise be compromised or impossible to store in nonspatial databases (Miles and Ho 1999). Spatial operations of GIS such as proximity tools is used to determine if a given point, line, or area lies within a certain distance of another point, line, or polygon. The method of Safety Assessment of Construction Site Layout Using GIS was presented in this section.

Crane operation area, in first, determines based on location of crane and crane's operating radius. Proximity functions such as buffer analysis tool determine area lies within a certain distance of crane location. By using the buffer analysis tool the area that covers the crane operating angles can be drawn according to its radius. If the radius of buffer enters equal to the reach of the crane (H), the new created area is the zone 2 in crane safety criterion. Commercial GIS does provide means to calculate the area of each zone with the attributes tables. Also, other required factors for calculating CSC such as m , θ_i , θ_f , and θ_u should be determined by manager. It is interesting to note the area of buffer around location of crane with a radius of crane jib is available in polygon attribute table, and is equal to area of zone 1 and zone 2* (i.e. $A_{z1} + A_{z2^*}$). Having A_{z1} and A_{z2^*} , CS_k^1 could be found and make it possible to calculate CSC for each crane based on Eqs. (1) and (2).

One of the main characteristics of commercial GIS is that it directly calculates the distance between spatial data such as location of hazardous material and related TF. In addition, the area of all objects that defined as polygons (e.g. hazardous material storage) are available in the related attribute tables. Also, recommended distance (d_r) is determined by planner. As a result, we will be able to obtain the hazards safety criterion (HSC) of each combination of hazardous material storage and its related TF.

Same approach is used for other safety criteria (i.e. ISC, SSC, FSC, and ESC). Having the area of major measures that affected construction site safety makes it possible to calculate these criteria in GIS environment. Therefore GIS is able to handle the whole process of safety assessment of construction site layout. These criteria, however, need to be aggregated into a single formula to facilitate the comparison between different site layouts. To this end, a weighted average formula was developed, as shown in Eq. (6). The relative weights in Eq. (6) can be provided by manager according to their importance.

$$SAC = \frac{\sum_{i=1}^N w_i \times C_i}{A_r} \quad (6)$$

where SAC = safety assessment criterion; w_i = relative weight for each safety criterion; C_i = safety criterion for the major TFs; N = total number of major TFs; and A_T = total area of construction site.

It is important to note that the above expression safety criteria include CSC, HSC, ISC, SSC, FSC, and ESC. Also major TFs are those affecting the construction safety (e.g. crane, hazardous material, scaffolding, and etc).

Application Example

An application example is considered to demonstrate the use of the GIS model and illustrates its capabilities in safety assessment of construction site layouts. The example involves the layout of a construction site for a real-life multistory office building. The example has a 29,500 m² site area and involved the construction of office buildings, with perimeter fences and gates. Fig. 5 shows the general site plan with a layout of the buildings to be constructed. Also, some other data that is related to the application example is given in Table 1. Furthermore, the sequence of major TFs is presented in Fig. 6.

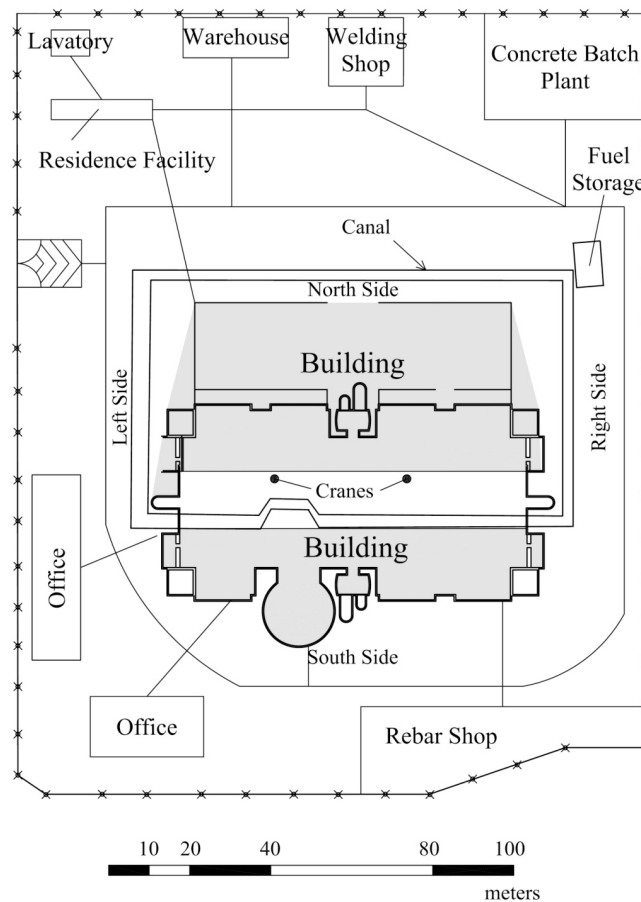


Fig. 5. Site layout of application example

Table 1. Input for application example

Data	m	H	J	θ_f	I_i	d_r
Value	10	50 m	30 m	300°	60%	100 m
C_i	CSC	HSC	ISC	SSC	FSC	ESC
w_i	0.6	0.3	0.8	1	1	0.2

Fuel storage was recognized as hazardous material and the closeness of welding shop and fuel storage can create hazardous conditions on site. Construction of buildings creates hazardous condition for falling objects. Therefore, falling safety criterion should be calculated in the duration of building construction. The proposed GIS model for safety assessment was implemented on a commercial package, ArcMap.

First, the given site layout was generated in AutoCAD environment. Because of facilitating the analysis process by GIS, some changes were performed including the location of TFs drawn as polygone and travel routes drawn as polyline. According to the sequence of TFs, analysis functions in ArcMap were used to calculate safety criteria. As describes in previous section, required area and distance such as overlapping area can be obtained by buffer analysis tool. Fig. 7 shows the safety assessment procedure generated by ArcMap for the safety criterion on July, 2007. In this figure, attributes tables illustrate safety criterion for major TFs.

Similarly, the safety assessment criterion has been calculated through project duration time. Fig. 8 shows the SAC for application example over time.

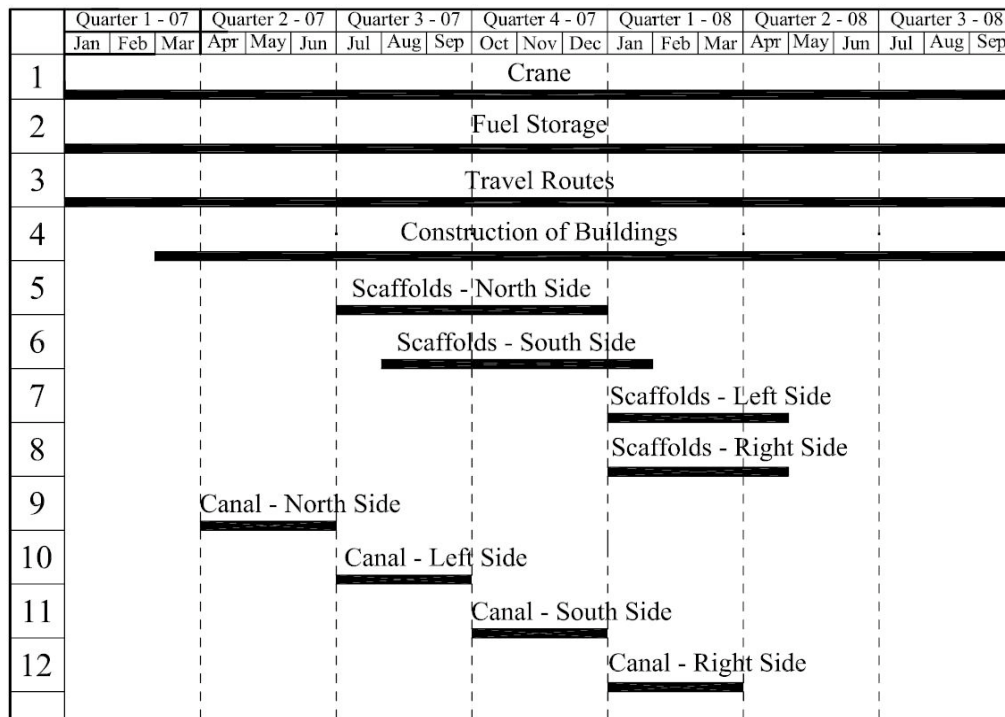


Fig. 6. Default duration and sequence of temporary facilities

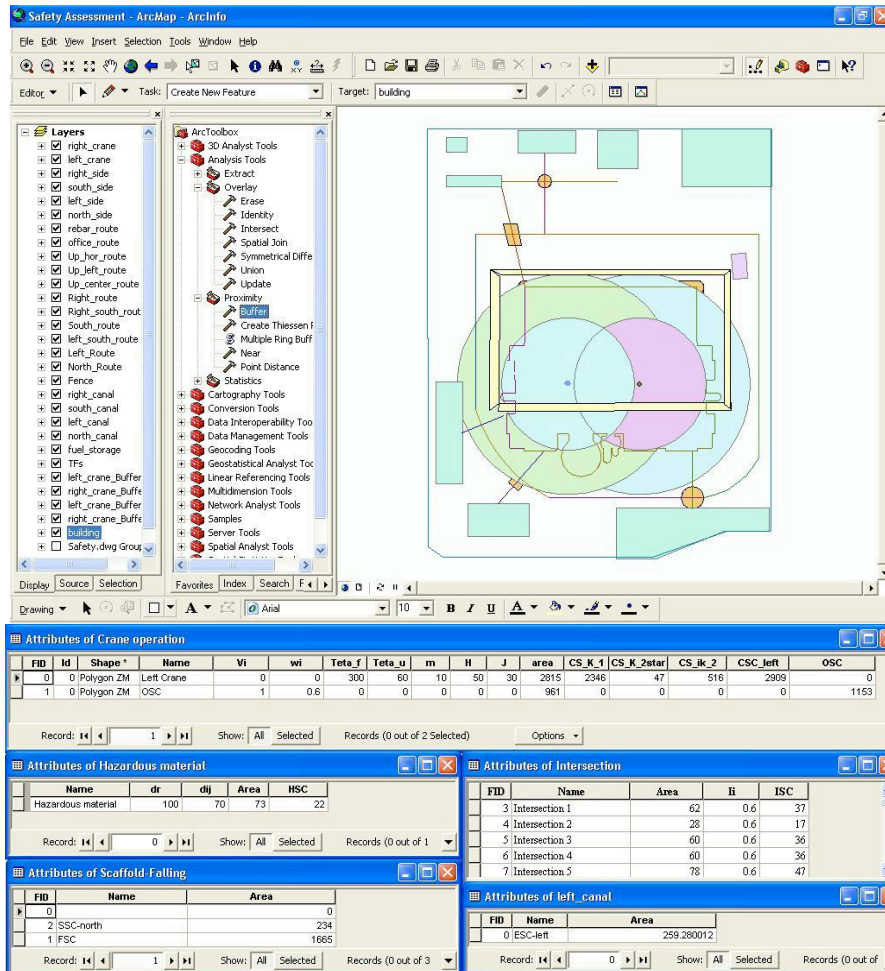


Fig. 7. Safety criterion on July 2007 generated by GIS model

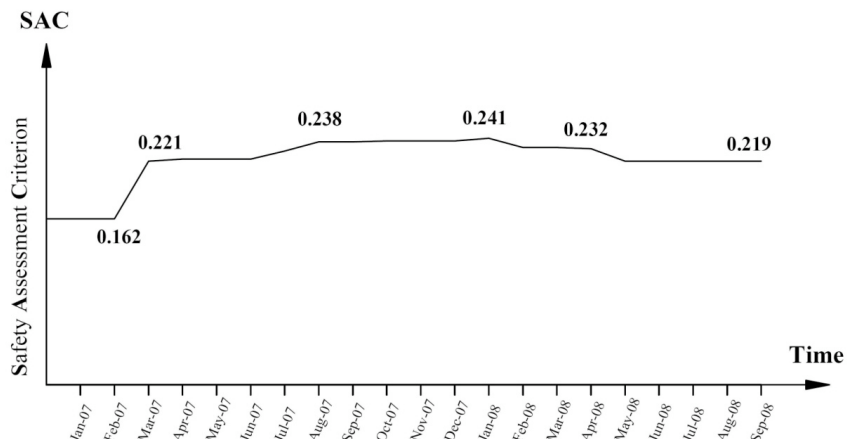


Fig. 8. Safety criterion during construction project for application example

Conclusions

This paper demonstrates that GIS is an effective tool for safety assessment, presenting a new process for quantifying hazards associated with the construction site layout. The key role of improving safety during project execution is the procedure to assess safety as one of the main activities of the construction managers nowadays. In this paper, a new method for dynamic safety assessment has been proposed using GIS and taking into account the impact of conflicts between hazard areas due to overlaps between TFs. An application example has been analyzed in this paper. Because GIS provides the means to handle spatial data such as geometric shapes and topological relationship, and perform spatial analysis, the proposed method for safety assessment can be easily performed. However, its feasibility and validation need to be proved by implementing it by a real construction management team on a real project. The writers are planning to test this method by implementing it on a real project.

Notation

The following symbols are used in this paper:

A_i = the area of intersection I ;

A_T = total area of construction site;

A_{z1} = total area of zone 1;

A_{z2^*} = total area of zone 2*;

C_i = safety criterion for the major TFs;

CS_k^z = the area of safety performance due to crane k in Zone z ;

d_{ij} = distance between hazardous material i and related TF j ;

d_r = recommended distance between hazardous material i and related TF j ;

H = reach of the crane;

H_i = the area of storage for hazardous material;

I_i = relative weight of intersection area i that ranges from 0 to 100%;

J = length of the crane jib;

K = total number of cranes on site;

m = ratio between the risk of falling objects from crane and the risk of crane collapse;

O_{ij} = overlapping area between facility i and j ;

SAC = safety assessment criterion;

V_{ij} = 0% for low, 50% for medium, and 100% for high sensitivity;

w_i = relative weight of facility i ;

w_j = relative weight of facility j ;

θ_i = angle measured clockwise from the north direction;

θ_f = set of crane operating angles;

θ_u =set of unfeasible crane operating angles.

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Use of Ground Penetrating Radar for Accurate Concrete Thickness Measurements

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Abstract

Ground Penetrating Radar (GPR) is commonly used for locating abnormalities or as a means of quality control in buildings and infrastructures. The objective of this project is to accurately detect the thickness of concrete pavement sections and to minimize the need for core samples using GPR. The project is sponsored by the Nebraska Department of Roads and several project sites in Nebraska are utilized for case studies. Through the use of the SIR-3000 GPR equipment and a 1500 MHz antenna, several pavement sections are scanned, validation cores are taken, and accuracy is evaluated. Laboratory and field experiments are also utilized to develop practical and inexpensive means to increase the accuracy of the GPR scans and further the confidence in the use of the system by professionals. Through the use of a strong reflection material placed at the bottom of the pavement, the thickness of the pavement may be determined accurately. With fewer core samples and a strong ground reflection at the bottom of the concrete, the GPR data can be calibrated for variances in concrete mixture, water content, and ground conditions. The study is still ongoing; however, the preliminary results show that with the use of inexpensive construction adjustments, such as use of a high reflection metal at the bottom of a concrete section, an accurate thickness measurement can be gathered without as many destructive core samples.

Introduction

The current method used for measuring the thickness of the concrete is limited and destructive. To measure the thickness of concrete pavement Nebraska Department of Roads (NDOR) adopts the ASTM C174 laboratory test method using drilled cores. Pavement cores are extracted every 720 ft and taken to the lab to determine the compliance of concrete construction with design specifications. A three-point calliper device is used to make length measurement at the center of the specimen and at eight additional points equally spaced along the circumference of the specimen (ASTM 2006a). The average of the nine measurements expressed to the nearest 0.1" is reported as the depth of the concrete core.

Although this procedure results in relatively accurate thickness measurements of concrete pavement, it only provides local information as cores are extracted approximately every 720 ft depending on the project importance. In addition, core extraction is a destructive process that is time consuming, hazardous, and it disturbs

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traffic. Therefore, there is a need for a nondestructive alternative for pavement thickness measurement that can provide continuous information along the pavement section in a rapid and cost effective manner. Ground penetrating radar (GPR) technology offers a solution to this need; however, as any other device, it has some limitations that must be overcome. The objective of this study is to investigate the feasibility of using GPR on a routine basis to measure the thickness of concrete pavements. The study is still ongoing and two case studies are carried out so far. In the first case study, ten sections of conventional concrete pavement are scanned and cores are taken at the center of each section. The resulting scans are evaluated using different dielectric constants, and resulting depths are compared to those from the core samples. Second case study involves use of metallic rings at the bottom of the concrete pavement, to provide a stronger reflection marking the depth of the section and potentially increasing the accuracy of the measurements. After this case study, the project team cast a study slab to provide a laboratory for several scans, and several metal objects are placed at the bottom of the slab. The scans on this study slab are ongoing. Once satisfactory results are gathered, these lab results will then be applied to sections of highway in Nebraska in the spring of 2008, and it is anticipated that the pavement thickness will be measured with minimal error.

Background

The GPR method involves the transmission of electromagnetic waves into the material under investigation and as a result of scans from the surface; the changes in the dielectric properties of materials can be identified. The reflections of these waves at the interfaces and the corners of the objects within the material are analyzed to determine the location (horizontal distance from a reference point) and depth (vertical distance from the surface) of the detected interfaces and buried objects. GPR can also be used to differentiate layers of material and to determine certain properties of the materials. In order to accomplish this, additional site specific information is needed such as direct depth measurements or relative permittivity of the material under investigation.

GPR utilizes the propagation of short pulse radar waves (pulse duration less than 1 ns) through the layers of materials under investigation. Figure 1 shows a radar signal that is emitted via an antenna into a structure composed of three different materials. Signals are reflected at the interfaces between the material layers. Reflected signals are received by the same antenna to present one scan or trace. Several scans are taken at different locations on the investigated structure and their data are recorded in the storage device of the central unit. These data are then processed and displayed on a monitor for further analysis (manual or automatic interpretation). Analyzed GPR data can reveal significant information about the materials within the structure and their condition.

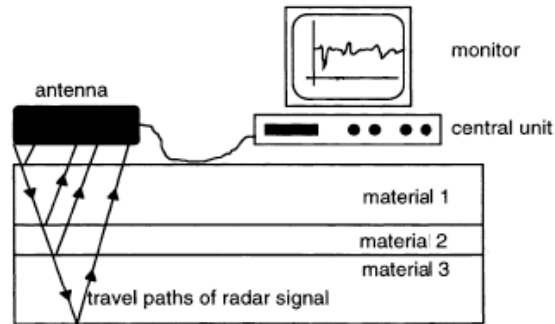


Figure 1: GPR Principals

From the electromagnetic standpoint, materials can be categorized as metallic or dielectric. Metallic materials have high conductivity and attenuate electromagnetic waves to a great extent resulting in shallow penetration, while dielectric materials have low conductivity and attenuate electromagnetic waves to a limited extent resulting in deep penetration. The relative dielectric constant of a particular material (ϵ_r , sometimes called relative permittivity) is the ratio of permittivity of the material to permittivity of vacuum ($\epsilon_0 = 8.854 \times 10^{-12}$ F/m). Although the transition from metallic to dielectric is gradual, this relative permittivity is used to indicate the nature of the material (high value for metallic and low value for dielectric). Table 1 lists the relative permittivity of different materials at an electromagnetic frequency of 1 GHz.

Table 1: Relative permittivity of different material at a frequency of 1 GHz

Material	Relative Permittivity
Air	1
Dry Masonry	3-5
Moist Masonry	5-26
Dry Concrete	5-8
Moist Concrete	8-16
Asphalt	3-5
Granite	5-7
Basalt	8
PVC	3
Dry clay/ dry sands	4-8
Wet sands/wet clay	16-32
Ice	3-5
Water	81

The propagation velocity (v) of a transmitted radar signal through a material is a function of its relative permittivity (ϵ_r) and relative magnetic permeability (m_r), as well as the speed of sound (c) as follows:

$$v = \frac{c}{\sqrt{\epsilon_r m_r}} \tag{1}$$

In low-loss materials, as most of the dielectric materials, the relative magnetic permeability (μ_r) can be assumed to be unity. Therefore, if the relative permittivity of the material under investigation is known, the propagation velocity can be calculated using Equation 1. The propagation velocity of the waves within specific materials is then used to determine the thickness of each material layer using the two-way travel time recorded by the GPR antenna. The difference in time between the reflected signals at the top and bottom interfaces of the layer times the velocity gives the distance traveled by the wave, i.e. the thickness of the layer. It should be noted that relative permittivity of a material is frequency-dependent and is influenced by several parameters, such as the temperature, moisture, and salt content of the material. These parameters have to be considered through calibration before calculating the velocity in order to obtain accurate thickness measurements.

When the incident signal meets the interface between two materials with different dielectric constants, part of the incident energy is reflected, while the other part is transmitted. The amount of reflected and transmitted energy is determined by the reflection and transmission coefficients (R and T) respectively. These coefficients are dependent on the relative impedance of the two materials (z_{r1} , z_{r2}), which are functions of the dielectric constant of the materials (ϵ_{r1} , ϵ_{r2}). These coefficients are calculated as follows:

$$R = \frac{z_{r1} - z_{r2}}{z_{r1} + z_{r2}} \quad [2]$$

$$T = 1 - R \quad [3]$$

$$z_r = \sqrt{\frac{m_0}{\epsilon_0 \epsilon_r}} \quad [4]$$

where, $m_0 = 4\pi * 10^{-7}$ H/m, is the magnetic permeability of free space.

As can be deduced from equations 2, 3, and 4, the smaller the difference in the dielectric constant of the two materials, the smaller the reflection coefficient and the larger the transmission coefficient. This means that the change of amplitude of the energy (i.e. attenuation) reflected from the interface between two materials is a good indicator of the properties of these materials. As the incident energy continues to penetrate other materials and meets successive interfaces, other reflections are sent back to the antenna and recorded over time to generate the waveform. Measuring the time and amplitude of reflections (peaks or valleys) in the waveform facilitates the determination of layer thicknesses, depth of buried objects, and changes in material properties (Eq. 1, 2, 3, and 4), which are the basic purposes of using the GPR technology.

Many different types of GPR and post-processing techniques have been researched to accurately measure the depth of concrete. Al-Qadi *et. al.* (2001) successfully measured the thickness of flexible pavement using two GPR systems simultaneously, air-coupled and ground-coupled, collected at 16 kph and 1 scan per 110 mm. Cores

were then taken to verify the data, and the average error was only 6.7%. By using GPR and PAVLAYER software, Maser (1996) measured as-built conditions of pavement with a +/- 7.5% accuracy when compared to core samples. Mesher *et al.* (1995) used a GPR system called "Road Radar" to measure pavement thickness on three sites. The systems consisted of a 2.5 GHz antenna and semi automated software for processing large sets of data and was self calibrating. An average R^2 value greater than .9 for all three sites was found by using linear regression statistical analysis. Louzili's optimization involves calculating the reflected frequency spectrum using the thickness and dielectric constants. When this method was applied on three pavement sections of known layer thickness, the error was less than 3.6%.

In contrast to the above studies, the goal of this project is to develop practical strategies that the state departments of roads can adopt in routine construction and quality control procedures to accurately monitor the thickness of pavements.

Case Studies

Two case studies are carried out within the scope of this project to date. These studies and the respective findings are explained in this section.

Case Study 1

The first case study involved scans on a section of highway outside of Hooper, Nebraska, which were collected on September 14, 2007. A 1500 MHz antenna was used with the SIR 3000 system using the *concrete scan* mode. Ten stations were scanned where cores had been previously taken and measured by NDOR. The scans were taken directly over the core location and were approximately 20 feet long, 10 feet before and 10 feet after the core sample location. SIR 3000 was set to 200 scans per second, which correlates to 60 scans per linear foot.

The standard dielectric for dry concrete is between 5 and 8, and for moist concrete it is between 8 and 16, as can be seen in Table 1. Therefore, the concrete under investigation can be assigned a dielectric within this range, and the final dielectric value can be selected through calibrations with the core testing results. Based on the GPR scans, two possible reflections are noted that could account for the bottom of the concrete slab as shown in Figures 2 and 3. These different possibilities for the bottom of the concrete change the dielectric substantially.

Scan shown in Figure 2 is around the core test station 322 + 43, which provides a depth of 10.2 inches. To achieve this depth at the layer of high reflection shown in Figure 2, a dielectric of 23 must be used, which is outside the known limits of moist-concrete dielectrics. Scan shown in Figure 3 is around the core test station 325 + 98, and points out a dielectric of 16. This scan not only estimates a better dielectric for concrete, but also has better clarity in the resulting scans in the sense that the filled concrete is visible in the form of a disturbance or change of material. The rest of the core locations, depth of each core and the corresponding dielectrics found from GPR results can be found in Table 2.

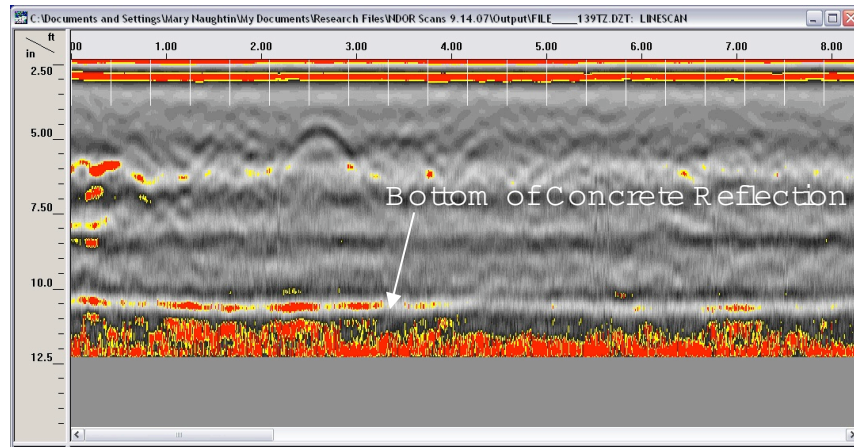


Figure 2: Scan at Station 322 + 43

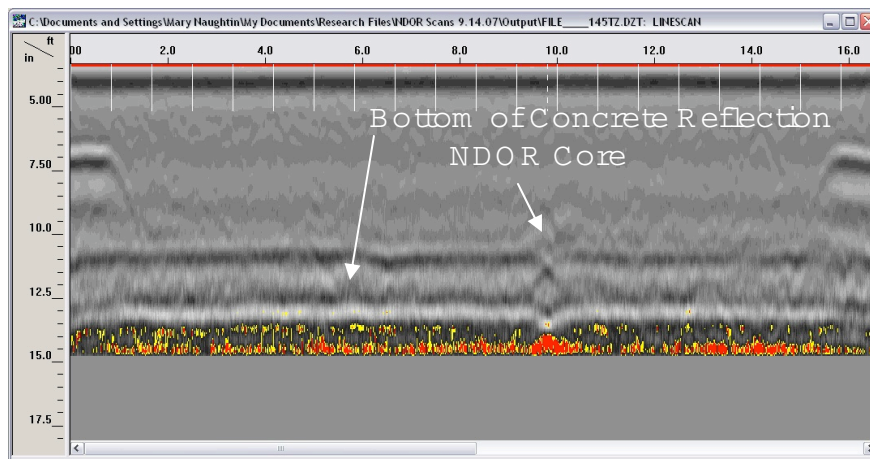


Figure 3: Scan at Station 330 + 80

The standard dielectric for concrete is between 8 and 16, therefore all of the dielectrics determined from these scans are in the moist-concrete range. This is expected as the concrete was younger than 28 days old at the time of scanning. Moreover, the week before the scans, over an inch of rain had fallen on this area.

The general conclusion from Case Study 1 is such that, due to similar dielectric values of wet concrete and subsurface material (such as highly compacted soil), the scans show weak signal reflections and the interpretation of the results is difficult. The use of a metal piece at the bottom of the concrete would produce a strong, unique reflection, which can be easily identified and more reliable measurements of depth and dielectric will be possible. This idea is pursued in Case Study 2.

Table 2: Summary of Results in Case Study 1

Station	Core Depth (in.)	Dielectric	Comments
<i>dielectric determined using very bottom reflection</i>			
322 + 43	10.2	23	Figure 2
325 + 98	14.1	14	
326 + 40	11.6	24	poor bottom reflection
<i>dielectric determined using higher possible bottom reflection</i>			
327 + 20	11.0	19	
328 + 99	10.7	15	
330 + 80	10.6	16	Figure 3
330 + 57	10.9	15	poor bottom reflection
328 + 17	10.7	14	
324 + 70	10.9	13	
325 + 12	11.4	13	
AVERAGE		16.6	Indicates very moist concrete

Case Study 2

For the second Case Study, two metal rings were placed in two different locations on Highway 34 before the concrete was laid and their locations were recorded. On September 21, 2007, two grid scans are collected. Grid scans consist of multiple scans evenly spaced across a section of concrete as shown in Figure 4. All of the scans can then be compiled together in RADAN (post-processing software for GPR) to produce a 3D scan of the section. By compiling the scans to 3D, more information is provided, resulting in easier data analysis and more accurate findings.

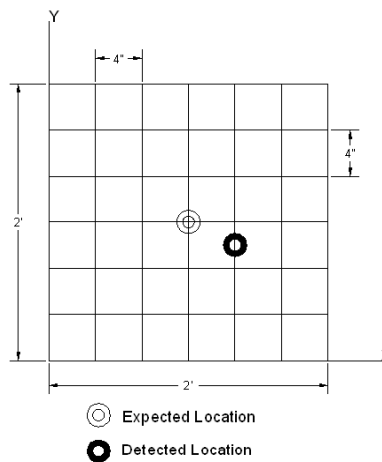


Figure 4: First grid scan

The first grid scan was a 2 ft x 2 ft grid made with 4 inch spacing. The bottom of the concrete was detected at 10 inches deep as indicated by the solid lines shown on the radar scans in Figure 5 and 6. The rectangles in Figure 5 and 6 indicate the metal ring reflections in the Y and X directions, respectively. The metal ring underneath the concrete was detected at a different horizontal location than expected, as shown in Figure 4. To verify that nothing was in the center of the concrete a core was taken and no object was found.

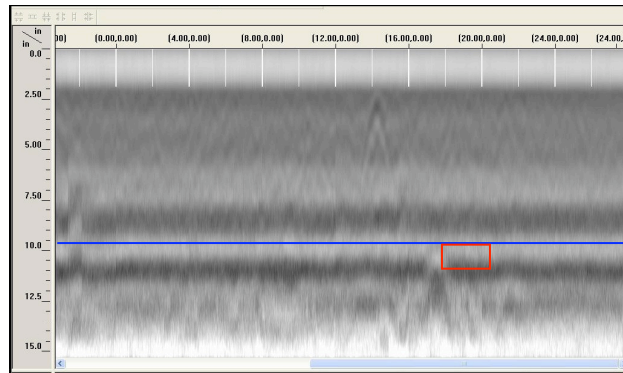


Figure 5: Y-scan

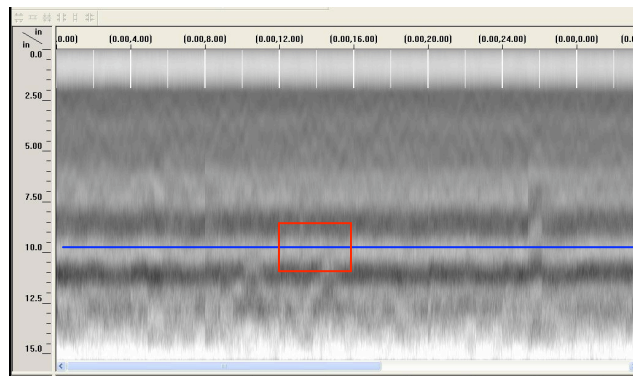


Figure 6: X-scan

The second grid scan was around the second metal ring previously placed under the pavement, and was also a 2 ft x 2 ft grid; however, this time a denser grid spacing of inch was used. The metal ring was also found in a different location than expected as shown in Figure 7. A metal detector was then used to verify the horizontal location of the metal ring as indicated by GPR. The bottom of the concrete was detected at 10.5 inches deep as shown in Figures 9 and 10 by the solid line. The rectangles in the radar scans (Figures 8 and 9) indicate the metal ring reflections in the X and Y directions, respectively.

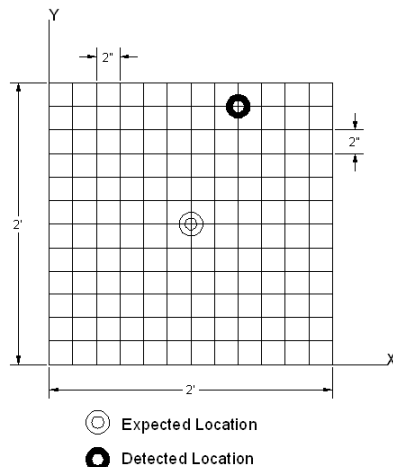


Figure 7: Second grid scan

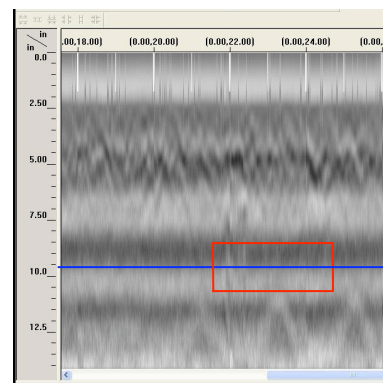


Figure 8: X-scans

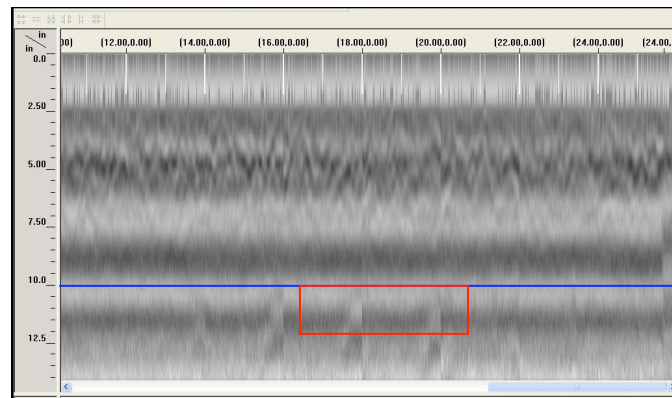


Figure 9: Y-scans

While the reflections from the metal object provide easier determination of the depth of the concrete pavements, the metal rings used in this first trial moved significantly resulting in substantial loss of time, thus reducing the efficiency of the methodology. As a result, the authors suggest use of a larger and linear metal object, or an object that can be secured to the base material. The benefit of a linear material would provide the additional benefit of only scanning in one direction, and reduce the time for setup and scans.

Concluding Remarks

With the goals of studying the feasibility and accuracy of GPR use in the detection of concrete sections, the project team carried out two case studies.

The first study proves the difficulty of accurate depth determination, especially soon after the construction stage as the curing concrete can have a range of dielectric values, which in turn mean a range of thicknesses. While the relatively poorer results of GPR in curing concrete is expected, a methodology to overcome this limitation is needed because soon-after-construction is when NDOR needs to perform quality control on the contractors' work.

The second study utilizes a high reflection object at the bottom of the concrete pavement. This study proves the increased accuracy of depth detection; however, the choice of the metal object must be further optimized. After the two case studies, the project members designed a test slab with various metal objects buried at the base of the concrete with known horizontal locations. As the project continues, scans will be gathered from this test slab and feasibility of each metal device will be studied considering the following parameters: ease of scan setup, detection of accurate pavement depth and cost of the metal object used.

Acknowledgements

The authors would like to express their gratitude to the Nebraska Department of Roads for the financial and technical support provided throughout the project.

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BIM and MIMS: Emerging Technologies

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ABSTRACT

Increasing complexity of our codes and standards as we delve deeper in the information era makes conforming accurately to the specifications a cumbersome process. The concept of intelligent digital design within the context of structural engineering in the 21st. century will provide key solutions in simplifying the engineering design process. The intelligent structural design described in this work focuses on the relationship among the Information Management Systems in general and Masonry Information Management System (MIMS) in particular along with the Building Information Modeling (BIM). By providing more accurate information and interactive structural elements, intelligent design offers engineers and consultants efficient and unambiguous methods to validate design assumptions against specifications and alternatives in a digital environment to achieve design objectives.

INTRODUCTION

Evolution of technology has continuously engaged the engineering profession over the past several centuries. From the wood covered lead pencil, which was first mass-produced in 1662 to the recently surfaced Building Information Technology (BIM) all note down the revolutionary impact of technology on structural engineering practice. Presently, the information era shows that a proper information infrastructure offers many benefits that help produce and share high quality engineering data and decision management systems more cost-effectively and more efficiently.

The process of detailed design of a structural system made of a large number of components is quite involved. Further, since design is an open-ended problem, i.e. in general there are a large number of design alternatives satisfying the complex Code provisions; the selection of the optimum design becomes an extremely challenging problem. The key approach to this problem is by the simplifying the design process. This includes how accurately and efficiently can complex Code specifications be

translated into a flexible and actionable format that would allow simple interoperability with our present digital design environment to facilitate simple design. This work addresses these issues using Masonry Information Management Systems and Building Information Modeling. Intelligent design is a term used to describe the integration of Code specifications, structural computational software, and the Building Information Modeling in a coherent digital framework.

BACKGROUND

Engineering Information Management is defined as a process that focuses on knowledge related activities to facilitate knowledge creation, capture, transformation and use, with the ultimate goal of leveraging engineering design and analysis decision to achieve optimum solutions.

In structural design, there has been much research in the area of design knowledge reuse. Very little has focused on handling complexity of Code provision and integration of different digital design tools. Most of the research in the past was focused on area of design knowledge reuse in structural engineering, with most of the approaches previously tested originating from Artificial Intelligence (AI). In the 1970s and '80s a large number of rule-based expert systems were developed for the purpose of knowledge reuse. However, these systems were not entirely successful because of the difficulties in producing a formal representation of the knowledge (Brandon 1990, Pu 1993, Davenport et al., 1998). The knowledge had first to be acquired from experienced designers and then generalized and transformed into rules. Using cases for design knowledge reuse is commonly termed Case-Based Design (CBD) (Johansson (2000)). Although many of these systems are useful in solving the specific problem they are intended for, CBD systems are seldom used in practice. One of the main reasons is that the representations used in these systems are system specific and differ considerably from representations used by the ordinary designer when documenting design information. This makes it difficult to achieve an automatic translation.

Recently, researchers start to work on engineering information management system to provide an efficient means for data storage, retrieval, data transfer, exchange and utilization to enhance and simplify design and construction process (Nawari, 2007). Good examples are the efforts made by many state and federal agencies to develop such information management systems. For instance, state DOT geotechnical specialists (Lefchik et.al. 2006) are pursuing means to better manage geotechnical data (e.g., boring logs, lab test data), geologic hazards (e.g., landslides, rockfalls, mine subsidence), and assets (e.g., walls, reinforced slopes). Several states have instituted electronic data management systems to manage geotechnical data for large projects. Some states have hazard management systems in place. And some states are

beginning to develop geotechnical asset management systems for elements such as piling or retaining walls.

OBJECTIVES

The ultimate goal is to enable an environment where virtual prototyping of whole structures or parts of the structures prior to their construction, can be examined against code specifications and requirements digitally in an efficient manner. This includes inter-operability, visualization, automation, collaboration and integration of different design tools and technologies. The paper discusses the implementation of two sets of information technologies, namely Masonry Information Management Systems (MIMS), and Building Information Modeling (BIM). The purpose of this implementation is to seamlessly bridge the gap between the Standard and Code specifications, structural computational software, and the Building Information Modeling.

MIMS

Designers of Masonry structures generally use vast amount of data that are distributed across various sources. For the most part, however, the fundamental value of these scattered data remains and their relationship hidden and unutilized. Designers often have to wait days or weeks for responses from different data sources. Accessing the appropriate information from these databases at the time when it is needed, and in the appropriate format, is not an easy task. The Masonry Information Management System provides timely access to these data and their interrelationships in order to facilitate effective masonry design and analysis (Nawari, 2007).

MIMS is a set of components, which interacts exchanging internal and external information with support of transaction and mining tools. It has the role to ensure the coupling between the system of operation and the system control: its function is to produce and memorize information as representations of the activity of the physical system then to place them at the disposal of the system control.

Masonry construction has been continuously growing over the past thousands of years. This trend will also continue in the future. Consequently, it is necessary to develop a comprehensive management system of an ever-growing masonry construction. This problem is made more complex given the accelerating rate of scientific and technical discovery, typified by the ever-shortening time period for the doubling of information (currently estimated at 18 months). With limited resources, engineers, contractors, developers, and builders are striving to improve the efficiency of their operations and better manage their staff time, funds, design and construction by delivering critical information in a timely fashion. Information Management systems can provide a means to assist in managing their construction and structural systems data while improving decision-making and performance. These systems

provide an efficient means for data storage, retrieval, data transfer, exchange and utilization to enhance design and construction of masonry structures.

BIM

The building Information Model (BIM) provides the 3-D objects library of the physical building. According to the National Institute of Building Sciences (NIBS), the BIM is defined as: Building Information Model, or BIM, utilizes cutting-edge digital technology to establish a computable representation of all the physical and functional characteristics of a facility and its related project/life-cycle information, and is intended to be a repository of information for the facility owner/operator to use and maintain throughout the lifetime of the structure. In essence, BIM is a way to construct a building virtually, before building it in the real world. BIM can also be described as an amalgamation of design information technologies that, when properly utilized, represent every building component in a virtual environment. This results in a 3-D object library that represents all components of the physical structure.

INTELLIGENT DESIGN

Intelligent design is not merely a matter of producing better structural design; it has to do with accurately translating the complex specifications and complaints in our Codes into a simple process. As technology advances, new constructions and computations techniques emerge, code specifications grow and client expectations are in turn extremely higher. All these suggest some kind of intelligent structural design system that accurately and efficiently transforms these complexities into a simple process. Figure 1 depicts the relationship between the advancement of information management systems and the structural design process and the tendency toward a migration path. This has clearly created the necessity for scalable, interoperable, flexible, and open intelligent structural design system. Looking back at the definition of Machines, as given by French Biochemist and Nobel Prize, Jacques Lucien Monod (1910-1976), are "purposeful aggregates of matter that, utilizing energy performs specific tasks." By this authoritative definition and by replacing matter by knowledge base and energy by information technology, intelligent design systems can be recognized as smart machines as well.

Integration of the enabling technologies such as structural computational software, Engineering Information Systems and the building information modeling (BIM) represent the core of such an intelligent design system. This, intelligent design process seeks to bridge the gap between disparate structural design tools to build a common digital design platform. Thus, improves the structural design quality and profitability from both engineers and clients perspective. Figure 2 illustrates this concept of Intelligent Structural Design (ISD) process. The main features of the ISD System include interoperability, adaptability, reuse information and infer actions, future proof, simplicity and ease of use.

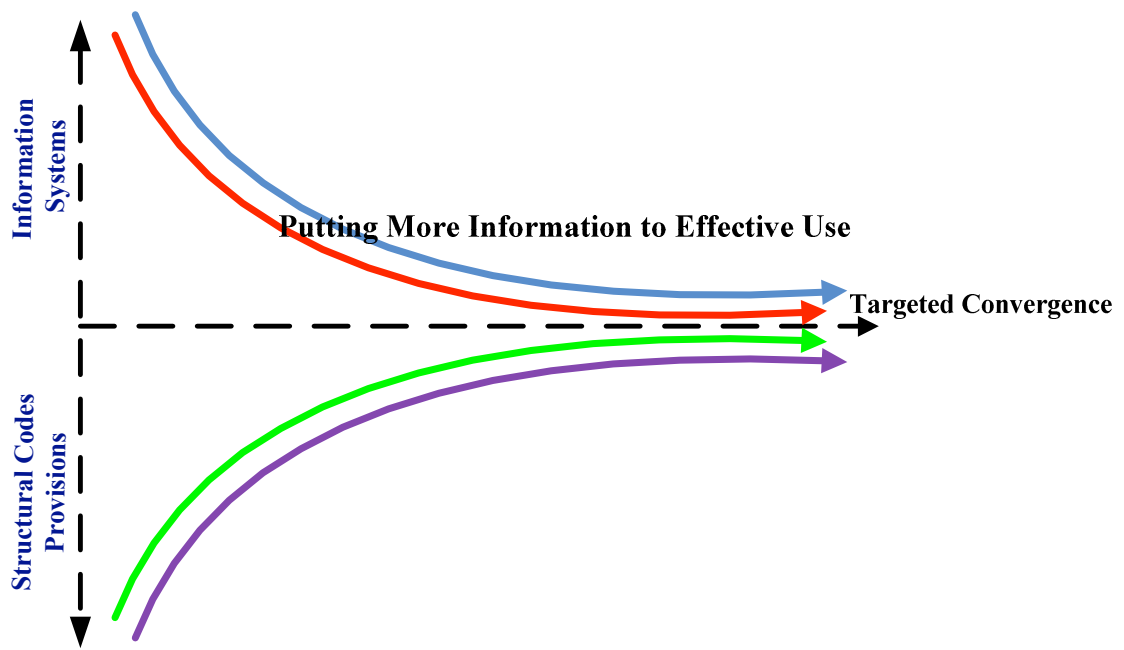


Figure 1. Integration of Information Technology and Structural Design.

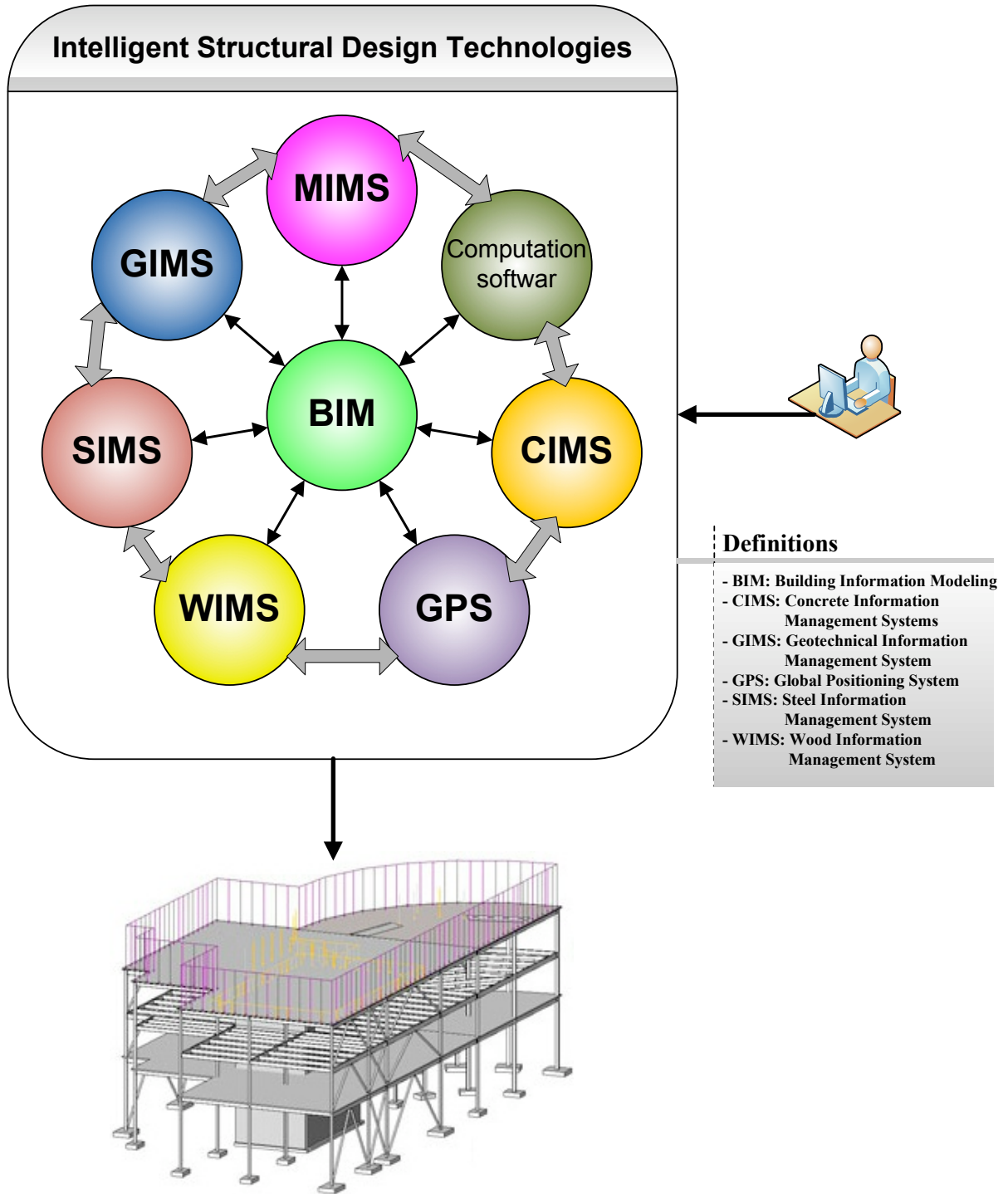


Figure 2. Intelligent Structural Design (ISD) System.

DISCUSSION

The structural design profession has undergone dramatic changes over the past years. With computer-Aided Design, complex building codes, fast-track construction, and many younger engineers taking on more responsibility earlier in their career, the need for simple intelligent design process has never been greater. Such a process will result in better design, increased profitability, high quality contract documents, and increased client satisfaction.

Currently, such goals are achieved by relying on experience, skills, oversight and expertise of highly trained engineers. This generally ends up in higher cost and less efficiency. Young engineers spent several years learning about the code and specifications under the guidance of experienced designer before more responsibilities can be delegated. Further more the design review process is taking more time as our code provisions grow in complexity and frequent updates along with rapid construction technology advancements. The introduction of ISD will greatly improve this traditional practice by simplifying the access to code provisions and complaints checks. Converting Code and Standards from a flat rigid format into dynamic actionable format does play the key role in the intelligent design concept. The next discussion demonstrates the role of Masonry Information Management System (MIMS) in the design of masonry structure.

Masonry structure is an organized assemblage of interdependent parts, which work together to form a building component. For instance, a masonry wall may be made of a combination of clay brick, concrete masonry units, stone, calcium silicate units, steel reinforcements, etc. An intelligent design system should provide the intricacies of each material and what detailing implications and specifications those characteristics may require. For example, figure 3 depicts a short representative data from MIMS. The data is represented in a dynamic and actionable format that allow for easy interoperability and different query and mining proceedings. A simple example can be illustrated by the application of the recently released LINQ language technology from Microsoft (2008). Language-integrated query (LINQ) allows *query expressions* to benefit from the rich metadata, compile-time syntax checking, static typing that was previously available only to imperative code (like C++, C# and VB.Net). Language-integrated query also allows a single general purpose declarative query facility to be applied to all in-memory information, not just information from external sources. XQuery is a component of LINQ, which represent a modernized in-memory XML programming API designed to take advantage of the latest .NET Framework language innovations. It provides both DOM and XQuery/XPath like functionality in a consistent programming experience across the different LINQ-

```

from m in MIMS.Elements("Materials").Elements("Concrete").Elements("Units"),
     s in m.Element("Solid").Element("Size")
where (string) m.Element("Solid").Element("Strength").Element("Compressive") >= "2000" &&
      s.Value.StartsWith("7")
orderby (string) s.Attribute("id")
select m;

```

enabled data access technologies. The following code snip illustrates some of the rich utilization:

```

- <MIMS>
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    - <Concrete>
      - <Units>
        - <Solid>
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            <length>15 5/8</length>
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          <average-area>91.5 sq.in</average-area>
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          </Strength>
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          </Deformation>
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          <ASTM>C 55</ASTM>
        </Solid>
        - <Hollow>
          - <type id="Splitter">
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</MIMS>

```

Figure 3. An Example of MIMS data Representation

The object library in typical building information modeling (BIM) environment can be linked to MIMS to deliver the necessary data and details about the masonry structural and architectural components of the building. For instance, in the case of a load bearing wall of the building, the geometry of the wall can be imported from the BIM into MIMS (see Figure 4). Then, over the course of the design process, information about the masonry system is provided by MIMS in various forms: data defining the physical, mechanical and environmental properties of clay brick, concrete masonry units, stone, calcium silicate units, etc. the, drawings detailing wall patterns, bond, single-wythe or multiple-wythe, cavity, solid grouted or partially grouted, reinforced or unreinforced walls, the and MSJC requirements. Table 1 blow shows an example of the structural design requirements according to the 2005 MSJC Code that can be obtained from MIMS.

Table 1. Bearing Wall Design Requirements (Allowable Stress Design).

Design Requirement	2005 MSJC Code Reference
Allowable stress in masonry (due to axial compression, flexure or combination) $\leq \frac{1}{3} f'_m$	Section 2.3.3.1 and 2.3.3.2
One-way shear out of plane: $f_v = \frac{V}{bd} \leq 3.0 \sqrt{f'_m} \leq 150 \text{ psi (1.034 MPa)}$	Section 2.3.5.1.1 and 2.3.5.2.3

Where,

f'_m = specified compressive strength of masonry, psi(MPa)

f_v = calculated shear stress in masonry, psi (MPa)

V = shear force, lb (N)

b, d = width of section and effective depth, in (mm)

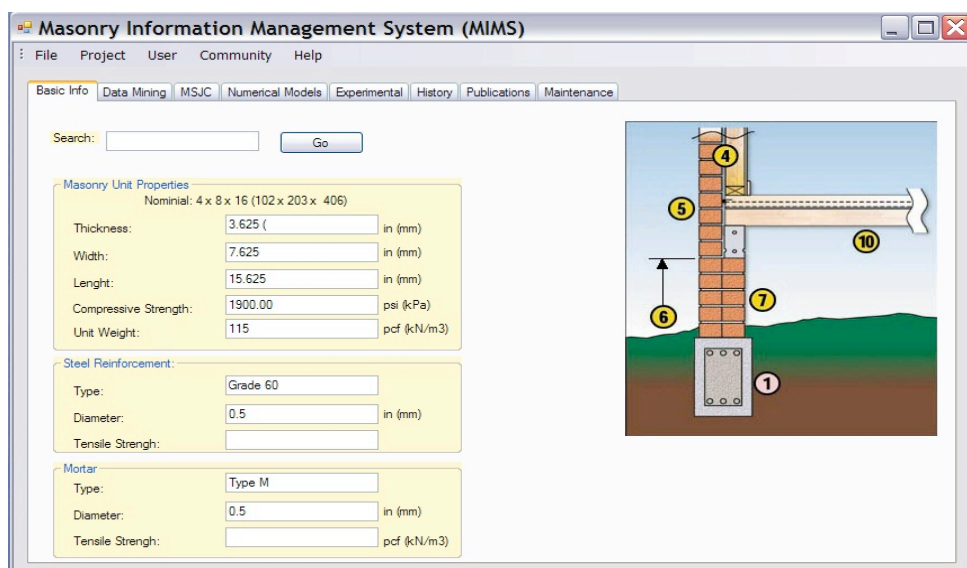


Figure 4. MIMS Design support for the Bearing Wall Object

CONCLUSIONS

The introduction of information management systems in structural engineering design has an important role in simplifying the current design process. By breaking through the precincts of Code and Standard provisions, structural computational software, and the Building Information Modeling a solution to insurmountable hurdle can be achieved. An intelligent structural design environment can be established by Linking and sharing engineering information management systems (i.e. MIMS), and other digital technologies with the Building Information Modeling (BIM). Such environment will provide significant improvement in design, construction, and sophisticated structural decision-support systems.

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AEI: Architectural Engineering Institute 2008 Conference
Title: Designing Inside (Out)

Abstract

This paper compares typical architectural comprehensive studios to a design project that I have created in which the traditional design order has been reversed. The traditional process for most academic building design projects is to study a project's macro-scaled issues first and eventually focus in to the building details. Through the macro-to-micro process, the building systems, structure, and assemblages are reactionary to the building design. In this studio's reversed order, students started at the building core, designing their building around an investigation of building systems. Next, students examined building structure, construction methods, assemblages, building organization, and then finally external context. This paper demonstrates that reversing the typical design order results in designs that have a greater conceptual understanding of building systems, assemblages, and materials than traditional design method.

Although the paper focuses specifically the design studios within architectural programs, I believe that the concepts of reversing our traditional order of design, designing inside (out), can be transferable to all building design professionals.

Designing Inside (Out)

By Dana K. Gulling, RA Assistant Professor University of New Mexico

The problem for many building designers is that despite the technological advances available—such as digital fabrication, new building materials, building information and parametric modeling—our process for architectural design still remains much the same. Traditionally, we design from the outside in, from the macro to the micro. Because the design process is traditionally completed in this way, the building systems, structure, and assemblages are reactionary to the building design and do not operate as efficiently as possible. The reality of the building is rarely investigated before the building design. Why do people within the building industry believe that the only way to design is to conceptualize building form and then figure out how it will be a reality? As an alternative, I would argue that the better design intention is to understand the building's reality and then conceptualize the building's form. This paper will investigate systems-integration comprehensive studios typical in most architectural programs and will compare those projects to a comprehensive design project that I have created in which the traditional design order has been reversed. It is my assertion that by reversing the traditional process of working from macro to micro (or conceptual to reality) we can design buildings that are more comprehensive and more innovative. I believe that this is especially true in the design of building systems, discovery of new materials and fabrication technologies, and building delivery methods.

This paper represents the results of a fourth-year, comprehensive studio that I created and oversaw while I was a professor at the Savannah College of Art and Design, or SCAD. What I am describing as the comprehensive studio (sometimes also referred to as the capstone studio) is typically the final studio in the architecture studio sequence. The intention of this studio is to provide a studio project that encourages the students to resolve their conceptual building design work with the reality of building assemblages and structure. This studio is a demonstration of the students' ability to convey an inclusive understanding of building through design. They need to demonstrate a clear understanding of building structure, assemblage, and some suggestion of building systems. Oftentimes these capstone studios will integrate knowledge and skills learned in the student's support courses. This can be done by requiring co-requisites to be paired with the capstone studio (such is the case at the University of Maryland and Catholic University of America) or can require a number of completed pre-requisites before entering the capstone studio (such is the case at Savannah College of Art and Design, SCAD and at the University of Notre Dame). These studios may be considered to be thesis projects, master's studios, or senior studios. Typically, the capstone studio project utilizes a complex building typology, but simple enough so that students do not spend all of their time on the building's conceptual design and space planning, but on the development of the design details. Of-

tentimes, the studio project culminates in the development of building assemblages through the demonstration of a series of walls sections, structural plans, or axonometric studies; some indication of passive and active ventilation systems may also be included. *See Figure 1.*

The traditional method for architectural design has been that we first analyze the external context of our building, be it urban, suburban, or rural. From the outside we work inwards, investigating the building through plan, section, and elevation. Then we zoom in further, looking at the building in more detail through interior elevations, room layouts, and bay details. Focusing further, we study building details, the joints between materials, and discuss construction methods and tolerances. Rarely is this traditional design method questioned, and in fact this process is reinforced by our education, our clients, our consultants, and the contractors. Problems that are created from working from the macro to the micro are exacerbated by the convention of engaging consultants late into the design process. Therefore an architect may complete a building's conceptual or even schematic design before investigating any new information or conceptual ideas that the consultants may bring to the project.

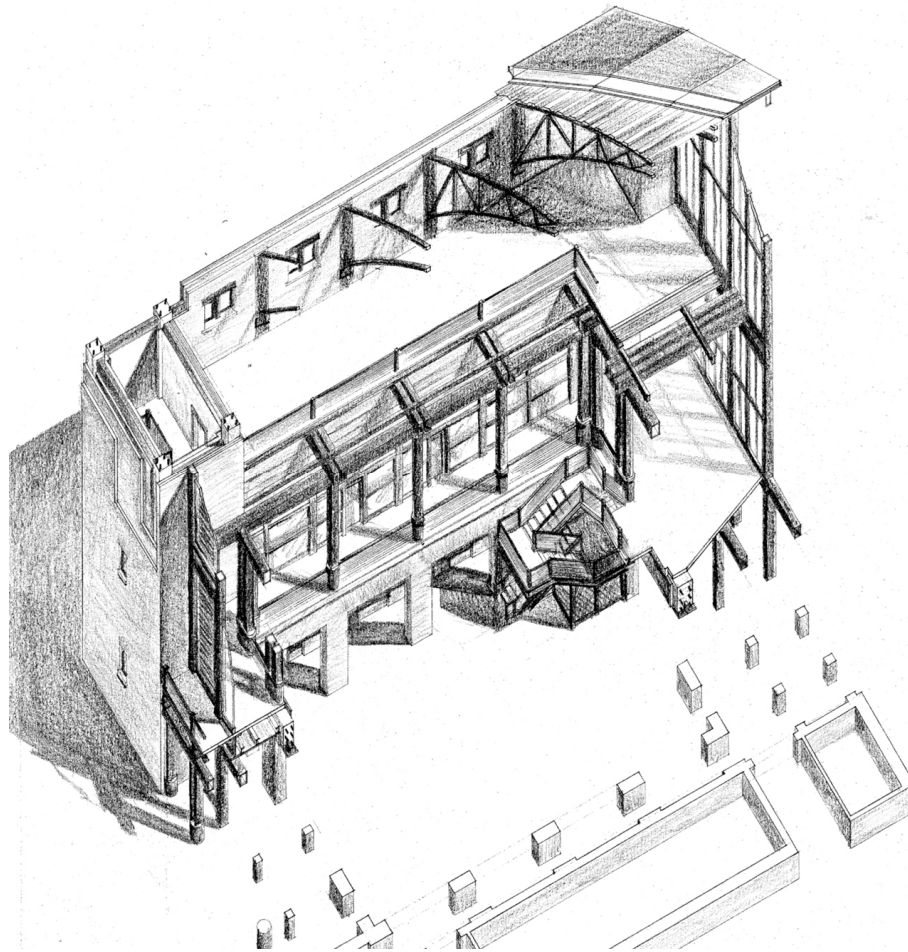


Figure 1 *Sample cut-way axonometric image from a comprehensive studio. (Image by author)*

There has been a resurgence among many architects to refocus building designers on building integration, with the majority of proponents looking toward Building Information Modeling (BIM) as a solution. The promise of BIM is the potential of a seamless integration of all building design disciplines into one virtual model. With BIM, building designers now have a model that incorporates building design with analysis of systems' performances, structural efficiencies, building costs, component deliveries, and fabrication technologies. BIM is a tool that allows us to have a more comprehensive approach to building design. However, BIM and its associated analysis software can only react to an already-conceived design and does not necessarily encourage designers to develop new materials, use building systems as a method to generative design, or to study inventive building assemblages. Instead, building designers still rely on the innovations developed by other industries to demonstrate new materials and production technologies, and then we incorporate their inventions within the BIM model for further study. In the 21st century we are moving into a period of unprecedented technical innovation for building components and yet architects and engineers have yet to assume a leadership role in these areas. We remain reactive rather than proactive, and BIM reinforces the reactionary approach to design.

As described previously, the traditional process for building design is to study a project's macro-scaled issues first and eventually focus in to the building details, and this in turn becomes the methodology that we ask of our students. Similar to architectural practice, architecture students design the building and then solve the task of designing the structure and the systems. Through the macro-to-micro process, students are asked to incorporate building systems, structure, and assemblages into their building design. This practice necessitates that the building systems be reactive to the building design rather than proactive. Instead, it was my approach to this studio that used the building systems as a way to generate architectural design rather than distract from it. This is why I propose reversing the traditional order of design. I propose that students start at the building core, and design their building around a self-directed approach to and an investigation of building systems. From there, students are asked to investigate building structure, construction methods, assemblages, building organization, and then finally external context.

This paper will demonstrate that reversing the typical design order will result in student work that demonstrates a greater conceptual understanding of building systems, is more innovative in its study of building assemblages and materials, and investigates a greater range of potential project delivery methods. Although this paper is focused specifically on the design studios within SCAD's architectural program, I believe that the concepts of reversing our traditional order of design, by designing from the inside-out, can be transferable to all building design professionals. As an educator and architect, my intent in developing a studio based on this new methodology, allows those students to reconsider the traditional means of delivering a building. It is my hope that they will take this education with them into the field of practice and begin to transform the building design profession.

SCAD has a five-year accredited master's of architecture program; as a result of this, the students have a number of architecture accredited course requirements that

must be fulfilled to earn their degree, leaving little availability for liberal arts courses or college electives. So despite SCAD's architecture program being housed within an Art and Design College, students complete a number of technical requirements—such as structures, construction technology, and environmental controls—before their final quarter of their fourth year. SCAD's quarters are only 10 weeks in duration (versus a standard 16-week semester schedule) with studio class meeting for two, five-hour classes each week (versus three, 4-hour class each week standard to most architecture programs). This compressed schedule makes a traditional comprehensive studio too difficult for the students to complete within the college's schedule.

Because of the limitation of time, the project for this studio necessitated something small, but at the same time allowed students to address the building complexity necessary for a capstone project. I believed students should spend more time and effort on researching, understanding, organizing, and developing their building's systems rather than wrestling with a complex program or a large-scaled building. Therefore for this studio, students were asked to design a simple domestic unit, under 1,500 square feet, complete with a kitchen, bathroom, living space, and sleeping area.

Even through the scale of the unit was small, the project was quite challenging. Students were asked to design the unit using modular or off-site construction techniques. This challenged the students to not only consider the building design, but to also consider building fabrication, off-site construction, delivery, and assemblage. The building had to contend with additional structural loads due to movement, limitations in dimensions (based on size restrictions dictated by the US Department of Transportation), and construction sequencing. The studio broadly defined modular, with the project description stating "The architectural product may be considered to be either for purchase in its entirety and shipped as a whole to its site, or it may be considered as a kit-of-parts to be constructed on site by its purchaser."

Once students were ready to begin the design process, they were asked to design the unit's core first before designing the unit's shell, plan, or site. Based on early analysis and exercises, students were asked to design a building core that would contain an area for food preparation (kitchen) and personal hygiene (bathroom). The core was to contain specific equipment—such as a refrigerator, stove, sink, toilet, water heater, heating and/or air conditioning source, and bathing facilities. For all initial design work the students were to focus on the building core while the design of the remaining programmatic elements (living and sleeping areas) would be designed in response to the core. The title of this exercise was "Body Services"; just as the inner workings and interrelationships of the body (digestion, filtration, distribution, and regulation) are complex but with a simple organization, so too should be the unit core. As the assignment brief explained, "focus for the design of the prefabricated unit is to be on the resolution of the organization, practicality, materiality, serviceability, and dignity of the core." For this studio, the core was to be the central component of the unit's design—not just physically but conceptually. The core would be central to the unit's organization and operation, and the dignity of the core's design would direct the concepts for the design of the remainder of the unit.

The core did not simply serve the unit. Instead the core was an active network of services that supported the unit and ultimately those living inside of the unit. For studio, I emphasized the inherent beauty that could be extracted from the design of

the unit services. As a class we did not merely discuss how to plumb a bathroom, alternatively we discussed how to best organize a bathroom around greatly designed plumbing. Each building service (plumbing, electrical, and conditioning) was treated in this manner. The systems were studied and planned with respect. They were understood as organizers to the architecture.

Students used this qualitative method for understanding and designing the building systems. Students were not required nor were they encouraged to complete the calculations necessary to design the systems; instead conceptual sizing charts, as typically found in Edward Allen's *The Architect's Studio Companion*, were referenced as resources. This aversion to a quantitative method was purposeful. As architects, we are not responsible for, nor do we have any real inclination for, calculating the systems that we design. I believe that it better for architects to work with other discipline designers to detail the specifics of each system. By allowing design students to work with the concepts of the systems, without being overwhelmed by design specifics, they have the freedom to discover how building systems work. Through this pedagogy, student gain confidence to work with services or building methods they have yet experienced, develop new possible uses of existing services, or even invent new building systems.

To understand the three-dimensional aspects to the building core, students were asked to design the core using 1"=1'-0" study models. *See Figure 2*. Using this method for design, even the most basic students understood the spatial and design implications of these systems for their building. Issues such as HVAC space requirements, vent stack penetrations, coordination between services and architecture, and structural needs were better resolved using the model. Students clearly saw how much space service equipment utilized and they identified how it may be better consolidated.

As stated, the intention of the studio is to have the unit design respond to the design of the core and this intent is evidenced by many of the student's work, including Jessica Young. *See Figure 3*. As a result of earlier exercises, Young established her consumers as active retirees located in the Southeastern United States. Because of her consumers, Young designed a core that would be wheelchair accessible and would contain additional amenities necessary to meet her consumers' need (i.e. full

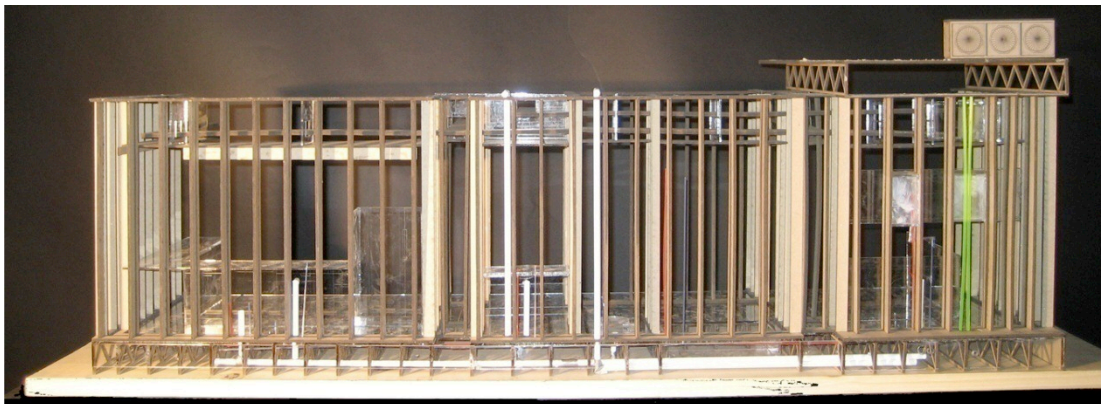


Figure 2 Sample image of 1"=1'-0" scale study model. Note all services and structure are exposed in order to understand the relationship between building systems. (Jessica Young, 2007)

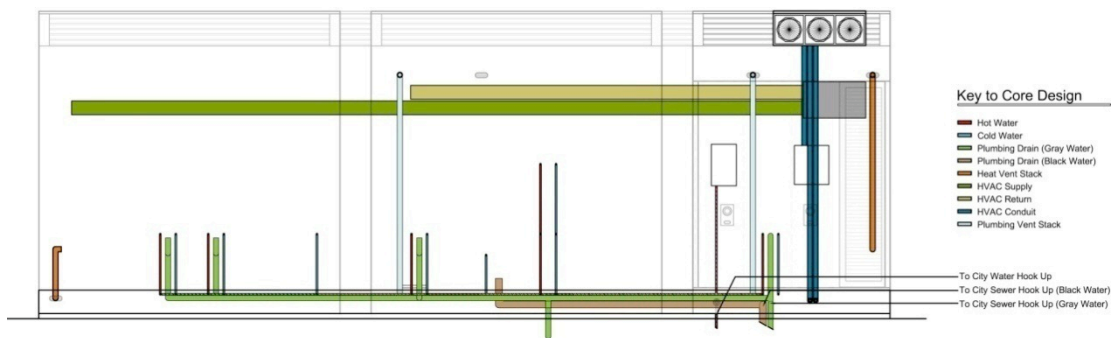


Figure 3 Building services are mapped over exterior elevation. (Young, 2007)

tub, dishwasher, washer/ dryer). She chose to organize all of her systems linearly, keeping all of her building utilities and services to the exterior wall. All of the services and equipment were located along the exterior wall and were designed with access panels for easy accessibility. For example, the horizontal plumbing and electrical services ran through the proposed wood floor trusses and the vertical connections to the core's fixtures would be accessed through exterior panels. The building services were treated much like the engine in a car, and with the systems being easily accessible they were also easily serviceable should any of the systems need maintenance, or replacement. The core as a self-structured linear service element contained all of the unit's services and circulation. Through this design, the core structurally supported and provided all services for the sleeping and living areas.

Some students use fairly standard building technologies for the design and construction of their building core. Even though the development of the cores themselves was fairly standard, the core's importance relative to the design and organization of the rest of the unit is not. A few of the students investigated non-traditional methodologies for creating and constructing the core. Michael Owens used a combination of inventive passive and active systems for his core in order to service his units. See Figure 4. Circular in shape, his core contains the plumbing, heating and electrical systems within its envelope, and also supports the building's precast concrete floor. Marketed for a region in Southwestern United States, Owens utilized such passive systems as thermal mass, water collection, stack ventilation (for cooling), solar heat gain (for heating), and a solar reflector. For his active systems he designed the roof to hold photovoltaic panels that would be connected to an anticipated electrical grid, and a pump for the cistern to bring the grey water up to the toilets.

The remarkable thing about Owens' project is not the simple combination of the active and passive systems that the core offers; instead it is that the active and passive systems work in conjunction with each other. The solar reflector directs the sun's rays to the portable hot water tank, where it heats the water. The heated water in turn heats the air at the top of the core, generating more heat at the top and thus offering more passive cooling through conduction. Additional photovoltaic panels are placed on the roof of the building and create the necessary electricity to offset the electricity required to power water pump at the cistern. Even the spatial allocation of the core's systems is efficiently overlapped with the water tank, exterior structural wall of the core, the stack vent, and the grey water collector utilizing the same space in plan.

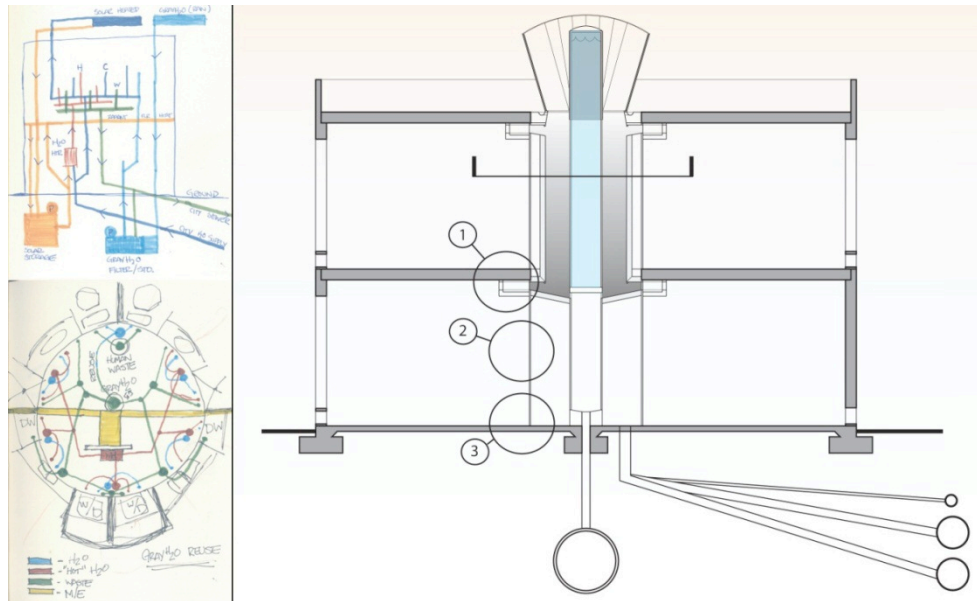


Figure 4 On the left are diagrams, organizing the services of the core. On the right is the core at the final review. (Michael Owen, 2005)

Scott Blew was also interested in adding sustainable features to his building core, while exploring innovated fabrication practices. In addition to a grey-water recovery system and photovoltaic panels, Blew investigated utilizing injection molding to construct the building core. See Figure 5. Injection molding would be used for the outer shell of the core as well as to create voids that would form the plumbing supply, waste, and vent stacks. Blew spatially organized all of the plumbing services within the core to meet the manufacturing requirements of this fabrication methodology. If purchased, additional options of storage and shelving could be added on either side of the bathroom portion of the core. These additions could be attached using epoxy and heat irons, giving a continuous surface and a water-tight bond between units. The continuous surface of the material allows for easy cleaning and reduced the number of joints where microbes could breed.

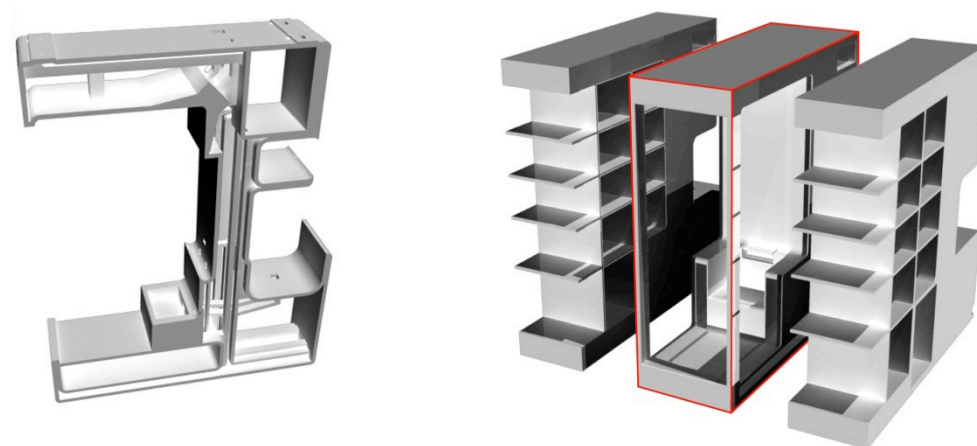


Figure 5 On the left is a section cut through the bathroom and kitchen sink; on the right is the bathroom unit (outlined in red) with the attachable storage units. (Scott Blew, 2007)

Similar to Owens' core, Blew was also interested in conserving space. In this quest he designed an extremely compact bathroom, with a molded plastic toilet with a fold-down sink that would cover the toilet. The tube-and-sleeve hinge for the sink would double as the drain, eliminating the fixed waste pipe and trap. The entire bathroom was no more than a shower stall, complete with an overhead fixture and the floor sloped to a trench drain along the back.

While many of the students, as demonstrated by Owens and Blew, were interested in designing an efficient core—both in terms of space and interrelations between the services, Luke Hellkamp took a contrasting approach. For spatial reasons and for the ability to customize his unit Hellkamp split his core. Instead of having the kitchen and bathroom programs back-to-back to easily consolidate the services, Hellkamp chose to face the bathroom and kitchen towards each other on either side of the unit, and support them off the building's frame. *See Figure 6.* The unit's hollow frames served as the chases for plumbing, electrical service, and were both the supply and return ducts for the forced air system. A custom-designed, split condenser and fan coil unit were integrated in the design of the kitchen and connected directly to the unit's frames.

Hellkamp was interested in a design for his unit that would remain as flexible as possible. Hellkamp's customers could decide the size of the unit (based on the number of frames ordered), select a particular style of bathroom and kitchen components that would attach to the unit's frame, and choose from a variety of exterior wall and storage components that would also be affixed to the frame. To make connections between the bays quicker, touch-pad electrical connectors were proposed between the frame and the floor, and between the frame and its horizontal supporting members.

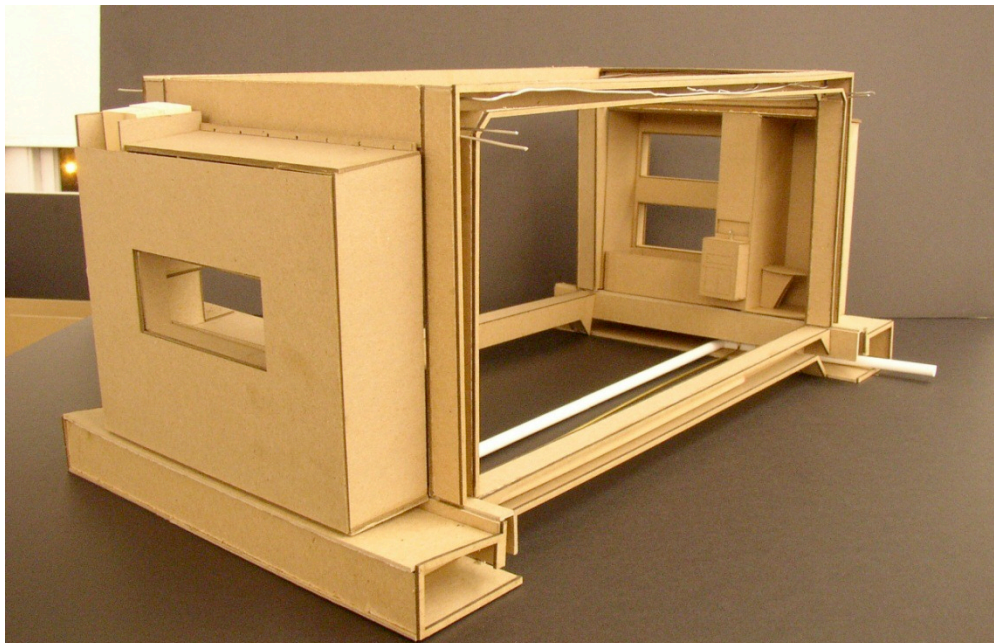


Figure 6 *Early study model of organizing the kitchen and bathroom on opposite sides of the unit. The project was developed so that the plumbing, electrical, and HVAC services would run through the building frame. (Luke Hellkamp, 2007)*

The success of the methodology of designing from the inside (out) is not limited to just the design of the building services within the unit's core. Through this methodology, students became adept at designing standard structural systems for the designs while other students discovered innovative structures and materials. In this studio, many students did investigate traditional framing in either wood or light-gage steel for their designs. Even when utilizing traditional framing for their unit, many students still understood that their structural decisions are not separate from architectural success, but can be integral to the architecture design concept. This approach is illustrated by Chae Carlson. Carlson designed a two-storey unit with the bathroom and kitchen cores stacked. *See Figure 7.* In order to simplify fabrication and to eliminate complex connections, Carlson proposed that the building plumbing and HVAC systems to be contained to one module, while the other module would meet the program's space requirements. She proposed that the modules would divide vertically (versus the traditional horizontal divisions) and because of this Carlson chose to frame her modules with balloon wood framing instead of the traditional platform framing. With the vertical studs being continuous for the full height of the module there was no need for a break in the load-bearing wall structure at each floor. This construction type allowed her building structure to conceptually support her vertical division of her modules and thus the architectural concept for her project.

A number of students researched alternative structural systems not typically associated with building construction. Returning to Blew's use of injection-molded plastic for the building core, because of injection molding fabrication size limitations it was unreasonable to construct his entire unit in the same method. Instead, Blew investigated using extruded plastic for his unit shell. *See Figure 8.* Extruded plastic created a stress-skin structural system, where the internal webbing of the structure would transfer the loads to the building skin. The material properties allowed for the structure to be water-tight with no additional assemblages required. Interior shelving would be integrated within the skin and insulating foam would be inserted into the interior voids to add additional thermal resistance to the unit skin.

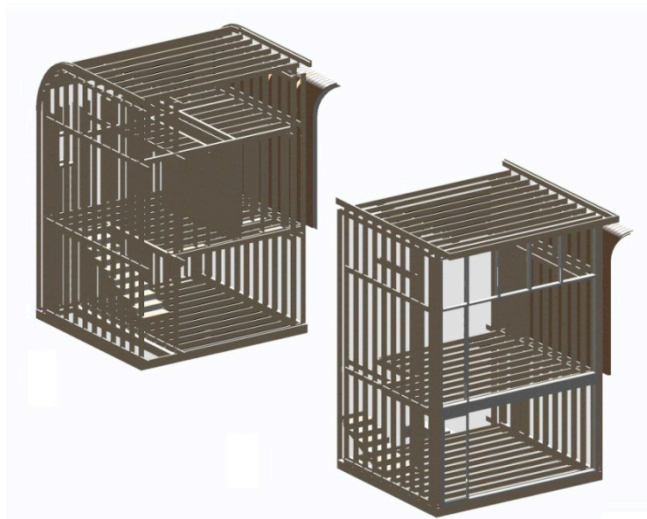


Figure 7 *The module on the left contains the building core and the one on the right fulfills the program requirements. (Chae Carlson, 2005)*

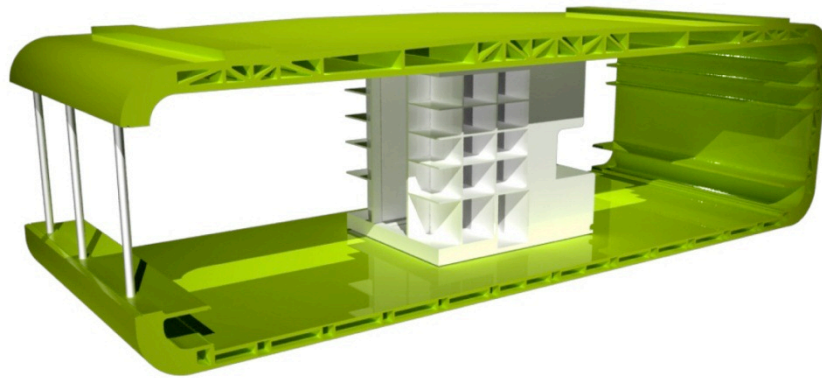


Figure 8 (*Blew 2007*)

C-shaped in its configuration, Blew proposed that the skin would remain flexible enough to use a hydraulic jack to spread the open end wide enough to slide in (from the open side) the building core. The core, located near the unit center, and additional columns, placed at the open mouth of the ‘C’ and shown in aluminum, would be used to support the roof. A plastic end cap would cover the exposed extruded edges along with a custom storefront assemblage for entry and glazing. It was intended that 3-4 units could then in turn be stacked upon one another.

Hellkamp also investigated extruded technology. For his unit, he proposed using structural moment frames, constructed from extruded aluminum. *See Figure 9.* The light-weight frames allowed for quick unit assemblage. First, bolts connected the frames to a pier foundation. Second, end caps were screwed to the frame to enclose the frame and its aforementioned systems. Cross beams would attach to the frame and structural insulated panels, SIPS, were added, creating the floor. All of the exterior walls were designed to offer shear strength to the frames for lateral strengthening. Finally the roofs would be attached to the frame. Hellkamp’s roof was fabricated from extruded vinyl with the structure and skin systems being the same material and assemblage. Although using a similar technology of extruded materials to Blew, Hellkamp designed a unit with a much more complicated assemblage process. Because of this, during his final presentation he presented the jury with an animated video demonstrating the process of building assemblage.

When I taught this studio in the Spring of 2006, many of my students were interested in disaster relief solutions for the victims of hurricane Katrina. One of the most successful solutions offered was a project by Katie Irons. Irons was interested in a collapsible unit (so that three units would fit on one truck) that was lightweight (requiring only a forklift to offload) and simply erected (so that only three able-bodied people were required). *See Figure 10.* In order to meet these criteria, Irons investigated fiberglass technology, similar to boat construction. Irons proposed a hollow fiberglass core and hollow building frames. The floor panels would be constructed of SIPS and would drop into the frames. The tops of the frames would be supported by aluminum rods that would simply slide into location. Lateral bracing and unit enclosure would be accomplished through the snap attachment of a breathable yet waterproof fabric (to be specified) to the sides of the frames. Because of the

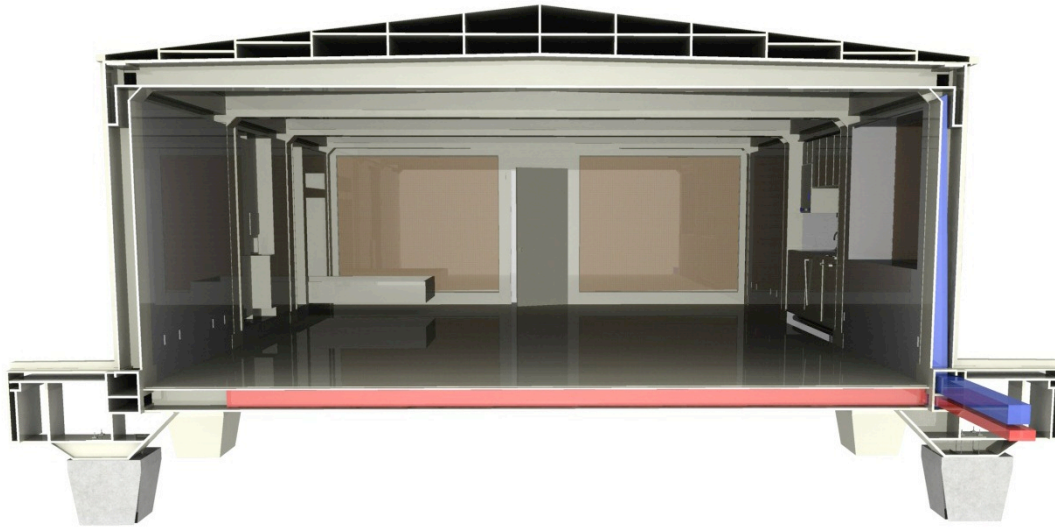


Figure 9 Building section/ perspective cut through unit frame. Note: red and blue bars indicate supply and return air services, respectively. (Hellkamp 2007)

lightweight and temporary function of these units, no foundation was required. To prevent against uplift or overturning, a reflective, waterproof cover would be thrown over the unit (similar to a tent) and connected to stakes that would be driven into the earth. This additional cover would provide a double envelop for the unit and would help keep the unit properly shaded. To reduce size during shipping, all materials required to erect the units could be stored in the unit core.

All of these structural innovations proposed through this studio relied on a qualitative, versus a quantitative, approach to understanding building structures. Calculations to support these innovative structures is both beyond the scope of the architectural studio as well as beyond the calculative abilities of most architectural students. As an architectural studio within an art and design college, we did not have access to the proposed fabrication technologies nor to testing equipment to physically test the proposals. Despite this lack of actual project realization, I do believe that the design and focus for the studio encouraged students to properly study their proposals and offer designs that had the potential for realization.

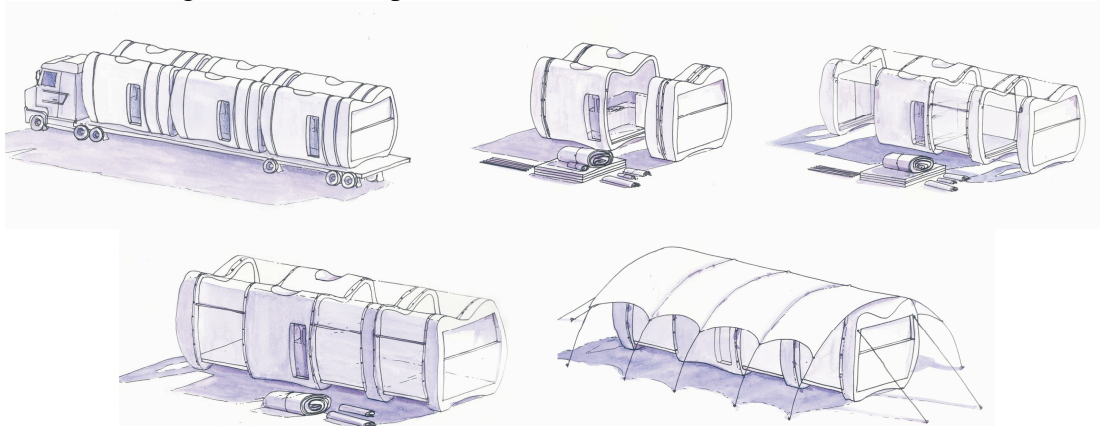


Figure 10 The proposed unit delivery sequence (Katie Irons, 2007)

In conclusion, I believe that we as designers we need to consider the reality of building services, fabrication and assemblage. These realities do not distract from designing, but can be used to enhance our building designs. The traditional method for building design has been establishing the macro criteria first and then later focusing on the design details of a building. This process is reinforced by both the profession and our education. I have argued to make buildings that are more technologically advanced, have a clear relationship between building systems, and are innovative in terms of building assemblages it is important that we reverse our traditional order of design. Although BIM offered a more integrated solution to design, our building services and systems are traditionally still designed in reaction to the architectural concept. This is not directly a fault of BIM, but is a fault of how we use the software. By designing a building from the inside (out), this comprehensive studio that I created at SCAD encouraged students to deliver innovative design solutions that addressed a simple architectural problem. By focusing on the building core and its services first, students offered design solutions that integrated services directly into the design concept, were more compact than traditional building cores, often utilized a combination of active and passive systems, and usually structurally supported the unit roof. After focusing on the core, students investigated structural systems for their building. Many students used traditional framing methods within their designs, while some detailed the framing to support their architectural concepts. A number of students discovered innovative structural systems and materials for the design of their units, including extruded aluminum and plastic, injection-molded plastic, and fiberglass. I believe that designing inside (out), as demonstrated by the student work, has the potential of generating projects that are more creative and more holistically designed than the traditionally designed counterparts.

Resources:

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KNOWLEDGE-BASED REPRESENTATION OF ARCHITECTURAL DESIGN PROCESS

Ajla Aksamija¹

ABSTRACT

The challenge in understanding immense aspects of architectural design process has been the central issue in an attempt to computationally define design process and design knowledge. Relationships between designs and the sources of the ideas used to generate them are extremely complicated to quantify, and to represent in the manner of explicit knowledge and data. Parameters that influence architectural design include a body of knowledge about the subject-matter, requirements of all involved parties, information about prevailing conditions on the site, and information about the traits and trends in the society served by the building. This paper discusses development of knowledge-based representations of architectural design process, particularly focusing on representations of design knowledge and factors in ontological framework. A case study for tall building design is used to demonstrate the representations. Web-based application is presented to exhibit the method for utilization in the schematic design, especially for decision-making process.

KEYWORDS: ontology, architectural design, information technology, knowledge management

INTRODUCTION

Ontology is derived from Greek words *ontos*, meaning to be, and *logos*, or word. In philosophy, ontology is the science or study of being. However, in computer programming ontology is a representation of a certain part of world in a machine-readable manner. Ontology is a knowledge-based model that represents a domain, and it is used to reason about the object in that domain, as well as the relations between them. It is a “formal, explicit specification of a shared conceptualization”, where conceptualization refers to an abstract model of a certain phenomenon by identifying the concepts of that phenomenon, explicit means that the concepts are explicitly defined, formal means that the ontology should be machine-readable, and shared refers to the fact that it should be a common, shared knowledge accepted by a group (Benjamin et al., 1998).

The reasons for using ontology for knowledge modeling are several, first being that they can be dynamically adapted according to the data, they can be shared, and they can be linked to larger models. Ontologies can be used for reasoning when there are certain uncertainties and with limited datasets. For example, the design of tall buildings is the most complex since the amount of information guiding the process is massive. Different components of tall building design include the response to the location of the project; decision on the building’s height and size depending on the client’s needs, codes and restrictions; selection of the structural system depending on the height, location and budget; selection of the building envelope depending on

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the function, usage, and environmental effects; and many other components and relationships between them.

The parameters that influence architectural design include a body of design knowledge, requirements of all involved parties, information about prevailing conditions on the site, information about the traits and trends in the society served by the building. The initial step in the design process is collection of information about these factors. The data collection is followed by a formal or informal organization process, categorization and analysis. Knowledge used in design takes different forms, and understanding of these parameters, as well as relationships among them, is crucial for any building design. The identification of design knowledge is the most basic and critical issue that must be addressed if design computation is to be achieved (Gero, 1990).

PAST RESEARCH AND FOCUS

Several earlier efforts have been developed for the purpose of conceptual modeling, classification systems and project management in architecture (Katranchukov et al., 2003). Substantial efforts have been undertaken in developing information models (Mitchell, 1990; Bjork, 1992; Eastman and Fereshetian, 1994; Gero and Fujii, 2000; Albars et al., 2004). In most of these conceptual models, the objective is to express expert conception of a certain part of the domain knowledge. The types of information included in the models vary depending on the purpose or the part of the domain. For example, Mitchell (1990) developed a method for expressing architectural style by linguistic computational representations. Eastman (1999) focused on the development of building product model, presenting physical and abstract representations of architectural design and buildings.

Classification systems, such as OmniClass and UniClass, provide taxonomies of building concepts and elements. The e-Cognos project is a comprehensive ontology-based portal for knowledge management in construction, developed by a set of web services supporting following activities: acquisition, transformation, indexing, updating, refreshing, searching, and sharing (Lima et al., 2005). The approach of e-Cognos classifies construction knowledge into three main groups: domain knowledge, such as administrative information, standards, technical rules, product databases; corporate knowledge or the intellectual capital of the company; and the project knowledge.

The purpose of the knowledge-based model presented in this paper is to capture design factors and process. The model contains representations of aspects that influence decision-making in the design, agents and their roles, and system considerations. Detailed structure of the ontology will be discussed in the subsequent sections. It has been used in a web-based application to demonstrate the use of ontology and the role of information modeling in conceptual stages of the design.

DESIGN KNOWLEDGE AND REPRESENTATION

Understanding the vast aspects of architectural design process has been the central issue in computationally defining design knowledge. Kalay (1990) states that “the complexity, ill-defined nature, and sheer volume of knowledge necessary for design have hampered its understanding and the development of universally

acceptable theories in design...The latest challenge to the enigma of design has come from attempts to explicitly define design knowledge and processes in a form that is computable.” The major problem is that the nature of architectural knowledge is implicit, meaning that it is acquired by education and practice and cannot be expressed numerically.

Initial stage of architectural design is the collection of information. The information that needs to be collected consists of set of spatial requirements or program, functional requirements, economical requirements, and information about the conditions on the site, such as physical, environmental, social, and cultural aspects. The design factors depend on the site, such as topography, orientation, and climate, budget, users and their needs, aesthetics, structural system, materials, and many other interrelated aspects. The designer processes this information, analyses, and searches for a solution that satisfies the requirements.

Ontologies capture the meaning of knowledge and process it, as well as contain data about specific instances. The considerable advantage is that not only physical properties, such as dimensions, boundaries, and geometrical characteristics can be represented, but also conceptual constraints, such as environmental factors, cultural and social aspects. The case-based reasoning is also an important aspect of the process, since the library examples and space use provide information about the specific buildings. The general concepts are represented through classes, which are the basic components of ontology. Classes are represented as nouns, such as “Agent”, “Building”, “Design_Factors”, etc. The taxonomical structure of classes provides a hierarchy of conceptual terms, and classes inherit characteristics from their parent-classes through this structure, as seen in Figure 1a. The subclasses inherit all the properties from the parent class, but they can also have additional properties and restrictions that apply only to that specific concept. Figure 1b graphically represents the overall structure of the ontology discussed in this paper. The ontology uses Open Web Language (OWL), specified by the World Wide Web Consortium, for logical descriptions, with an example shown in Figure 2. The multiple inheritance is allowed in OWL language; therefore a class can belong to several superclasses.

ONTOLOGY DESIGN AND ORGANIZATION

An ontology design starts with introduction of competency questions in order to define the scope, as well as the clear definition of the purpose and domain. Since the discussed ontology is used to represent design factors and process, the main classes contain general aspects, such as building, agents, and definitions of process. The properties are assigned to concepts, and are used to define the relationships among different classes. The classes include environmental, social, cultural and physical properties—the rest of the ontology follows a hierarchical model describing the main concepts. The properties are of two types—object and datatype. Object properties describe the class, and can be used to assign relationships between different classes. The object properties can be functional, inverse functional, symmetric and transitive. Functional properties are single valued, meaning that they can have only one value for an instance of a certain class.

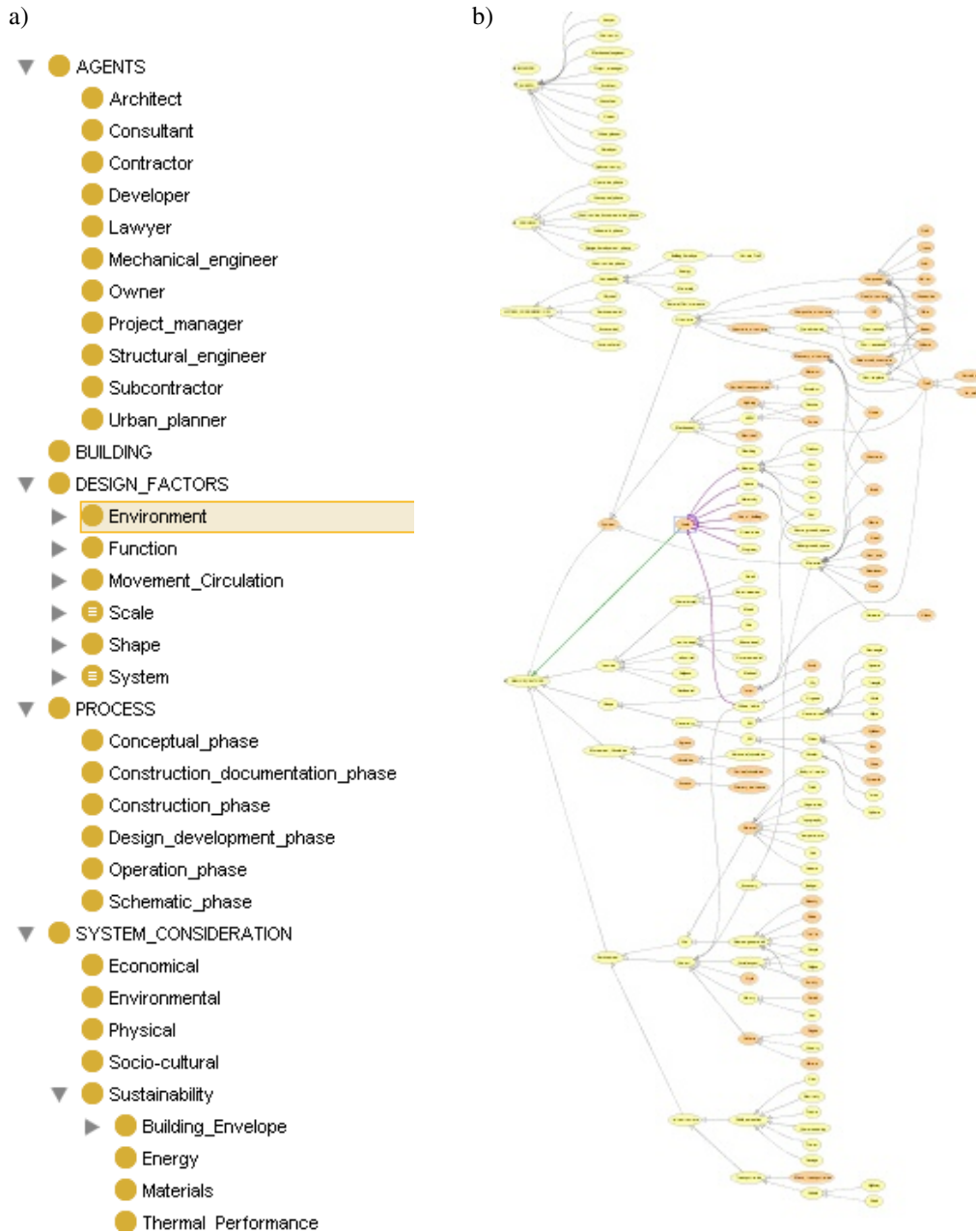


Figure 1: a) Major classes in the ontology, b) Ontology structure.

Transitive and symmetric properties describe series of classes and the relationships among them. Datatype properties link a class to a datatype, such as an integer, string, boolean, date, etc. Datatype properties can only be functional and inverse functional. The types of properties that are used in this ontology include both object and datatype properties, and they describe physical, economical, social, spatial, geometrical and other characteristics. For example, properties of a structural system are described using the geometry and size of elements, such as cross-sectional area of

a column. Users and activities are described using the type of persons, taking into account age, personality and social activities.

```

<owl:ObjectProperty rdf:ID="has_Agent">
  <rdfs:domain>
    <owl:Class>
      <owl:unionOf rdf:parseType="Collection">
        <owl:Class rdf:about="#BUILDING"/>
        <owl:Class rdf:about="#PROCESS"/>
      </owl:unionOf>
    </owl:Class>
  </rdfs:domain>
  <rdfs:range rdf:resource="#AGENTS"/>
  <owl:inverseOf rdf:resource="#is_Agent_of"/>
  <rdfs:comment rdf:datatype="xsd:string"
    >This property defines agents for a building.</rdfs:comment>
</owl:ObjectProperty>
<owl:Class rdf:ID="Activity">
  <owl:equivalentClass>
    <owl:Class>
      <owl:intersectionOf rdf:parseType="Collection">
        <owl:Restriction>
          <owl:onProperty rdf:resource="#depends_On"/>
          <owl:someValuesFrom rdf:resource="#Culture"/>
        </owl:Restriction>
        <owl:Restriction>
          <owl:onProperty rdf:resource="#depends_On"/>
          <owl:someValuesFrom rdf:resource="#People"/>
        </owl:Restriction>
      </owl:intersectionOf>
    </owl:Class>
  </owl:equivalentClass>
  <rdfs:subClassOf rdf:resource="#Social_aspect"/>
  <rdfs:subClassOf rdf:resource="#Human-generated"/>
  <rdfs:subClassOf>
    <owl:Restriction>
      <owl:onProperty rdf:resource="#depends_On"/>
      <owl:someValuesFrom rdf:resource="#Function"/>
    </owl:Restriction>
  </rdfs:subClassOf>
  <rdfs:comment rdf:datatype="xsd:string"
    >This class defines the activities in a building.</rdfs:comment>
</owl:Class>
  
```

Figure 3: OWL logic description language.

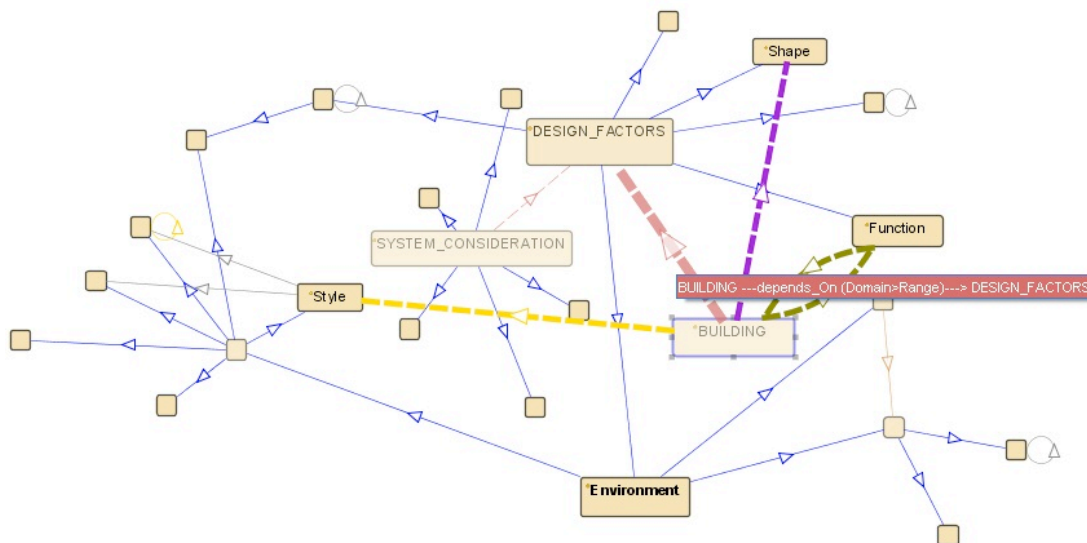


Figure 4: Restriction relating classes “Building” and “Design_Factors”.

Existential restrictions are used to describe a class or individuals of a class that have at least one kind of relationship to another class. Universal restrictions describe the classes that only have relationships to other classes or individuals of a

material databases, collaboration, precedent studies and examination of design factors through ontology. It has been developed as an educational tool to assist in teaching integrated design.

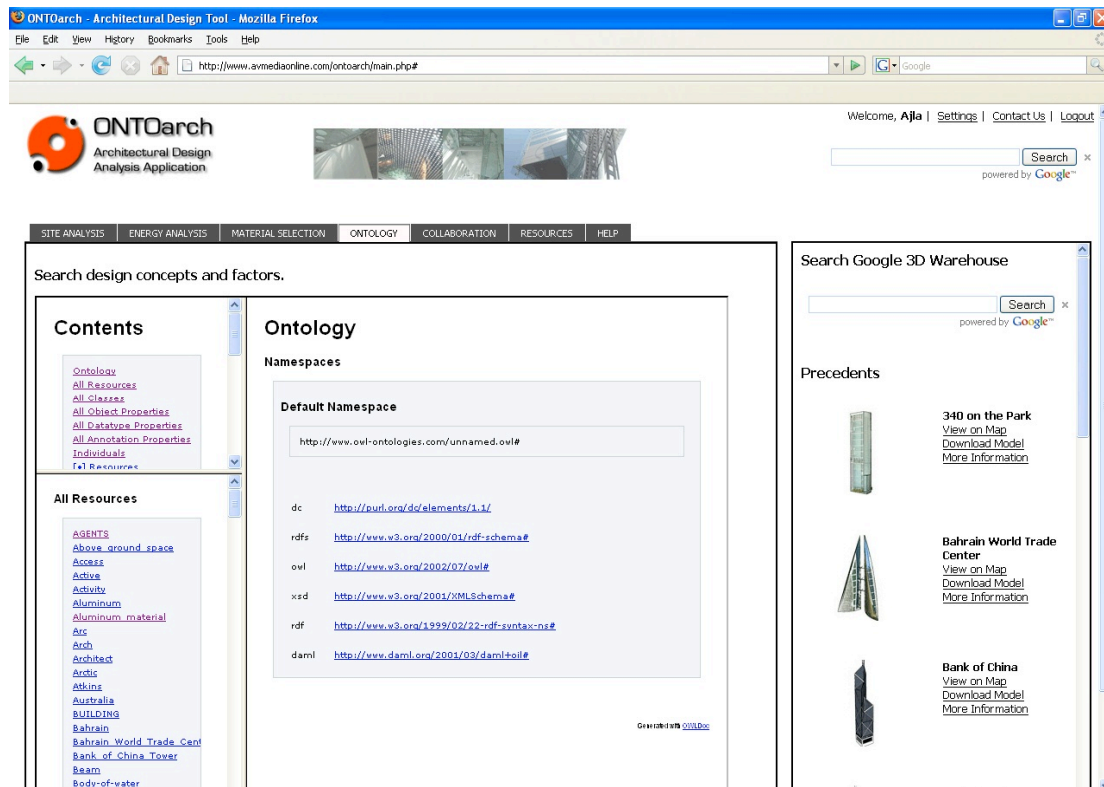


Figure 6: Contents and navigation through ontology.

Schwabe and Rossi (1995) suggest that in designing hypermedia applications using information models separation of content design, navigational design and abstract interface design should be achieved in order to allow seamless evolution from abstract domain models to concrete implementation of hypermedia applications. Design approach for ONTOarch followed similar methodology, where the knowledge-based model was integrated within the application; however, it was developed separately and independent of the application.

Ontology in this application represents architectural design factors in a semantic model. It is also linked to the precedents, displaying information for existing buildings. The frames allow for navigation of classes, individuals and properties, presenting the context and relationships. It can be searched for classes, individual, datatype properties and object properties. The upper left frame displays the contents, and available selections, as shown in Figure 6. For example, once class “Agents” is selected, the usage, syntax and descriptions are displayed. Information about particular buildings can be searched through the ontology by selecting “BUILDING”.

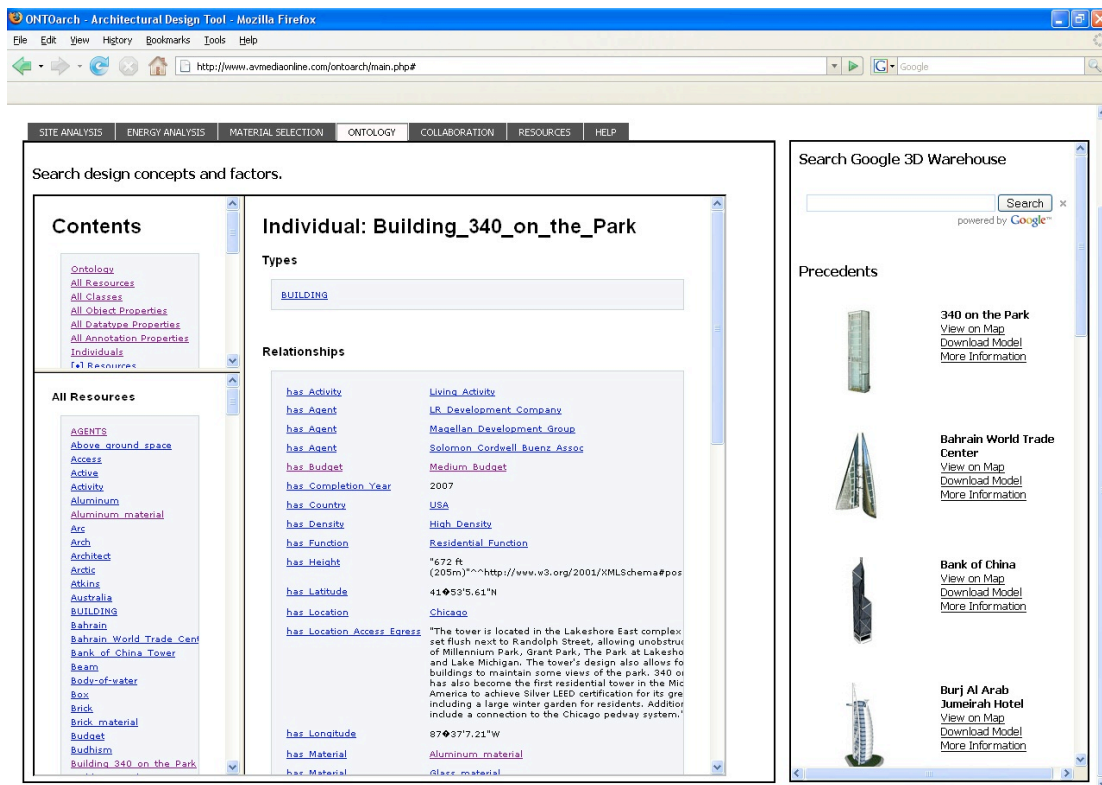


Figure 7: Information about a specific building displayed through ontology.

Components of site analysis include a map with geographical and satellite information, as well as demographic information and census map. Geographical map requires an address or location for input, and is able to search worldwide locations. Census map requires an address and zip code for US Census Data, and searched for population and housing information according to a 1 mile, 3 mile or 5 mile radius. Energy analysis components are external applications and require input such as location, functional type, and types of components. Materials selection consists of materials databases that can be searched for products, manufacturers, CAD details, and 3D models. Forum and real-time discussion are methods to collaborate. The purpose is to allow users to simultaneously share knowledge and ideas while searching for information.

Precedents include existing buildings that can be viewed on the map, and whose 3D models can be downloaded. Information about these buildings is stored through ontology, and can be retrieved, as seen in Figure 7. Search for other buildings is available through general web search, and specified search to retrieve three-dimensional models.

CONCLUSION

The challenge in understanding the breadth of architectural design has been the central issue in an attempt to computationally define design process and design knowledge. The knowledge used by architects and engineers in building design takes many complex forms, and this complexity accounts for the difficulty in computing

design. The focus of this research has been computational representations of design factors, particularly capturing design knowledge to be used by software applications. A knowledge-based model, or ontology, has been developed to capture design factors, such as environmental, social, physical, and contextual factors, as well as agents, design process, and considerations in the design.

Web-based application for decision-making process in architectural design is presented, focusing on early-phase schematic design. The application integrates several functions necessary for schematic design: site analysis, energy analysis, material selection, collaboration, and exploration. The benefit of this approach is that this system actively assists designer in the decision-making process.

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