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Forensic Engineering: Informing the Future with Lessons from the Past

Proceedings of the Fifth International Conference on Forensic Engineering organised by the Institution of Civil Engineers and held in London, UK, 16–17 April 2013

Edited by John Carpenter CEng, FICE, CFIOSH
Organising Committee Chairman

Published by ICE Publishing, One Great George Street, Westminster, London SW1P 3AA.

Full details of ICE Publishing sales representatives and distributors can be found at
http://www.icevirtuallibrary.com/info/printbooksales

www.icevirtuallibrary.com

A catalogue record for this book is available from the British Library

ISBN 978-0-7277-5822-4

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Printed and bound by CPI Group (UK) Ltd, Croydon, CR0 4YY



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Preface

In April 2013 I was able to welcome some 150 delegates to the 5th International gathering of those with an interest in forensic engineering. That this was the fifth ICE conference is testament to the world-wide interest and recognition of this important subject area.

The last conference was held in 2008. Much has occurred since that time, but in the main the events of the period have reinforced the basic tenets of forensic engineering; the need to:

- Learn from the past
- Facilitate and exchange knowledge
- Input into codes and guidance such that they reflect contemporary knowledge and understanding
- Encourage education and career learning in this field
- Improve investigative and monitoring techniques
- Support the work of expert witnesses

for all aspects of the built environment.

Sincere thanks are due to the conference sponsors Ramboll, Thornton Thomasetti, Sandberg, RSK and Fugro which provided valuable support.

The plenary presentations prefixed each of the two days with wide-ranging and thought provoking examples of forensic engineering: Rob Ratay led off as the key note speaker with an overview (**Rewards of failure – changes in codes, standards and practices resulting from failures),** followed by **How failures initiate progress in engineering design and construction practice** given by Jonathan Wood, and finally, on the first day, **Learning from failure – the challenges and opportunities** by Sean Brady. This was a broad spectrum from an international perspective.

On the second day we commenced with the key note address by Wei Lee on **Forensic investigation and performance analysis for bridges under extreme natural hazards.** This was followed by **Investigation into highway bridge damage and failures during the November 2009 Cumbria flood event,** by John Collins and finally **New York City 51st Street Tower Crane Collapse Investigation** by James Cohen. Another truly international perspective.

These papers indicated the importance of forensic work and also the common lessons that apply, regardless of the detailed circumstances. This conference was the first I believe to include a ‘temporary structure’ example in its opening session. These are perhaps a neglected grouping given their importance and safety-critical nature and we devoted a session to them in the conference.

Critical and contemporary aspects to forensic engineering were reflected in the topics chosen for the conference sessions themselves, viz:

- Education and continuing professional development in Forensic Engineering
- Legal, contract and learning

These represent the foundations of any discipline; without the underpinning knowledge, enhanced by an understanding of contemporary practice, and without an understanding of the operational framework within which we have to operate (and which is often a causal factor in failures), the industry will not improve and our infrastructure will suffer.

- Case Studies - Bridges
- Case Studies - Ground and below Ground Works
- Building Failures and Investigation

The detailed examination and analysis of past failures is essential to progression. Hence it was no surprise to see an emphasis on this topic in the conference. However, it is from these examples, and others, that we often see the recurrence of known basic shortcomings and are able to develop new research, guidelines and codes of practice

- Materials Failures and Investigation

Past experience tells us that we need to be cautious about the potential for material failure e.g. concrete (AAR), steel (LMAC),

- Temporary Structures

As mentioned, for the first time we had a session on temporary structures, including one which although classified as ‘art work’ was in fact a very specialised lightweight structure, and another which is a tower crane.Temporary structures often have characteristics which differentiate them from permanent structures, however they demand as much attention and care as their permanent counterparts.

You will find this publication a useful reference source for both research and contemporary design. We forget the lessons of the past at our peril.

John Carpenter CEng, FICE, CFIOSH

Organising Committee Chairman

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SECTION 1:
OPENING PLENARY PAPERS

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ISBN 978-0-7277-5822-4

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doi: 10.1680/feng.58224.003

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Rewards of Failure – *Changes in codes, standards and practices resulting from structural failures*

Robert T. Ratay, PhD, PE, Consulting Structural Engineer and Visiting Professor, Columbia University, New York, NY, USA

Synopsis of the keynote presentation delivered at the
ICE 5th International Conference on Forensic Engineering, London, April 15-17, 2013.

Following the spectacular twisting, squeaking and collapse of the Tacoma Narrows Bridge in 1940, the Governor of Washington State proudly declared '*and we will rebuild the bridge exactly as it was before*', to which the Hungarian-American scientist John von Neumann declared '*and it will fall down exactly as it did before*'. Aside from its amusement, the story, whether true or not, underlines the need for pragmatic changes in codes, standards and practices after failures.

Failures are bad, often with serious and sometimes with catastrophic consequences but, if followed up pragmatically with deliberate constructive effort, they can and often do result in "rewards" to both the profession and society.

The presentation calls attention to **one form of "rewards"** that derives from structural and construction failures: **the improvement of codes, standards and practices**. The presentation is, in essence, a review of an ongoing survey of changes in structural design and construction codes, regulations, approval, oversight and other industry practices, resulting from catastrophic structural and construction failures in the US. Some rather high-profile as well as lesser known cases of failures of bridges, buildings, temporary works, construction cranes, liquid storage tanks, aircraft, and other structures are cited as examples to illustrate the types of changes that have resulted from disasters.

The author's intent is two-fold: (i) to bring awareness to the fact that welcome changes in many areas of the design-construction industry have and do come about with benefits realized as the result of catastrophic failures, and (ii) to urge that we, engineering professionals, press for change when warranted, and extract all possible benefits from failures. All of this is for the ultimate purpose of mitigating future failures!

Structural failures are the result of human activities which – in the design-construction industry – are prescribed in part by codes, standards, regulations and industry practices. If it is found through the investigation of a failure that adherence to the governing documents and practices allowed or, indeed, created the cause of the failure, then it makes "good sense" to critically review those codes, standards, regulations and industry practices and, if warranted, to start a process to revise them. Sometimes, but unfortunately not always, the "good sense" is followed. In the US the effort may be initiated and carried out by city, state or federal agencies or by professional engineering or trade organizations, with the assistance and talents

of professional societies, trade organizations, volunteers from private engineering firms, construction companies, and academia. Sometimes, tightening of requirements start at the urging of insurance carriers. Changes to existing, and even the creation of new, specifications can and at time do, originate from individuals. (Leon & Rossberg 2012; Duntemann 2010). This relative ease in the US is in contrast with the more rigid bureaucratic practices in many other countries.

As expressed at the beginning of the two consecutive references of Ratay 2010 and 2011,

"*To the credit of our profession, failures have been and continue to be used to improve design and construction practices. We do not just pay up, rebuild and walk away – we delve, we learn, and we improve.*

Following a failure, engineers often carry the forensic investigations to great details, so as to have reasonable engineering certainty not only in the cause(s) of the failures but also in the identification of the responsible parties – needed for the frequently inevitable dispute resolution. A valuable peripheral benefit of the laborious search is a clearer understanding of structural behavior and a better appreciation of pitfalls in the current practices. These can provide information and material to affect eventual changes in design and/or construction practices, codes, standards, oversight and regulatory procedures.

The 'lessons learned' from failures are interesting but worthless, if not heeded and not acted upon to prevent their re-occurrence."

The objective of the presentation is not so much to show the audience actual failures and specific resulting changes, as it is to make them aware of the practice and the process of bringing about the changes. While the end results are generally the same, the process can start with a failure (or several failures) pointing ahead to a needed change, or with an accomplished change pointing back to a failure (or to several failures).

Actual failures and highlights of specific follow-up actions are cited in the presentation as examples of changes in structural design codes, standards, specifications, construction safety regulations, approval, oversight, and other industry practices, that followed (and some that should or should not have followed) catastrophic failures.

The author has compiled cases not only from the US but from several other countries as well. The latter are not addressed at this time; some of them will be mentioned by speakers from other countries at this conference.

The presentation begins with Xerxes I of Persia, whose first response to the collapse of his storm-shattered bridge at Hellensport (now known as the Dardanelles) in 480BC was to whip the sea 300 times and for good measure to behead the engineers. A new batch of engineers studied the failure, changed their methods and rebuilt the bridge in an improved configuration – and both they and the new bridge lived happily after.

The list, at the time of this writing, ends with the flipping and breaking of the boom of a tower crane 96 floors above the street during the October 30, 2012 hurricane Sandy in New York City that together with a rash of recent crane accidents is certain to bring about tighter rules. The examples in between include, as time allows, a number of the events listed below (some of which were reported in Ratay 2010 and 2011, as well as some others that are almost certain to occur in the United States between the time of this writing (December 2012) and the time of the oral presentation (April 2013).

The oral presentation includes images of all of the cases shown, and pertinent details of some of them.

1. One of the best examples of continuous code improvement is the development of fire safety codes in response to past disasters and for mitigation of future ones. *Historical fires* such as the 1860 tenement fire in New York City, the Great Fire of Portland (1866), the Great Chicago Fire (1871), the Great Toronto Fire (1904), the 1906 San Francisco Earthquake and subsequent fire focused attention on the vulnerability of cities to the threat of distraction by fire. Building departments have been structured and restructured to meet each era's challenges, and fire and structural safety codes evolved following the lessons learned from earlier catastrophes.
2. Introduction of the 1979 US National Bridge Inspection Standards, in accordance with the provisions of the American Association of State Highway and Transportation Officials (AASHTO) "Manual for Maintenance Inspection of Bridges" at regular intervals not to exceed two years. This followed the 1967 collapse of the *Silver Bridge* in West Virginia, where forty-six persons died, nine were injured, and thirty-one vehicles fell with the bridge.
3. Rules, regulations, local laws (e.q., Local Law 11 in New York City), façade ordinances in a number of cities – including Boston, Chicago, Columbus, Detroit, Milwaukee, Philadelphia, Pittsburg, New York, St. Louis – to assure public safety against crumbling facades, falling appendages and other hazards by requiring and strongly enforcing periodic inspection, maintenance and repair of building facades. The façade ordinances are mostly in response to *incidents of falling materials from buildings* onto sidewalks that had caused damage, disruption of traffic and serious injury. In particular, some of the much-publicized incidents that triggered the responses included those of the 1974 falling of a façade tile and the 1999 falling of a piece of wind-propelled glass in *Chicago* that struck and killed two people; the 1979 falling of a stone appendage that struck and killed a college student; and the 1997 dangerous bulging of the brick veneer on a wall high above Madison Avenue in *New York*.
4. New laws in several states on the matter of delegation of design responsibility by licensed professional engineers to contractors, influenced by the catastrophic failure of a suspended walkway in the *Hyatt Regency Hotel in Kansas City*, Missouri, where 114 people died (1981). The widespread conclusion of the nation-wide debate was that while the engineer may properly delegate the work of performing engineering design functions, he/she cannot delegate his/her responsibility for the structural design where it concerns professional engineering functions.
5. The 1986 publication by the US Federal Highway Administration (FHWA) of a manual of inspection of fracture-critical bridge members, following the 1983 fracture failure and catastrophic collapse of the *Mianus River Bridge* in Connecticut.
6. Stricter design and construction requirements in some states on lift-slab construction, following the collapse during construction of the *L'Ambiance Plaza* 16-story residential building in Bridgeport, Connecticut, in which 28 workers were killed (1987). Some aspects of the catastrophe also entered the debate on the matter of delegation of design responsibility
7. Activities to require peer reviews of structures beyond a certain size, following the catastrophic collapses of the space-frame roof of the *Hartford Civic Center* in Hartford, Connecticut (1987), and other long-span roof collapses at the *C. W. Post Center*

Auditorium of Long Island University in Greenvale, New York (1978), the *Kemper Memorial Arena* in Kansas City, Missouri (1979), and the *Rosemont Horizon Arena* in Rosemont, Illinois (1980).

8. Development of the American Petroleum Institute (API) Standard 653, Tank Inspection, Repair, Alteration, and Reconstruction, first published in January of 1991, precipitated by the *Ashland Oil Tank* failure tank failure in 1988 at Floreffe, Pennsylvania, due to a crack emerging from a locally embrittled area that could not be arrested, spilling some one million gallons of diesel fuel into the Monongahela River affecting the local drinking water supply.
9. ***Executive Order 12699 - Seismic Safety of Federal and Federally Assisted or Regulated New Building Construction,*** dated January 5, 1990, was issued by the President of the United States with design requirements for earthquake safety of new federal buildings. The Order followed the October 17, 1989, *Loma Prieta*, California earthquake.
10. An example of a code change that may or may not have been warranted is the *North Georgia snow storm* on March 13, 1993, which resulted in some large roof collapses. Measured snow samples weighed approximately 22 psf where the governing code called for 5 psf design ground snow. This was not the 50-year MRI nor the 100 nor the 500. It was almost spring; it was a freak of nature. There were a small number of people fervently urging the ASCE 7 Standard Committee to significantly increase the design snow loads. The Snow Load Subcommittee declined on the bases that the increase was not scientifically warranted and we would be overdesigning buildings and wasting resources.
11. Stricter and additional requirements to improve the progressive collapse resistance of cast-in-place reinforced concrete structures by providing continuity of reinforcement, first in the *ACI 318-02* and later in the *ACI 318-08.* to "improve the ductility and redundancy of structures" These actions were precipitated by the 1995 bombing of the *Murrah Federal Office Building* in Oklahoma City which resulted in the collapse of a large part of the building.
12. Establishment and implementation of the concept and the rule of the Limit of Validity (LOV) to prevent widespread fatigue damage (WFD) to ageing aircraft in compliance with Title 14, Code of Federal Regulations (14 CFR) 26.21. Owners/operators of aircraft must eestablish a limit of validity of the engineering data that supports the structural maintenance program that corresponds to the period of time, stated as a number of total accumulated flight cycles or flight hours or both, during which it is demonstrated that widespread fatigue damage will not occur in the airplane. The FAA rule was the result of the widespread fatigue damage in *Aloha Flight 243* in 1988 that caused separation of a large section of the fuselage from the aircraft in mid-flight killing a flight attendant and injuring many others.
13. The "Steel Erection Final Rule" issued in 2001 by the US Department of Labor, Office of Safety and Health Administration (OSHA), mandating the use of four, rather than two, anchor bolts in structural steel column base plates. This rule was "negotiated" as a result of numerous construction accidents caused by the *toppling of unbraced steel columns during erection*.
14. Modifications of the construction approval and oversight procedures in the New York State DOT and in the AASHTO bridge design specifications, following the 2002 collapse during construction of the *Marcy Pedestrian Bridge* in Utica, New York, in which one worker was trapped in a pile of fresh concrete and suffocated.

15. Publication of the ASCE/SEI standard ASCE 37-02 Design Loads for Structures During Construction for specifying design loads on temporary structures that provide support and access in the construction process and for permanent structures during their construction phases. The standard was developed in response to the findings that many *construction failures* are the result of improper design considerations.
16. New and stricter requirements in the New York City Building Code, for demolition documents signed and sealed by a registered design professional submitted for the Building Department's review and approval. This followed the complete or partial collapses of several multi-story buildings during demolition work New York City and finally in 2005 the complete *collapse during demolition* of a one-story building at 2633 Broadway.
17. Nineteen safety recommendations of the National Transportation Safety Board (NTSB), one of which was specifically addressed to the American Concrete Institute (ACI) "to inform design and construction agencies of the potential for gradual deformation (creep) in anchor adhesives and to make them aware of the possible risks associated with using adhesive anchors in concrete under sustained tensile-load applications", and ACI's response by issuing Appendix D addressing adhesive anchors to ACI 318-11, *Building Code Requirements for Structural Concrete*, which must be qualified per ACI 355.4, *Acceptance Criteria for Qualification of Post-Installed Adhesive Anchors in Concrete and Commentary*. These and other developments were the result of the 2006 collapse a section of the precast concrete hung ceiling in the *Central Artery Tunnel (CA/T) in Boston*, which crushed a car and killed a woman motorist.
18. The National Transportation Safety Board (NTSB) made recommendations to the Federal Highway Administration (FHWA) and the American Association of State Highway and Transportation Officials (AASHTO) regarding, among other things, safety inspection procedures, following the 2007 catastrophic collapse, of the eight-lane 1,900-foot long *I-35W highway bridge* over the Mississippi River in Minneapolis, Minnesota, where thirteen people died and one-hundred forty five people were injured. This incident is producing pending changes in, among other things, gusset plate design.
19. Two crane-safety bills were introduced in the Washington State Legislature in: Senate Bill 5990, and House Bill 2171, and signed into law in 2007, making the state's new crane-safety regulations into some of the nation's strictest. Effective January 1, 2010, the Crane Safety Act requires, among other things, cranes to be load-tested, inspected, and certified at least annually, after any significant modification or repair of structural parts, and before and after each setup at a new site. Crane owners are required to have an independent professional engineer review and approve plans for any non-standard tower crane base. These laws came about as a result of the 2006 *collapse of a 210-foot tower crane*, used in building construction, in Bellevue, Washington, damaging three buildings and killing a man in his top-floor apartment.
20. New requirements in Florida's Building Construction Standard, for "threshold inspection" of certain construction activities in order to ensure structural component conformance of large structures, following a number of collapses, ending with that of the multi-level reinforced concrete parking structure at *Berkman Plaza* in Jacksonville, Florida (2007).
21. Section R322.2.2 of the 2012 International Residential Code requires flood openings in foundation walls and walls of enclosures below the base flood elevation to allow waters to enter the enclosed space during floods. This is in response to cracking, warping and *collapse of residential foundation walls* due to un-equalized hydrostatic loads observed during floods throughout the US.

22. The practice of installing hurricane tie-downs and stricter construction inspections in southern states of the US. The need was realized from the rash of *damage to wood-frame houses* in hurricane-force winds, most emphatically by the 1992 Hurricane Andrew that hit Florida as a Category 5, and proceeded across the Gulf of Mexico to Louisiana and Mississippi. Design and construction criteria were developed from wind tunnel testing.
23. Introduction of Chapter I - Fabrication & Erection, Chapter J - Quality Control & Assessment, and Chapter K - Prequalifications & Cyclic Quality Testing Provisions in ANSI/AISC 341-10 Seismic Provisions for Structural Steel Buildings, and publication of ANSI/AISC A358-10 Prequalified Connections for Seismic Application, all resulting from observations made in the damaged structures after the 1994 *Northridge*, California earthquake, as well as after the 1995 Kobe, Japan *earthquake*. The observations were notable because we had not seen such failures of welded structural steel moment-frames before.
24. As one of its first contributions to anti-terrorist mitigation activities, the American Society of Civil Engineers formed a standard committee which assembled existing design criteria and in 2011 published ASCE 59-11 Blast Protection of Buildings to provide design professionals with a better understanding of blast-resistant design principles and a definitive consensus standard for the design of buildings for blast resistance. The activity was in *response to terrorist events* over the past two decades that involved a wide range of explosive devices, including the Marine Barracks in Lebanon in 1983, the Murrah Building in Oklahoma City in 1995, the Air Force housing at Khobar Towers in Saudi Arabia in 1996, and others.
25. An event and its catastrophic consequences that are certain to precipitate both short-term and long-term changes in codes, standards, procedures, zoning laws, public and political attitudes and whole new approaches to mitigating the destructive effects of wind storms and storm surge waves is the very recent, October 30, 2012, *hurricane Sandy* and its aftermath especially in New York City, Long Island, and the coastline of New Jersey.

The above list is only a few of the dozens of documented cases of actual, pending and desirable changes in codes, standards and practices compiled by the author.

The intended "message" of the presentation is that we forensic professionals are the ones who through our investigations acquire the deepest understanding of the origins, causes, mechanics and consequences of failures, and therefore we should champion the effort for productive changes when warranted for the ultimate purpose of mitigating future failures!

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Forensic Engineering: Informing the Future with Lessons from the Past
ISBN 978-0-7277-5822-4

ICE Publishing: All rights reserved
doi: 10.1680/feng.58224.009

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Evolution of design and construction, guided by failures.

Jonathan G M Wood BSc PhD FIStructE MICE CEng.
Structural Studies & Design Ltd, Northbridge House, Chiddingfold, Surrey GU8 4UU UK, jonathan@ss-design.demon.co.uk

Abstract

Significant developments in engineering design and construction are usually triggered by failures which demonstrate the limitations of previous practice and initiate research and innovation. It is important for each generation of engineers to understand 'the story so far' if they are to have an appreciation of how innovation can be successfully introduced and if they are to avoid the repetition of mistakes of previous generations.

Teaching the principles of engineering design can beneficially be based on showing how the stepwise evolution of design has been interrupted and redirected by failures. Collapses are freaks. Functional failures and unnecessary remedial works or demolition after 30 years are not sustainable and have greater economic and social consequences.

Forensic Engineering should not be considered a separate speciality or taught as a distinct topic. Throughout an engineer's career case studies should be integrated into all aspects of teaching, CPD and in house training.

Prior to embarking on any design or construction task engineers at all levels should familiarise themselves with incidents and deficiencies of fitness for purpose in recent and historic precursors of their project. This should also be obligatory for standards committees.

Introduction

For an inexperienced graduate the reality of design and construction is a daunting prospect. Often his skills have been focussed into a narrow speciality. He is provided with software, whose hidden processes are a mystery, and codes of practice which dictate without explanation. How can a young engineer aspire to create structures which are 'Fit for Purpose, Delight the Eye and Cost Effective', without understanding the evolution of current good practice from centuries of successes and failures?

This evolution of bridge and building design has followed, somewhat sluggishly, the Forensic Cycle, Figure 1, by which information from failures has progressively found its way into design guidance, construction best practice and health and safety criteria. Research to aid design has often been funded only after failures.

From this process the common forms of concrete and steel construction have been evolved by the remorseless Darwinian selection with the survival of those fit for purpose, cost effective and with visual appeal. Innovations in form, materials and processes have often led to failures, the proper investigations of which have then enabled successful forms to evolve.

This should be the central theme of all teaching of design and construction practice at Universities [1] and in continuing professional development. Contract and construction procedures are often the root cause of both structural failures and health and safety incidents. So lessons from failures need to be stressed in construction management as well as design courses.

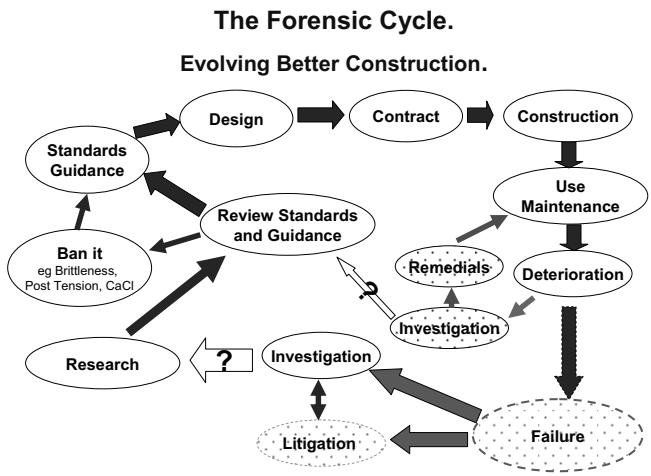


Figure 1. The Forensic Cycle: information flow for better construction.

There is no novelty in advocating this approach. Reading the historic proceedings of the ICE, now fully accessible ICE Virtual Library [2], shows that the analysis of failures and discussion of improvements in practice has been a major feature of ICE meetings for approaching 200 years. However the Forensic Cycle can only function properly when full accurate reports of failures are published [3].

How did our design standards evolve?

The ICE was founded of the 19th century at the start of the transition from timber and masonry construction to iron, steel and concrete. Most of our current road, canal and rail infrastructure and buildings were designed and built using Victorian or earlier methods. We need to understand these methods so we can continue to adapt and maintain these assets, as has been done in the refurbishment of Robert Stephenson's 1849 High Level Bridge, Newcastle-upon-Tyne [4].

The 19th C started as an era of practical empiricism and trial and error and progressively incorporated lessons from failures with testing and analysis. Thomas Telford, a master mason and George Stephenson, a master mechanic, were followed by I K Brunel and Robert Stephenson who, with better education, applied mathematics, physics and a growing understanding of material properties to build increasingly bold and complex structures without a code to guide them. Annually updated compendiums [5] of formulae of physics and chemistry and examples of their practical application covering marine works, steel production, electricity, steam, surveying and bridge design inter alia, enabled Victorian and 20thC engineers with a slide rule and common sense to design and build successfully all round the world.

Major innovations, like Telford's 179m span Menai chain suspension bridge opened in 1826, were not without problems, but these were analysed and resolved [6] and the lessons applied to subsequent designs like Clifton Bridge. This led to an understanding of the importance of balancing stiffness, damping and articulation in large bridges, which lessons have had to be relearned periodically.

Thomas Bouch made his name building for rail companies who pressed for cheaper, faster construction and wanted longer bridges to fill gaps in the network. This design approach led to the collapse of his Tay Rail Bridge in December 1879 as a train was crossing in a strong gale, 75 people died. The enquiry [7] concluded the inadequate design of wind bracing for only 10lb/ft (0.48 kN/m^2) wind pressure was the primary cause.

A range of other deficiencies in construction became apparent including the weakness and poor tolerancing of brittle cast iron lugs on the columns to which the wrought iron bracing was connected. Tay Bridge was rebuilt to a more robust design and remains in use, but with reduced loading. Cheaper, faster, bigger construction and wind load, tolerancing and brittleness are recurrent feature in reports of failures to this day.

Detailed studies of wind loading were carried out for the enquiry and they led to 56lb/ft^2 (2.7 kN/m^2) being adopted for the robustly configured and braced Forth Rail Bridge with 521m spans which opened in 1890. This was also innovative as the first major structure in Britain to be constructed of steel.

Buckling failures. Imitators of innovators often take risks from a combination of cost cutting and ignorance. A cantilever construction, similar to the Forth Bridge, was adopted for the 549m span Quebec Bridge, but with slender and 'economical' design of its struts. It collapsed after a strut started to bow and then suddenly buckled during construction 1907. A more robust design was then adopted, but construction was interrupted by a lifting failure of the main span in 1916. It was finally opened in 1919 and remains in service.

The Quebec buckling failure highlighted the importance of slenderness and imperfections with columns and struts. Developing simple safe procedures for design against buckling of columns was a research priority in the first half of the 20^{th}C. However in most structures the columns interact with the beams in structural frames. As jointing improved with the black bolts and rivets being superseded with high strength bolts and welding these beam column interactions became more important. The steel industry set up the Steel Structures Research Committee, 1929 to 1936, whose work and their frustrations at the poor implementation of their results by BSI committees, is well set out by Baker [8].

This illustrates the tension between those engineers who want fundamentally based evaluation of real strength (essential for all Forensic Engineering) and those who want simple codes which lose all the complexities in a substantial factor of safety and requirements about configuration to ensure that structures are ductile enough to redistribute stresses to fit the rules. The committee loaded test frames to failure and also instrumented commercial buildings during construction, measuring changes in stress distribution as loads increased. This work showed the pin ended assumption, then universally used for designing members, overestimated moments in beams which were reduced by partial end fixity. However these moments led to over stress from bending induced in the columns.

Despite its initial rejection by BSI, this research bore fruit after the war when the proper consideration of joint stiffness and permissible redistributions of moments from joint stiffness and plasticity slowly became incorporated in codes for both steel and concrete. With the advent of computer structural analysis from the 1970s this could be fully considered in 2-D and eventually 3-D analysis of frames

Baker [9] went on to apply forensic investigation and analysis to the sometimes remarkable robustness of bomb damaged structures as large deflections established alternative load paths. Byfield [10] has built on this work in considering the prevention of progressive collapse after blast damage.

Freyssinet [11] building the innovative concrete arch bridges over the Allier, of which only Boutiron (1912) survived the war, found that shrinkage and creep led to problems with excessive deflections. Previously it had been assumed that concrete would be elastic like steel without longer term deformations. His subsequent research led to his development of higher strength concrete and post tensioned and prestressed concrete construction methods without which many landmark structures of the 20thC (eg Sidney Opera House) would not have been possible.

Load, Strength and Factors of Safety.

By the end of the 19th C the basis of design were loadings approximated to the worst condition expected. For bridges herds of cattle and steam engines gave the worst loading. Design of structural members was based on elastic analysis of each element normally with assumed pinned connections. Under the design load the permissible or working tensile stress was limited to (minimum yield stress)/(factor of safety). The factor of safety adopted had been progressively reduced from about 4 to around 2 as testing and improvements in materials gave greater confidence. This convention was universal until the 1970s.

The majority of design in the 1930s to the 1950s followed the approach as set out in Steel Designer's manual [12] with safe load tables for steel beams and columns and in the Reinforced Concrete Designer's Manual [13]. These summarised and tabulated the requirements of standards so elements could be sized from axial loads, bending moments and shears determined by simplified structural analysis. Both manuals continue to be updated and now cover Eurocodes. Knowledge of the evolution of these Manuals is essential when appraising older structures [14].

There were concerns that the use of this single universal factor of safety applied to all types of structure was leading to some uneconomic designs, hindering progress and in other instances to great risks of failure. Pugsley, whose previous career had been in aircraft design and testing, chaired the Institution of Structural Engineers (IStructE) committee on Structural Safety which reported in 1955 [15]. They recommended the adoption of factors to reflect the uncertainty of Loading, Analysis and Workmanship and the Economic and Public consequences of a failure, as summarised in Figure 2. This provided a logical framework which differentiated between theatre balconies and isolated water towers, as well as covering the range from best to worst practice in design and construction. Sir Alfred Pugsley also led the formation in 1976 of SCOSS, (The Standing Committee on Structural Safety), now Structural-Safety.

It was only in 1972 when CP110 was brought out, to update and unify concrete design for reinforced, prestressed and precast concrete, that partial factors for load γ_f and strength γ_m were introduced, albeit in a very simplified form. It was intended that they should be properly calibrated and reflect the uncertainties and potential consequences for each mode of possible structural failure. This was to be combined with the statistical evaluation of the variability of loading and of strength replacing 'maximum' and 'minimum' values with 'characteristic' values with a defined probability of being exceeded. This was too innovative for many engineers.

X factors. Loading, Analysis and Workmanship

		B. Loading Uncertainty		
A. Workmanship	C. Analysis	very good	good	fair
good	good	1.45	1.75	2.05
	fair	1.60	1.95	2.30
	poor	1.75	2.15	2.55
fair	good	1.70	2.05	2.40
	fair	1.90	2.30	2.70
	poor	2.10	2.55	3.00
poor	good	1.95	2.35	2.75
	fair	2.20	2.65	3.10
	poor	2.45	2.95	3.45

Y factors. Economic and Public Consequences

E. Economic Consequences	D. Danger to Public		
	Not Serious	Serious	Very Serious
Not Serious	1.0	1.2	1.4
Serious	1.1	1.3	1.5
Very Serious	1.2	1.4	1.6

Factor of Safety = X x Y

"Report of a Committee on Structural Safety" (IStructE 1955).

Figure 2. Simplified Table for Factors of Safety [15].

To appease the traditionalists, the old concrete code, CP114, remained acceptable, but was not updated. Another consequence was that instead of calibrating the new partial factors to achieve economy with more consistent reliability, the factors were adjusted so that designs to the old CP114 and new CP110 codes would be similar. Calibration studies which showed that shear failure merited a higher partial factor γ_m than the generally conservatively designed flexural modes of failure were not acted on. For concrete the partial factors, with minor adjustment, have remained un-calibrated to this day and have been passed on into the Eurocodes. The proper evaluation of partial factors for concrete is a challenge for the 21stC.

Some Failures of Concrete Structures since 1965.

In the 1960s and 1970s concrete was perceived and aggressively marketed as a 'maintenance free' material. However a range of problems with concrete construction become increasingly apparent and their investigation led too slowly to changes in standards and guidance. These include:

Ferrybridge Cooling Towers, 1965. Three out of eight 115m high cooling towers collapsed in severe wind conditions. This highlighted the inadequacy of the wind loading standards which were not intended for structures of this form and scale. In the 1930s these concrete shells had been of modest height, typically to 30m. Over the years they had been progressively enlarged while retaining the 125mm wall thickness which eroded safety margins.

Ronan Point, 1968. The 22 storey tower block was newly constructed using the Larsen-Nielsen precast concrete large panel system. A gas explosion in one flat at the 18th Floor level, lifted and pushed out the wall panels, which triggered the progressive collapse of a corner of the building. Four people were killed. The original panel system was designed for six storey

buildings, but had been extrapolated. It had poorly designed joints which did not tie the panels together. The detailing also led to a range of construction defects, deficient fire protection and sensitivity to deterioration which only became fully apparent when it was demolished in 1986. The enquiry recommended the strengthening of similar structures with explicit design for explosion pressures and the adequate tying of elements of the structures. This tying forms the 'generally', but not universally, applicable means of preventing progressive collapse, as specified in BS8110. Lack of durability, poor thermal efficiency but above all lack of fitness for purpose as homes, have led to the continuing progressive demolition of tower blocks.

The enquiry also found that wind loadings for this scale of structure were inadequate, which expedited the new code. While the 5 pages and 1 table in RC Designer's Handbook of 1957 were inadequate, it is questionable if the increasing elaboration with overstated precision of the 140+ pages of BS 6399 is appropriate, given the inherent uncertainty of gusts in extreme wind conditions and the oversimplification of other aspects of design.

High alumina cement. HAC was favoured for some precast concrete elements because of its rapid strength gain. However, if made with too high a water/cement ratio and exposed to warmth and humidity, it loses strength (conversion), with time. The collapse of roofs made of prestressed concrete beams using high alumina cement in three buildings led to HAC being banned in the UK in 1976. Detailed investigations showed that poor tolerancing and bad fit of precast elements were also contributory to these collapses as well as several other precast system failures. Precast elements in buildings from before the ban are subject to special checks for HAC conversion.

Alkali Aggregate Reaction. AAR became apparent in a range of UK bridges and buildings in the early 1980s, causing severe and growing cracking after five to ten years. Material scientists identified the slow swelling of silica gel created by the attack of high alkali cement on some siliceous aggregates. However there was no information on how this could affect structural strength. This was serious as the structures included the twin Marsh Mills three lane viaducts each with twelve 30m spans and a 400 bed 12 storey hospital. Another Forensic Cycle started to develop specifications for new construction and methods of appraisal and remedial work for damaged structures.

Site investigation and monitoring was carried out on over 100 structures with AAR, with laboratory testing and liaison with universities and research centres in the UK and internationally. By combining data from extensive test programmes on affected structures and parallel research by others, the IStructE guidance [16] on appraisal of AAR was developed. Severe cracking of foundations, Figure 3, and other problems led, after a 10 year programme of testing, monitoring and local strengthening, to the demolition and reconstruction of the Marsh Mills viaducts and the hospital. However, most structures, with mild AAR and/or good containing reinforcement, remain in service, Wood [17].



Figure 3. AAR cracked pile cap.

Myths in the literature about AAR abounded at the start of this work and some persist. *'It doesn't develop below ground',* but that is where the worst damage occurs, Figure 3. *'The reaction stops after 10 years',* but long term monitoring has now shown that cracking can continue to spread and enlarge after 50 years. In consequence some 'mild' cases are becoming 'severe' requiring re appraisal. RILEM TC ACS [18] is now bringing together international experience and preparing guidance in AAR-6.1 (Diagnosis & Prognosis) and the forthcoming recommendation AAR-6.2 on Assessment.

Once the risk of AAR was identified UK specification recommendations were based on limiting the alkali content of mixes (<3kg/m^3 Na_2O_{eqiv}), but where cement supply and aggregate types are different other approaches are needed.

Chlorides. It was well known that Chlorides could induce corrosion of steel in concrete. From the start of the 20thC higher specification concrete and cover was specified for marine conditions. However CaCl was a permitted accelerating admixture for increased strength gain in precasting and for winter concreting. During the 1960s reports of corrosion from CaCl accumulated, with prestressed element recognised as being a particular risk. Initially piecemeal restrictions on the use of CaCl were introduced. The IStructE recommended banning it for structural concrete in 1975, but it was 1977 before BSI amended CP110 with strict limits on chlorides in concrete from all sources including admixtures, cement and aggregates. When investigating corrosion in earlier concretes, checks for CaCl beyond the carbonation front are prudent, but its distribution can be patchy as it would only be used in cold conditions.

Deicing salts. NaCl was rarely used to deice roads before 1965 and it took more than a decade for the Cl to penetrate to the reinforcement and for the consequent corrosion risk to bridges to become apparent. Road salt becomes concentrated by ponding and seepage and so becomes more aggressive than marine conditions. Information from premature chloride induced corrosion of tunnels, bridges and car parks has provided the basis for concretes with improved but not immortal resistance to the ingress of Chlorides. Important lessons include configuring the structure to avoid the creation of aggressive conditions where the structure is highly stressed and improved detailing to facilitate compaction.

Punching shear failure. The partial progressive collapse of Pipers Row car park in 1997 was initiated by a punching shear failure at one column head which spread to seven others. HSE commissioned a full investigation and published the comprehensive report, Wood [19]. The collapse occurred in the early morning with no applied load on the deteriorated slab, so there were no casualties. Errors in design standards, in the design and in construction and progressive deterioration all contributed to the collapse. Salt induced corrosion was not a factor, but this is often wrongly reported. Frost damaged concrete with deep carbonation leading to corrosion had been repaired above two columns but had gone undetected over the column where the collapse initiated.



Fig. 4, Pipers Row Punching Failure.

The collapse initiated revisions of guidance on design, IStructE [20], appraisal and management of car parks. Consultants and clients cooperated in pooling information from investigation reports on 200 deteriorating car parks, Henderson [21], to provide input for the revised guidance. Continuing deterioration has led to demolition and premature replacement of many car parks. In 2012 CROSS (Confidential Reporting on Structural Safety) and the British Parking Association issued a further warning to owners about inadequate inspection and maintenance.

Carbonation induced corrosion. Corrosion of shallow reinforcement as carbonation penetrates neutralising the protective alkalinity is the most widespread form of concrete deterioration. Once a structure has been badly constructed the timescale is fixed for deterioration. In time this leads to a decade or two of expensive cosmetic repair before premature demolition. This is the consequence of the neglect of good practice establishes 100 years ago for mix design, compaction and curing and the provision of adequate cover to the reinforcement. Lax British Standards, which had no prospect of achieving their unsustainably short quoted 50 year 'Design Life', have contributed to these failures. Poor site quality control is another factor.

Progress, based on investigations of premature deterioration, has been made in improving durability by better design, detailing, materials and site workmanship for major bridge and tunnel structures. But commercial pressures, architectural caprices and contractual innovations which fragment responsibilities on lesser projects are creating durability problems for the future.

The principles for repair of concrete are well set out in Eurocode, BS EN 1504-9, but in practice it is hard to achieve more than temporary patching. This is a particular problem with listed structures [22], where matching the brutalist exposed concrete features, so beloved by architects, adds to the difficulties.

Box Girder and Suspension Bridges

In 1970 buckling initiated collapses of steel box girders during construction at Milford Haven (four died), at West Gate (Yarra) in Melbourne (35 died) and then at Koblenz in Germany in 1971 (nine died), led to major Forensic Cycle from which much of current international bridge design and construction practice emerged. All engineers involved in designing and constructing bridges should read the detail in the full investigation reports on these failures. They should also read the wider analysis of known bridge failures by Smith [23], which is as valid today as when it was written in 1977.

Flint & Neill coordinated the Merrison investigations of steel box girder collapses and deficiencies, the UK research program, and the development of interim assessment rules. At the same time, the team acted for the contractors in applying the still developing new rules to the rebuilding of both West Gate and Milford Haven Bridges. Much of the Merrison research was reported at the Institution of Civil Engineers (ICE) Conference [24] and by Rockey [25]. The quantification of variability of loading and strength and the calibration of partial factors followed the principles set out by Pugsley and developed in CIRIA Report 63 1977. This was simplified by requiring the highest standards of design and workmanship (with independent checks) for major bridges for which the 'Economic consequences' and 'Danger to the public' are always 'Very Serious'. The conclusions from Merrison were oversimplified to become the basis for UK bridge design in BS5400, much of which is now incorporated in the Eurocodes.

The detailed and damning Australian Royal Commission West Gate Bridge report [26], published within 8 months, on the technical, contractual and human factors that led to the collapse, should be read in full by all involved in bridge design and construction. It provides an interesting contrast to the scarcity of published information on Milford Haven and other collapses where information and proper prompt investigation have been prevented by bureaucratic, police or legal constraints.

The West Gate report highlights that it was the combination of poor communication between organisations and individuals and weak connections between elements of the structure that led to the collapse.

The eccentricities in the bolted stiffener connections in the flanges were a contributory factor in the West Gate collapse. It is much more difficult to achieve a full strength site joint in bolted construction than with welding, which was almost universally used elsewhere. Bolted construction was specified because in 1962 brittle fracture of welds caused the failure of the innovative King Street Bridge in Melbourne. This was the first bridge to adopt welded high strength steel construction in Australia. The problem arose because the steel testing and welding specification, which had been well prepared for the main contractor, was not passed to the fabrication sub-contractor or to the steel supplier. Aversion to one form of construction after a failure can lead to failure with an alternative form.

The collapsed bridges were a small part of the problem. Checks on other UK box girders showed that 32 of 51 completed bridges and 28 of 37 of those under construction needed strengthening. Independent design checks by a separate company for both the construction stages and for the in service design of all major bridges were recommended by Merrison and immediately adopted by the UK government. Traffic controls while this strengthening work was carried out caused economic disruption until the end of the 1970s.

In 1945 Germany had few bridges, little steel and many out of work engineers from the aerospace industry. These innovative German engineers developed light deep stiffened box girder bridges using an understanding of three dimensional stress analysis and buckling gained from aircraft and rocket design.

In the UK some engineers tried to imitate the innovators using old British Standards for girder bridges. They did not check the validity of these codes for structures of a scale and slenderness where local plasticity and buckling rather than yield became the limiting factor. Finite element analysis was in its infancy and the complex 3-D stress field of shear with biaxial compression in and around the diaphragms at supports, was not understood, so they were not adequately stiffened. Figure 5 shows the buckled webs and diaphragm which initiated the collapse of Milford Haven Bridge.



Figure 5. Milford Haven diaphragm.

The UK bridge loading code in the 1960s was for bridges up to 50 m long, but it had a graph of reducing intensity of load for long spans up to 1000m. This was based on an assumption in

1947 of widely spaced traffic. On the Severn Bridge the independent check showed that the designers had not considered torsional loading on the 1000m main span, so the rocker supports under design loads could be overloaded by a factor of over two.
The designers said '*Don't worry; one never gets the design loading'*. Detailed analysis of traffic showed that close spaced slow moving early morning traffic, predominantly laden lorries awaiting the tolls, could give loads about twice those in the old code. Re-strengthening for these higher loads necessitated a further round of traffic disruption on longer span bridges. The loading was then further increased as permitted lorry weights increased from 32 tonnes to 44 tonnes.

Reacting to the Tacoma Narrows suspension bridge collapse from wind oscillations, the designers of the Severn Bridge and Humber Bridge adopted an innovative torsionally stiff streamlined aerodynamic profile which was intended to reduce drag and wind loading. It was argued that in extreme winds there would be no traffic on the bridge. The aerodynamic profile significantly increased the wind speed across the deck so that overturning empty lorries became a major problem on the bridge. More rigorous evaluation of risks showed that with rising winds the most likely scenario is a bridge with high drag from jammed traffic after an overturning, with no way of clearing the traffic before the peak gust could hit the bridge. So the 'innovative' aerodynamics adopted was counterproductive. It was over twenty years before the lesson was learnt and a degree of wind shielding for traffic was introduced for long span UK bridges to reduce traffic disruption by high winds.

Most designers shunned steel for bridges in the UK for three decades after 1970 and the few long span bridges adopted plate girders to avoid the three dimension complexities of boxes. So when the research had developed the knowledge for the design good box girders, it was not applied in the UK. The oversimplifications introduced as the Merrison rules were replaced by BS5400 also made it difficult to exploit the full potential of slender stiffened box girders

Conclusions

Each new generation of engineers must learn how current design practice and standards have evolved from the investigations of past failures, so that they can innovate successfully without repeating the mistakes of previous generations. Maintaining our infrastructure will require a thorough understanding of historic design and construction procedures.

The unimpeded flow of information around the Forensic Cycle is essential for improving construction practice. Progress is inhibited by the fragmented processes for developing and revising Eurocodes, which are increasingly remote from the construction industry. Legal and commercial factors which conceal information are also detrimental.

To create sustainable durable structures for the future we must improve construction practice and materials, but most importantly the detailing. Investigations of our prematurely deteriorating infrastructure will provide the best guide for improving durability performance.

There are patterns common to failures of all types of structures which provide an indication of risk. The Innovative, Complex and Unusual 'ICU' in the structure and/or in contract procedures are often risk factors which merit particular attention in design and construction management.

Prior to embarking on any design or construction task engineers should check the validity of published standards for their project and familiarise themselves with incidents and

deficiencies of fitness for purpose in recent and historic precursors of their project. Standards committees should also regularly review performance of structures and materials covered by their standards and they should be explicit about the limits of applicability of standards.

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Forensic Engineering: Informing the Future with Lessons from the Past
ISBN 978-0-7277-5822-4

ICE Publishing: All rights reserved
doi: 10.1680/feng.58224.021

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publishing

Learning from Structural Failure: The Challenges and Opportunities

S. P. Brady, Brady Heywood, London, UK & Brisbane, Australia.

ABSTRACT

Learning the lessons from structural failure necessarily involves investigating the organisational and technical causes of failure, and ensuring these lessons are disseminated to the wider structural engineering profession to prevent reoccurrence.

Yet despite the obvious benefits of such an approach, individuals and organisations are generally reluctant to disclose or acknowledge failure, and these psychological reactions can dominate their ability to learn the critical lessons. Understanding the role these psychological reactions play in inhibiting learning is a key step in developing practical methodologies for ensuring the lessons of past failures are not forgotten.

This paper reviews and discusses some of the unique aspects of anticipating, responding to, and learning from all forms of failure, not only those of a structural nature. The paper reviews the barriers to learning, the challenges of identifying and analysing failure, the importance of 'near-misses' in anticipating failure, and explores how these insights can provide guidance to the structural engineering profession on how best to ensure effective learning occurs.

Introduction

Many engineering authors have examined structural failure and discussed the importance of learning its lessons. Texts such as Feld and Carper's *Construction Failure*[1] and Kaminetzky's *Design and Construction Failures*[2] introduce structural failure and illustrate some of the common failure mechanisms in a range of structures and materials. Carper's *Forensic Engineering*[3] and Ratay's *Forensic Structural Engineering Handbook*[4] provide an introduction to forensic engineering, the investigation of failure. And Delattes's *Beyond Failure*[5] provides case studies for engineers wishing to understand some of the key lessons learned from the influential structural failures. Similarly, the ASCE's *Journal of Performance of Constructed Facilities,* the ICE's *Forensic Engineering* journal, as well as organisations

such as *Structural-Safety*[6] (formally SCOSS and CROSS), play an important role in ensuring the lessons learned from past failures are disseminated to the wider engineering community.

Unsurprisingly, a common theme in the literature is the importance of using the lessons learned from past failures to prevent future reoccurrence[3,4]. While the benefits are self-evident, Henry Petroski, in *To Forgive Design - Understanding Failure,* elegantly articulates the wisdom of such an approach: 'engineering design is all about projecting past experience through the lens of present methods into the future. Documented failures are among the most valuable experiences because they reveal weaknesses in reasoning, knowledge, and performance that all the successful designs may not even hint at. The successful engineer is the one who knows not only what has worked in the past but also what has failed and why.'[7]

There is, of course, a rich history of the structural engineering profession learning from past failures. In some cases, these lessons have resulted in advancements in the profession's expertise. For example, the collapse of the Tacoma Narrows Bridge in Washington State in 1940 resulted in significant developments in wind modelling techniques and structural dynamics, the failure of the Ronan Point apartment block in London in 1968 illustrated the consequences of progressive collapse, and the failures that occurred in the U.S. in the 1970s and 80s emphasised the importance of controlling the 'human factors', such as human error and procedural issues, that contribute to structural failure[8].

However, the literature also suggests that learning the lessons from failure, particularly when 'human factors' are involved, is not a straightforward process[3]. Indeed, the wider (non-engineering specific) literature indicates that individuals and organisations are generally reluctant to disclose or acknowledge failure, and these psychological reactions can dominate their ability to effectively capture and learn the critical lessons[9].

The objective of this paper, therefore, is to examine the challenges involved in learning from failure from a general, rather than solely a structural engineering, perspective, and to explore how these insights can provide guidance to the structural engineering profession on how best to ensure effective learning occurs. The paper reviews and discusses the barriers to learning, the key aspects of identifying and analysing failure, and the role played by 'near-misses' in anticipating failure. The paper concludes by exploring how these factors highlight opportunities within the structural engineering profession to better learn the critical lessons from past failures.

The Challenges in Learning from Failure

A significant body of literature from a diverse range of backgrounds, such as psychology, business, and medicine, addresses the challenges involved in effectively learning from failure. A comprehensive literature review is beyond the scope of this paper, so a number of specific topics that are of interest to the structural engineering profession are examined, namely: the barriers to learning from failure, the challenges of identifying and analysing failure, and the importance of near-misses in anticipating failures.

Barriers to Learning

While there are numerous definitions, failure can broadly be defined as a 'deviation from expected and desired results'[9]. The definition not only covers obvious failures, such as the catastrophic collapse of the I-35W bridge in Minneapolis or the financial collapse of a company, but also includes the more routine, day-to-day failures, which although less dramatic, occur more frequently.

The wisdom of learning from failure is well accepted, yet research indicates that organisations find such learning difficult. Indeed some authors conclude that learning rarely occurs[9,10]. Cannon and Edmondson identified technical and social barriers to learning based on research spanning a ten year period[9].

From a technical perspective, the authors conclude that human intuition, in the absence of a scientific approach, has limitations when identifying and analysing failure. These limitations can lead to individuals and organisations drawing incorrect conclusions as to the cause of a failure, with the potential for inappropriate or incorrect lessons being learned.

From a social perspective, barriers to learning exist on both an individual and organisational level:

- **Individual level**: Cannon and Edmondson conclude that the natural human reaction to failure is for the individual to distance themselves from that failure[9]. Even with 'small' failures, the authors' research concluded that 'rather than acknowledge and address a small failure, individuals have a tendency to deny the failure, distort the reality of the failure, or cover it up, and groups and organizations have the tendency to suppress awareness of failures.'[9] This reaction appears to have deep psychological roots, and it is related to the human need to be held in high regard by others. Ultimately, individuals prefer to move on from failure, rather than learn its lessons.

  Further, even if the intent is to investigate the failure, the authors' research indicates that heuristics (e.g., "rules of thumb") and psychological biases reduce the individual's ability to undertake objective analysis[9]. The individual's natural reaction to failure is, therefore, inconsistent with objective failure analysis. For example, individuals tend 'to be more comfortable attending to evidence that enables them to believe what they want to believe, denying responsibility for failures, and attributing the problem to others or to "the system".'[9] Consequently, the authors stress the importance of the scientific method in providing a basis for rigorous analysis to overcome these barrier and minimise the reliance on subjective intuition.

- **Organisational level:** Organisations typically reward success and discourage or punish failure, which works as a disincentive to failure identification, let alone failure analysis. For example, Cannon and Edmondson found that: 'pervasive social barriers, psychological and organizational, discourage *reporting* failure, just as technical barriers such as system complexity and causal ambiguity inhibit *recognizing* failure.'[9] Even when failures are identified, lessons may not be learned because 'social factors inhibit the constructive discussion and analysis through which shared learning occurs.'[9]

  The issue is further complicated by a lack of understanding of the difficulties involved in learning from failure, for example 'not only do few organizations systematically capture failure's lessons, most managers lack a clear understanding of what a proactive process of learning from failure looks like.'[9]

In summary, these technical, individual, and organisational factors introduce barriers that must be overcome before practical strategies for learning from failure can be implemented.

Actively Learning from Failure

Cannon & Edmondson conclude it is possible to learn the lessons from failure, but stress that it is a process, rather than an outcome, and it is counter-intuitive and difficult to implement in practice[9].

Proactively learning from failure consists of three distinct phases, namely identifying failure, analysing failure, and deliberate experimentation[9]:

- **Identifying failure**: Identifying that a failure has occurred is the first step in failure response. However, while the identification of the obvious or significant failures can be straightforward, identifying the 'smaller' failures or near-misses is more challenging.

  The barriers to identifying failure are a consequence of the individual and organisational reactions to failure discussed in the previous section. However, identifying small failures and near-misses is of great importance, with the wider literature concluding that the more significant failures and collapses are generally preceded by small failures or near-misses that went unnoticed. For example, Edward Rogers, the chief knowledge officer at NASA's Goddard Space Flight Center, concluded that 'almost every mishap at NASA can be traced to some series of small signals that went unnoticed at the critical moment'[10]. (A more comprehensive examination of how these small signals or near-misses can be used to anticipate failure is presented later in this paper.)

  Therefore, what is required is a combination of systems and procedures to collect the information necessary for failure identification, along with a leadership approach that aims to remove the stigma of reporting failures. (Indeed, one of the first steps in overcoming the barriers to identifying failure is to recognise that failure, in some form, is inevitable when operating in a complex environment, such as medicine or structural engineering[9]). For example, Cannon and Edmondson describe an approach taken by Electricite De France, a nuclear power plant operator, in identifying failure: 'The organization tracks each plant for anything even slightly out of the ordinary and has a policy of quickly investigating and publicly reporting any anomalies throughout the entire system so that the whole system can learn.'[9]

- **Analysing failure**: Analysing failure to determine its root causes is a complex process, with the wider literature highlighting a number of barriers that can compromise sound failure analyses.

  Social barriers include the negative reactions that individuals have to failure - as discussed above. These reactions tend to result in heuristics and biases that compromise the objectivity of the failure analyses, potentially leading to incorrect failure lessons.

  This issue is further exacerbated by technical barriers relating to a lack of expertise on the part of failure investigators. Cannon and Edmondson found that this factor results in 'an inadequate understanding of the scientific method and an inability to engage in the following aspects of rigorous analysis: problem diagnosis, experimental design, systematic analysis of qualitative data, statistical process controls, and statistical analysis.'[9] In the absence of rigor, subjective intuition has the potential to dominate and compromise the analysis.

- **Deliberate experimentation.** Deliberate experimentation is essentially trial-and-error in a controlled manner. For example, some organisations 'recognize failure as a necessary by-product of true experimentation, that is, experiments carried out for

> the express purpose of learning and innovating. By devoting some portion of their energy to trying new things, to find out what might work and what will not, firms certainly run the risk of increasing the frequency of failure. But they also open up the possibility of generating novel solutions to problems and new ideas for products, services and innovations.'[9]

The Role of Near-Misses in Anticipating Significant Failures

As indicated earlier, near-misses can play a key role in anticipating significant failures. Dillon et al, in *How near-misses influence decision making under risk: a missed opportunity for learning*, define near-misses as 'successful outcomes in which chance plays a critical role in averting failure.'[11]

Cannon and Edmondson argue that near-misses are 'often the "early warning signs" which, if detected and addressed, may be the key to avoiding catastrophic failure in the future.'[9] Taking a familiar example from the construction industry, it is common for construction companies to insist that not only incidents, but also near-misses, be reported - a practice based on the assumption that near-misses are incidents waiting to happen.

The concept of near-misses can be further explored with reference to NASA's Columbia Disaster. In 2003, the Columbia space shuttle disintegrated during re-entry, with the subsequent failure investigation identifying that the probable cause of the incident was the result of a briefcase-sized piece of foam detaching from the main fuel tank and impacting the left wing of the Columbia shuttle[12]. This impact compromised the Shuttle's Thermal Protection System (TPS), allowed the heat generated during re-entry to penetrate the shuttle's wing structure, with the resultant failure of the wing culminating in the loss of the shuttle.

However, the investigation also found that foam impact on the day of the failure was not an isolated event. Foam impacts had also occurred during shuttle launches in previous flights, and their occurrence was raised as a concern by NASA's engineers: shuttle design assumed that the TPS would experience no impacts whatsoever, and engineers were very aware that any impact had the potential to damage the TPS and result in loss of the shuttle on re-entry.

In the previous flights, the size of the foam and the location of its impact were such that no catastrophic failures occurred. In other words, chance played a role in averting these failures, i.e., these impacts were near-misses. During Columbia's re-entry, however, catastrophic failure did occur because a sufficiently large lump of foam had struck the heat shield at a critical location during the launch. Hall, in *Columbia and Challenger: organizational failure at NASA*, concluded that, given the evidence of previous impacts, the potential for failure of the TPS was well known, yet these near-misses went unheeded[12].

Tinsley et al, in *How to Avoid Catastrophe*, an article based on seven years of research in different industries, indicates that failing to learn from near-misses is an issue for many organisations, not only NASA. The authors found that 'multiple near misses preceded (and foreshadowed) every disaster and business crisis we studied, and most of the misses were ignored or misread.'[10] Therefore, learning from near-misses appears to be just as difficult as learning from the more significant failures. Tinsley et al found that 'two forces conspire to make learning from near-misses difficult: Cognitive biases make them hard to see, and, even when they are visible, leaders tend not to grasp their significance.'[10].

The authors conclude that two biases in particular cause these near-misses to be ignored:

- **Normalisation of deviance**: Normalisation of deviance is 'the tendency over time to accept anomalies—particularly risky ones—as normal'[10]. Returning to NASA,

engineers viewed foam impacts to shuttle TPS's as a concern because the shuttles were operating outside their design parameters - the foam impacts were a deviation from NASA's expectations. However, as the number of impacts continued to occur during numerous flights, and no catastrophic failure occurred, the impacts became less concerning. In other words, 'what was originally a cause for concern no longer raised alarms, deviance became normalized'[11].

- **The outcome bias**: Research shows that individuals and organisations tend to treat near-misses as successes[9], and when 'people observe successful outcomes, they tend to focus on the results more than on the (often unseen) complex processes that led to them'[10]. In essence, while causes of significant failure are occasionally investigated, causes of success are rarely investigated, and near-misses tend to fall into the category of successes. For example, research indicates that successful managers typically attribute successes to their abilities or the good decisions they made, rather than the misfortune of their competitors or the role played by 'blind' luck[13]. As a consequence of the outcome bias, identifying the root causes of near-misses is typically not undertaken.

The literature indicates that there are seven stages to examining and learning from near-misses: managers should '(1) be on alert when time or cost pressures are high; (2) watch for deviations from the norm; (3) uncover the deviation's root causes; (4) hold themselves accountable for near misses; (5) envision worst case scenarios; (6) look for near misses masquerading as successes; and (7) reward individuals for exposing near misses'[10].

These steps are consistent with the three stages of learning from failure discussed above (identify, analyse, deliberate experimentation), with one additional aspect: the concept of a *deviation*. When it comes to developing an information collection system for identifying near-misses, the measurement of a deviation, rather than a failure, is a powerful distinction. While deviations can be measured objectively, failures are much more difficult to identify and quantify - as has been discussed previously.

For example, returning to NASA, shuttle TPS's were impacted on numerous occasions during launches, despite the 'expectation' that no impacts would occur. Therefore, the evidence of impacts confirmed the presence of a deviation, and the deviation indicated the potential for failure.

Before leaving the subject of near-misses, it is important to note that while the literature indicates that near-miss information can assist in anticipating failures, it also concludes that near-miss information can lead to risky decision making.

The issue is related to how individuals separate *statistical* risk from their *perception* of that risk. For example, Dillon et al take the analogy of an individual attending meetings in a part of the city where there is a statistically high probability of being mugged[11]. If, however, the individual attended a number of these meetings and was not mugged (and felt safe), then the individual's *perceived* risk would decrease, even if the *statistical* risk remained the same. As a consequence, the individual is now more likely to be comfortable going to this part of the city in the future than an individual who is only in possession of the statistical risk information. The research concludes that given the same level of statistical risk, individuals in possession of near-miss information perceived a lower level of risk[11].

In summary, possession of near-miss information is analogous to aircraft pilots avoiding collisions: 'every time a pilot avoids a collision, the event provides evidence both for the threat and for its irrelevance. It is not clear whether the learning should emphasize how close the organisation came to a disaster, thus the reality of danger in the guise of safety, or the fact that disaster was avoided, thus the reality of safety in the guise of danger'[14].

The Opportunities for the Structural Engineering Profession

As has been discussed, there is a rich history of the structural engineering profession learning from failure. This is particularly true for significant failures, which have typically been subject to comprehensive and transparent investigations, in some cases Royal Commissions or Commissions of Enquiry. These investigations are good examples of the application of the scientific method and forensic process to accessible failure information to objectively determine the causes of failure – which is consistent with the key elements identified in the wider literature as being critical to learning from failure.

However, the wider literature suggests two areas where the structural engineering profession can enhance how it approaches learning from failure: the role of the scientific method in failure investigation, and the role of small failures and near-misses in anticipating significant failures.

The importance of the scientific method in the analyses of failures is highlighted in the literature. From a structural engineering perspective, the scientific method underpins the forensic process of failure investigation, and typically consists of collecting evidence, developing failure hypotheses, testing each hypothesis against the collected evidence, and determining the cause of failure. This process is quite different to the more conventional process of engineering design, which is a process of synthesis that utilises simplifying performance assumptions relating to probable loads, structural behavior, and material properties to create a structural design that meets a client's requirements. Engineering investigators who are familiar with implementing the design process may find themselves ill-equipped to investigate failure in a forensically sound manner - which may lead to incorrect diagnosis of causation[15]. Therefore, ensuring the wider structural engineering profession is aware that the application of the scientific method to failure investigation, in the form of the forensic process, is critical to ensuring the correct lessons are learned.

The wider literature also stresses the importance of the role played by information on ‘small failures’ or near-misses in anticipating the significant failures. However, access to information on small failures or near-misses is difficult to obtain, and the wider literature indicates that individuals and organisations are reluctant to identify or discuss failure, with most eager to move on from the failure. In some cases, especially where a dispute is likely to arise, legal barriers can prevent the open discussion of the failure and the lessons learned. Of course, monitoring and analysing this near-miss information is not a simple task and is likely to present many implementation difficulties. However, the wider literature strongly indicates that such monitoring has the potential to assist in anticipating the significant failures and highlights the importance of organisations such as *Structural-Safety*.

Closure

Future structures will be designed and constructed based on the lessons learned from the past - whether or not they be successes or failures. Structural engineering has a proud history of learning from failures, with many of the contributions to engineering expertise occurring as a result of significant failures. Failure will remain an ever present aspect of the profession, and while it can be minimised, it cannot be eliminated. It is, therefore, critical that lessons are learned so as not to repeat previous mistakes.

The profession's ability to learn these lessons will require more than good intentions about the wisdom of learning from failure. Rather, it needs to appreciate the unique nature of failure, listen to the early warning signs from near-misses, and train its engineers to identify, analyse, and disseminate the lessons from failures.

In summary, the profession needs to ensure that failures are discussed as much as successes.

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Forensic Engineering: Informing the Future with Lessons from the Past
ISBN 978-0-7277-5822-4

ICE Publishing: All rights reserved
doi: 10.1680/feng.58224.029

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Forensic Investigation and Performance Analysis for Bridges under Extreme Natural Hazards

Wei F. LEE, National Taiwan University of Science and Technology, Taipei, Taiwan
Masato ABE, BMC Corporation, Tokyo, Japan
Misko CUBRINOVSKI, University of Canterbury, Christchurch, New Zealand
Chung I YEN, National Taiwan University of Science and Technology, Taipei, Taiwan
Men-Feng WU, Directorate General of Highways, Taiwan
Chin-Fa CHEN, Directorate General of Highways, Taiwan

Abstract

This paper is aim to report authors' work on forensic investigation and performance analysis of bridges under recent major natural hazards, including 2009 Typhoon Morakot in Taiwan, 2010-2011 Christchurch Earthquakes in New Zealand, and 2011 Tohoku Earthquake in Japan. In each studied case, reconnaissance works of bridge failures are first presented. Forensic analyses on damage causes or bridge performance are then introduced in details. In addition to the case studies, an integrated multi-hazard monitoring system for safety warning and performance analysis is also introduced including its verification during 2009 Typhoon Morakot. Lastly, lessons learned as well as engineers' awareness to multi-hazards including design improvement and safety monitoring are summarized.

Introduction

In recent years, extreme monsoon events and devastating earthquakes had caused serious damages to infrastructures including bridges. Failures of bridges not only result in threats to vehicles and people travelling on bridges, but also interrupt emergency evacuate and rescue passages. Such viaduct failures often led to enlargement of disaster resulting in severe causalities and economic loss. Bridges failed during extreme natural hazards are often in large number and have complicated failure causes including environment system collapses and massive structural malfunctions. Forensic investigation and performance analysis for bridges

damaged in natural hazards are more challenging than those failed in general service condition; general investigation guidelines or specific observational methods for such forensic investigation are not an easy task. However, needs on design or retrofit code revisions are usually raised rapidly after these extreme hazards. Results of the forensic investigation and analysis have deterministic influences on upgrading guidelines of risk assessment, hazard mitigation, structural design, and retrofit. Purposes of this paper are to introduce the forensic investigation on bridge structures conducted for recent extreme natural hazards, and to provide insights of performance analysis for bridge failures caused by such extreme natural hazards. In this paper, authors' work on forensic investigation and performance analysis of bridges under recent major natural hazards, including 2009 Typhoon Morakot in Taiwan, 2010-2011Christchurch Earthquakes in New Zealand, and 2011 Tohoku Earthquake in Japan, are reported. Features of these three major events and reconnaissance works of bridges damaged by scour, flood, debris flow, tsunami, strong ground shaking, and soil liquefaction are first summarized. Forensic analysis on selected bridge failures are then depicted in detailed. In addition to the case studies, an integrated multi-hazard monitoring system for safety warning and performance analysis is also introduced including its verification during 2009 Typhoon Morakot. To end, lessons learned as well as engineers' awareness to multi-hazards including design improvement and safety monitoring are summarized in an effort to improve design and retrofit of bridges under threats of natural hazards.

Collected Study Cases

Bridge damages occurred during 2011 Tohoku Earthquake in Japan, 2010-2011 Christchurch Earthquakes in New Zealand, and 2009 Typhoon Morakot in Taiwan are collected by the authors as the studied cases. Abe and Shimamura report railway bridge damages caused by strong ground shaking and Tsunami waves during the 2011 Japan Tohoku Earthquake (Abe and Shimamura, 2013). Cubrinovski and his colleagues describe performance of bridges in liquefied deposit during 2010-2011 Christchurch Earthquakes in New Zealand. Lee and his colleagues present results of forensic investigation and research progress on performance monitoring on bridges suffering scour, flood, and debris flow during Typhoon Morakot invaded Taiwan in 2009.

Railway Bridge Damages during the 2011 Japan Tohoku Earthquake

2011 Tohoku Earthquake occurred at Japan Standard Time 14:46 on March 11th. The magnitude of the earthquake was 9.0, which has been the largest in Japanese observation history. Observed Japan Meteorological Agency's seismic intensity scale includes a record with an intensity of 7, which is the maximum value of the scale. Both strong ground motions and a tsunami were observed in wide area of east half of Japan. It caused approximately fifteen thousand deaths and over three thousand people missing. Railway systems in the area

were also heavily affected. Direct loss to JR-EAST is estimated at least 58.7 billion JPY. Figure 1 shows the observed seismic intensities recorded by JR-East seismometer network during the earthquake. Figure 2 shows the locations of infrastructure damages along the Shinkansen (the bullet train) line running through the Tohoku area (Abe and Shimamura, 2013).

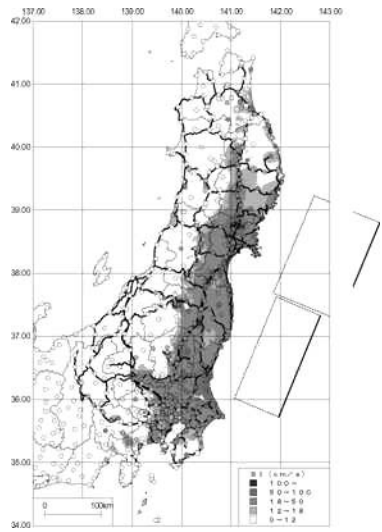


Figure 1 Observed seismic intensities recorded by JR-EAST seismometer network in 2011 Tohoku Earthquake (after Abe and Shimamura, 2013).



(a) 3.11 main shock, (b) 4.7 aftershock

Figure 2 Locations of infrastructure damages along Tohoku Shinkansen (after Abe and Shimamura, 2013).

Railway bridge damages during this devastated earthquake could be categorized in two major groups: damages caused by strong ground motions and failures caused by tsunami waves. For reinforced concrete viaducts of Shinkansen, some cracking and spalling of concrete were observed at the top of the columns as shown in Figure 3. However, failure or collapse was not observed. No major damage is reported for structures with seismic retrofit after the 1995 Kobe Earthquake. Interestingly, inclination and buckling failure of electric poles, majority of which is made of pre-stressed concrete, mounted on bridges and viaducts were widely observed as shown in Figure 4. Moreover, bearing and restrainer failures of bridge superstructures due to seismic motion were found, yet no major failure or collapse of superstructure was reported. At a steel girder bridge with skewed angle of 60 degrees, rotational motion of the girder seemed to be excited by strong horizontal ground motion and excessive displacement was observed at the girder end as shown in Figure 5 (Abe and Shimamura, 2013).



Figure 3 Column damage in 2011 Tohoku earthquake (after Abe and Shimamura, 2013).



Figure 4 Damage to Shinkansen electric poles (after Abe and Shimamura, 2013).

Along the coastal line in Northeast Japan where tsunami invaded area, many bridge girders were uplift, floated away, or totally washed away as shown in Figures 6 and 7. This type of failure modes was widely observed for bridges whose girders are not connected to piers. On the other hand, total collapse of rigid frame viaduct on coastal lines was also observed as shown in Figure 8. These failure patterns indicate that conventional structural design for lateral loads is not sufficient for the enormous tsunami load (Abe and Shimamura, 2013). Tsunami waves storming in and out had also caused serious scour to bridge substructures as shown in Figure 9.



Figure 5 Displacement of a skewed girder (after Abe and Shimamura, 2013)





Figure 6 Decks of bridges without fix connections between girders and piers were uplifted and floated away by the Tsunami waves during 2011 Tohoku Earthquake.

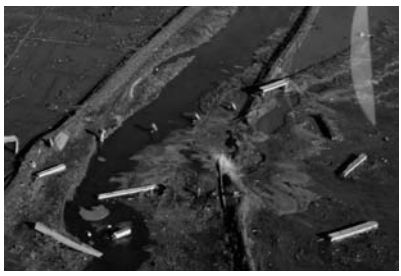




Figure 7 Collapsed pre-stressed concrete girder bridge in heavy tsunami inundated area (after Abe and Shimamura, 2013) .



Figure 8 Damage to reinforced concrete rigid frame viaduct by tsunami (after Abe and Shimamura, 2013).



Figure 9 Substructure scours caused by storming tsunami waves.

Soil Liquefaction induced Bridge Damages during the 2010~2011 New Zealand Christchurch Earthquakes

In the period between September 2010 and December 2011, Christchurch (New Zealand) and its surroundings were hit by a series of strong earthquakes including six significant events (Cubrinovski et al., 2013):
4 September 2010 (Mw=7.1),
22nd of February 2011 (Mw=6.2),
13th of June 2011 (Mw=5.3 and Mw=6.0) and
23th of December 2011 (Mw=5.8 and Mw=5.9) earthquakes.

The causative faults of the earthquakes (as shown in Figure 10) were very close to or within the city boundaries thus generating very strong ground motions and causing tremendous damage throughout the city. Among these series of earthquakes, the 22 February 2011 earthquake was particularly devastating. It caused 185 fatalities, collapse of two multi-storey reinforced concrete buildings, collapse or partial collapse of many unreinforced masonry structures including the historic Christchurch Cathedral. Widespread liquefaction in the

suburbs of Christchurch affected tens of thousands of residential buildings and properties, and shattered the lifelines and infrastructure over approximately one third of the city area. The total economic loss caused by the 2010-2011 Christchurch earthquakes is estimated to be in the range between 25 and 30 billion NZ dollars, or 15% to 18% of New Zealand's GDP (Cubrinovski et al., 2013). Cubrinovski and his research team had conducted extensive forensic investigation on infrastructure damages caused by soil liquefaction and consequence lateral ground deformation, i.e. "lateral spreading." In this paper, results of forensic investigation and performance analysis on bridges damaged by soil liquefaction and later spreading are reported in details.



Figure 10 Causative faults of the 2010-2011 Christchurch earthquakes (after Cubrinovski et al., 2013).

Figure 11 shows locations of the studied bridges, the Anzac Bridge and the South Brighton Bridge. Both bridges locate along the Avon River where serious liquefaction was observed. Figures 11 and 12 show characteristic soil profiles with SPT and CPT penetration resistances obtained near the studied bridges. As shown in Figure 11, the profile of South Brighton Bridge consists of uniform fine and medium sands, which are relatively loose in the top 5-6 meters, and then medium dense at greater depths down to at least 25 m depth. Figure 12 depicts the soil profile at the Anzac Bridge site. The profile consists of a variable top sandy soil to about 2 m, then a uniform fine to medium relatively loose sand from 2 to 12 m depth, and slightly coarser and denser sand below 12 m depth. Preliminary liquefaction analyses based on the soil profile data indicate that the shallow loose fine to medium sand deposits are where massive liquefaction occurred during 2010 to 2011 earthquakes (Cubrinovski et al., 2013).



Figure 11 Locations of the studied bridges (after Cubrinovski et al., 2013)

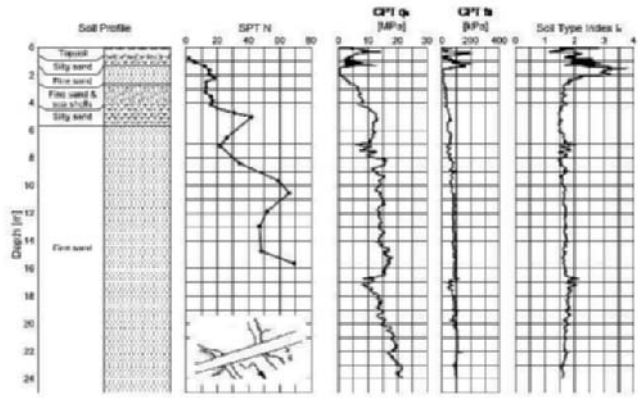


Figure 12 Soil profile of the South Brighton Bridge site (Cubrinovski et al., 2013).

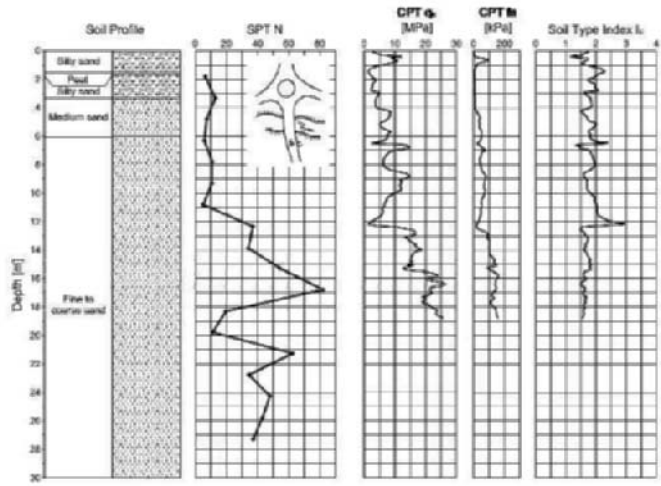


Figure 12 Soil profile of the Anzac Bridge site (Cubrinovski et al., 2013).

The South Brighton Bridge is located at the mouth of the Avon River. The 65m long three span RC (reinforced concrete) bridge was constructed in 1980. The bridge superstructure consists of cast in situ RC deck on precast I beams, and is supported through elastomeric bearings on two octagonal 'hammerhead' RC piers and seat-type RC abutments. The foundations of the bridge consist of 44 precast concrete piles, ten beneath each abutment and twelve beneath each pier. The piles are 450 mm octagonal in shape, either vertical or raked (4 on 1), rigidly connected to the abutments or 1.3 m thick pile caps at the piers. The abutment piles are 18.7 m long while those beneath the piers are 13.3 m long, making all pile tips to reach similar embedment depth below the river bed. In the area of the bridge, substantial lateral spreading occurred both during the 4 September 2010 and 22 February 2011 earthquakes. Permanent spreading displacements reached 2.18 m to 2.9 m respectively after the 22 February 2011 earthquake. Figure 13 depicts the mechanism of spreading-induced damage to the South Brighton Bridge which was also found to be typical for all bridges affected by lateral spreading. It involved large spreading-induced movement of the banks towards the river/bridge (as illustrated in Figure 13a), which was resisted by the stiff bridge superstructure. The rigid beam-deck superstructure practically prevented any displacement in the longitudinal direction of the bridge which resulted in deck-pinning and consequent back-rotation of the abutments (Figure 13b) about the beam-abutment point of collision, because the foundation piles could not resist the spreading movement of the foundation soils towards the river. The permanent tilt of the abutments of the South Brighton Bridge was about 7-8 degrees. This back-rotation of the abutments in turn caused damage at the top of the abutment piles, as shown in Figures 13c and 13d. Assuming that the abutment piles have rotated for the same/similar amount, the 8 degree abutment rotation infers that the top of the piles has displaced laterally towards the river approximately 20 cm. These large lateral displacements at the pile top in conjunction with the rotational constraints imposed by the rigid pile-abutment connection caused bending of the piles resulting in tensile cracks on the river side (Figures 13c and 113d) and concrete crushing/spalling on the land side of the piles (Figure 13d). A large distress in the foundation soils surrounding the piles with ground cracks, fissures and slumping towards the river were indicative of permanent spreading displacement of the foundation soils towards the river (Figure 13e).

The Anzac Bridge was constructed in 2000. It is a 48m long three span bridge which consists of a precast concrete deck, precast concrete beams, and in situ concrete pier and abutments. Foundation of the bridge is reinforced concrete piles with dimensions of 1.5 m in diameter with 8 mm thick permanent steel casing and 20m in length. Design spacing of the pile foundation is 4.7times of diameter. Each abutment is supported on 16 "H" piles (310x133). The piles are 22 m long (reaching the same depth as the piles beneath the piers) and at 1.5 m spacing.

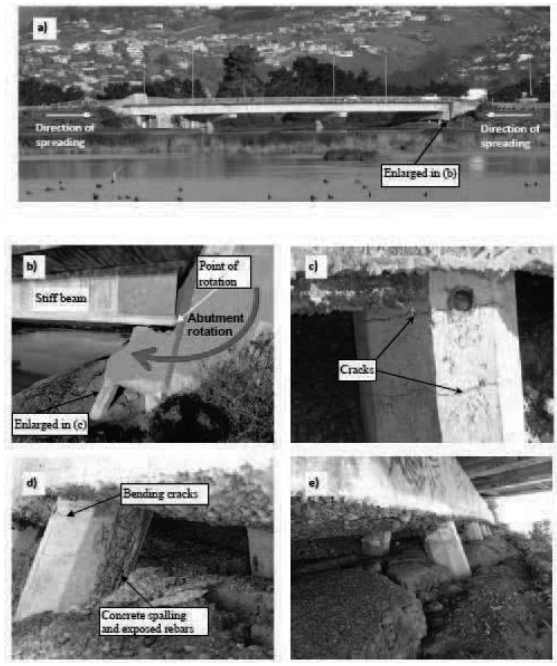


Figure 13 South Brighton Bridge - Illustration of typical spreading-induced deformation /damage mechanism for short-span (stiff deck) bridge (after Cubrinovski et al., 2013).

Massive liquefaction occurred in the area of the bridge during the 22 February 2011 earthquake, as illustrated in Figure 14a with the large volume of sand ejected, which is particularly visible on the south side of the bridge. There was a clear evidence of substantial lateral spreading towards the river, with large cracks and fissures in the ground running parallel to the river (Figure 14b). Permanent lateral spreading displacements of 0.94-1.12 m were measured at the river banks adjacent to the south abutment. The spreading-induced mechanism involving deck-pinning and back-rotation of the abutments was evident at the Anzac Bridge. Figure 14c illustrates with the red arrow the back-rotation of the south abutment which reached a permanent tilt of about 6 degrees. The horizontal arrows in Figure 14c indicate permanent lateral displacements of three elements of the structure and the surrounding soils: (i) lateral displacement at the bottom of the abutment (top of the H-piles) due to back-rotation of the abutment; (ii) lateral offset of the precast concrete underpass (Figure 13d); and (iii) lateral offset of the surrounding soil at the interface between the paved track and the RC underpass. The spreading-induced damage again caused slumping of the approaches resulting in large vertical offsets between the deck and the approaches, and the need for an immediate repair/infill of the approaches (Figure 14e). Compression deformation of the pavement at the approach – deck interface (Figure 14f) was also indicative of the

spreading action - superstructure reaction mechanism resulting in back-rotation of the abutments described above (Cubrinovski et al., 2013). Figure 15 is the schematic illustration that summarize the characteristic spreading-induced deformation (damage) mechanism of short-span bridges involving deck-pinning with consequent back-rotation of abutments, damage to abutment piles, and slumping of the approaches



Figure 14 Lateral spreading near the Anzac Bridge and induced structural damages of Anzac Bridge caused by lateral spreading (Cubrinovski et al., 2013).

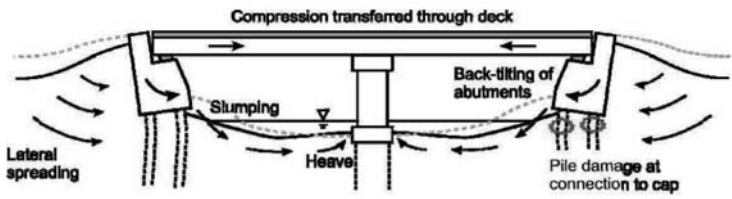
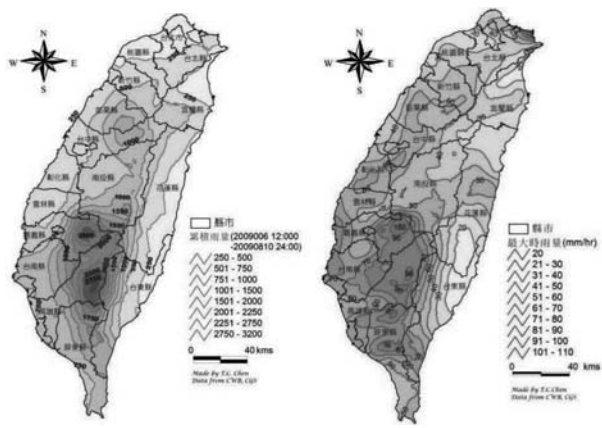


Figure 15 Illustration of lateral spreading induced bridge damages (Cubrinovski et al., 2013).

Bridge Damages during the 2009 Typhoon Morakot in Taiwan

On August 8th, 2009, Typhoon Morakot invaded Taiwan and had caused serious damages to south of the island. It had brought record high precipitation in both accumulation and intensity aspects to Kao-Ping river watershed. A major portion of highway bridges located in the affected region were suffered from various natural hazards, such as slope failures, debris

flows, floods, and scours. Typhoon Morakot was formed in south pacific and turned into mid-size typhoon while approaching Taiwan. Morakot strolled through northern Taiwan, yet it brought enormous precipitation to south-western Taiwan. Figure 14 present the statistical contours of accumulated rainfall and maximum rainfall intensity during Morakot's invasion. Accumulated rainfall exceeded 2500mm over a larger area in southern Taiwan; while the maximum rainfall intensity went beyond 70mm/hr and reached 100mm/hr for the most affected area down south. The observed enormous rainfall and rigorous rainfall intensity over southwestern Taiwan were believed to be the major factors resulted in the sever fatalities and infrastructure damages.



Accumulated precipitation hourly intensity

Figure 14 Precipitation accumulated and maximum hourly intensity of Typhoon Morakot (Lee, Ishihara, and Chen, 2011)

In an effort to study the performance of highway bridges under such an extreme hazard event, Lee and colleagues were under the assignment of Directorate General of Highways, Taiwan (DGH) to collect damage data and environmental information of total 52 highway bridges those failed during Morakot. Figure 15 shows locations of the investigated bridge sites. Major portion of the failed bridges situated along Ko-Ping River and its branches. The Ko-Ping River went from southern ridge the highest mountain, Jade Mountain in Taiwan. This area did suffer from 1999 Chi-Chi earthquake by serious ground shaking, yet no major typhoon event had ever attacked it directly before Morakot. Forensic investigation and performance analyses were conducted to provide DGH assesses to failure causes and mitigation strategies. Failure types and related causes are summarized as follow and shown in Figure 16 (Lee, Ishihara, and Chen, 2011).

Type I: Slope Failures- Bridge structures such as abutments, foundations, and even approach ways were hit by enormous soil mass produced by nearby landslides.

Type II: Debris Flows- Bridges located at the outlets of the watersheds where serious debris flows occurred. Debris either piled up the channels or directly hit the bridges

Type III: Over Scale Floods- The over-scale floods that caused by the record high heavy rainfall overflew the bridges and pushed the superstructures and substructures of bridges

Type IV: Improper Bridge Locations- Bridges located at the curved sides of the river channels. Large volume floods damaged the bridge foundations or related structures such as embankments or abutments. This type of failure could also be the results of sudden flow line changes or immense surges caused by debris or landslides mass blocking the river channels.

Type V: Flood Erosion- Large volume flood eroded the embankments, abutments, and approaches. Retaining structures failed by topping sliding, or backfill loss.

Type VI: Scour- Serious scours accompanied by floods damaged the bridge foundations.

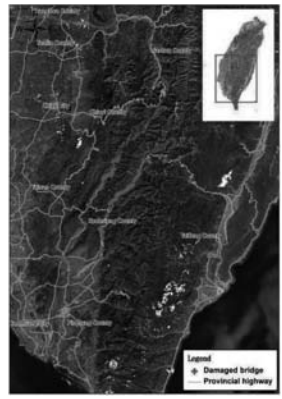


Figure 15 Locations of the damaged highway bridges during Typhoon Morakot in 2009 (Lee, Ishihara, and Chen, 2011).

In reality, most of the damaged bridges had suffered the combination effects of at least two failure types in reality. For upstream areas, most bridges were destroyed by debris and landslides. Locations of these affected bridges were at outlets of watershed areas or inlets of branches to main streams. For downstream areas, bridges were damaged by over-sized floods. It was a combination of heavy rainfall and massive debris pile up in the river channels. Figure 17 shows the before and after satellite environmental images and bridge photos. As depicted in the figure, aggressive landslides occurred in the catchment area had contributed tremendous amount of debris. The debris filled up the river valley and rammed the bridge substructures and foundations. The fill-up could be as high as 20m to 25m and leave very low to almost no clearance for the flood passing through the bridge. Superstructures of the bridges then suffered from the direct impacts of massive and rapid stream and debris flows.

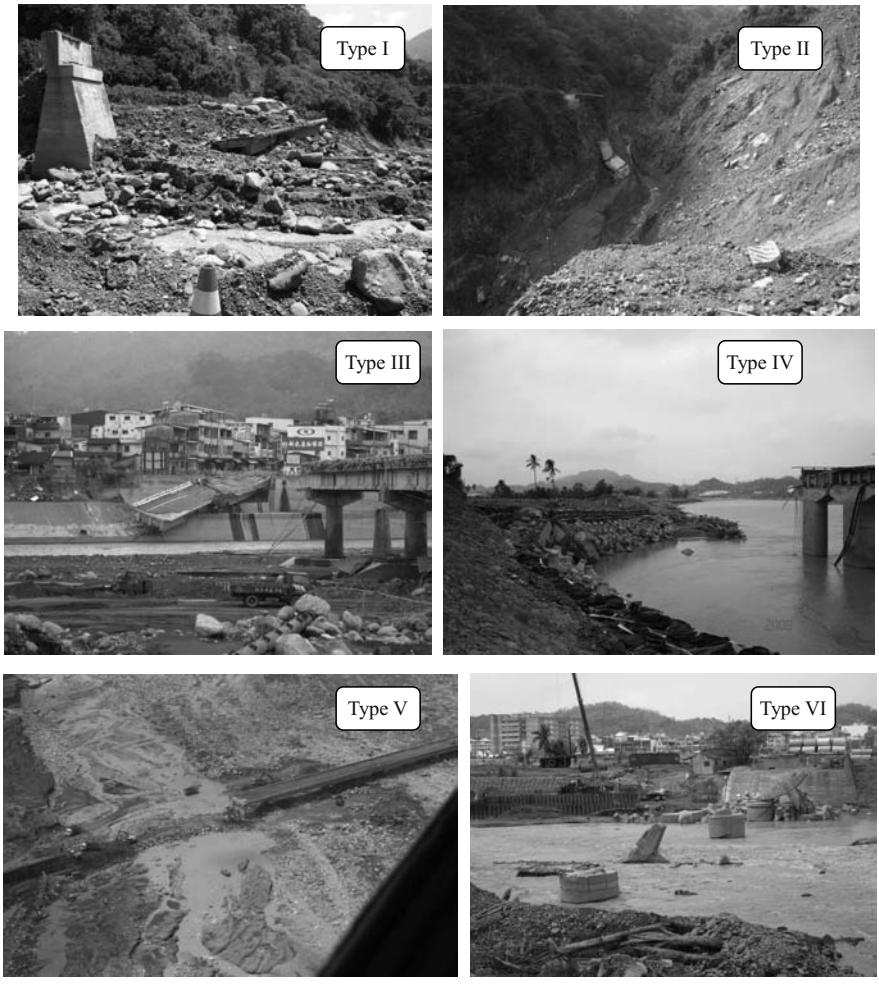


Figure 16 Summary of failure types and generic causes of bridge damages during Morakot (after Lee, Sihihara, and Chen, 2011).

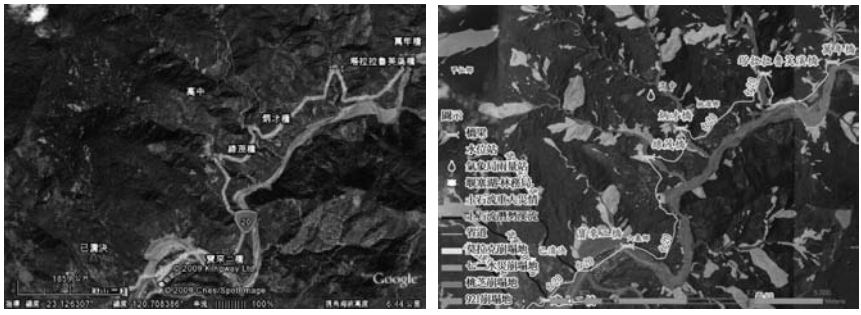




Figure 17 Before and after event images of a bridge failed during Morakot.

Performance Monitoring of Bridges under Natural Hazards

Intelligent Performance Monitoring System

For bridges encountering floods or debris flows, both time domain vibration amplitudes and frequency domain vibration intensities would increase from their normal time values because flow pressure acting on foundations increases. Flow pressure acting on bridge foundations could increase due to increase of flow velocity, debris mass hitting on bridge structures, and increase of exposed foundations caused by scour, i.e. larger foundation area suppressed by flow pressure. Furthermore, if piers or foundations of bridges were damaged, the apparent stiffness of substructure systems would fall resulting in probably natural frequency decrease and vibration amplitude (intensity) increase. Vibration characteristics of substructure systems including characteristic frequencies, corresponding time domain vibration amplitude, and corresponding frequency domain spectrum intensities, could be adapted as the observing performance indices for safety monitoring of bridges under multi-hazards including scours, floods, and debris flows (Lee at al., 2013). An intelligent performance monitoring system had developed by Lee and his colleagues in an effort to monitor variations of such vibration characteristics directly, thus stability of bridge structures could be refereed effectively. The proposed system integrates various sensors including 3 accelerometers aligned in three directions (X: bridge traffic, Y: transversal direction, usually flow direction, Z: vertical direction), 2 dynamic inclinometers aligned with XZ and YZ planes, build-in microcomputer, and hydraulic environment sensors including thermometers, water level sensors, and flow velocity sensors. Figure 18 depicts contents of the proposed system and the typical installation location. Fundamental theories, data acquisition scheme, as well as the adapted data interpretation method are described in details in the paper written by Lee and his colleagues (Lee et al., 2013). Advantages of the developed system include more secured location for installation, direct correlation to structural modal and extended applications to multi-hazards (Lee et al., 2013).



Figure 18 Contents and typical installation of the developed system (after Lee et al., 2013)

The proposed instrumentation was installed onto Shi-Tzou Bridge, which is one of the important highway bridges crossing the longest and largest river in Taiwan, Jou Suei River, just before Morakot hitting Taiwan for system verification and demonstration. Figure 19 illustrates the location of Shi-Tzou Bridge and Jou Suei River catchment and its photo prior to Morakot. Shi-Tzou Bridge is a 2.7km long PC girder bridge with regular span length of 35m. During the monitoring period, foundation retrofit works for improving scour and seismic resistance were undertaken. The developed system was installed onto two piers, Pier 36 and Pier 42. Pier 36 sits next to the northern bank of Jou Suei River and had suffered serious scour prior to Morakot. Foundation of Pier 36 was group pre-cast pre-stress concrete piles with 80cm in diameter and 28m in length. Pier 42 locates in the main river channel of Jou Suei River. It had just retrofitted by "under pinning" method one year before Morakot. Additional cast-in-situ piles with 1.5m in diameter and 50m in length were installed to strengthen the foundation. The foundation cap was lowered to 10m under the existing river bed. The pier structure was enlarged and reinforced by wrapping steel plates. Performance of Piers 36 and 42 gives good comparison between a scour-critical substructure and the retrofitted one. Both piers were equipped with an intelligent monitoring system. The water level sensor was installed under the deck between Pier 42 and Pier 43. During Morakot, although Shi-Tzou Bridge was not located in the above mentioned seriously damaged areas, it still experienced critical flood and scour as shown in Figure 20. Serious flood and debris hit foundation of Pier 36 and Pier 42 had possibly suffered contraction scour during Morakot. Traffic of Shi-Tzou bridge was closed on August 8th to 9th for safety reason. In addition, the power supply was shut off from around noon of August 9th to noon of 11th resulting in system interruption and data loss. Despite of loss of data near the end of the event, valuable performance data for bridge under floods and scours was still obtained when power supply was sustained.

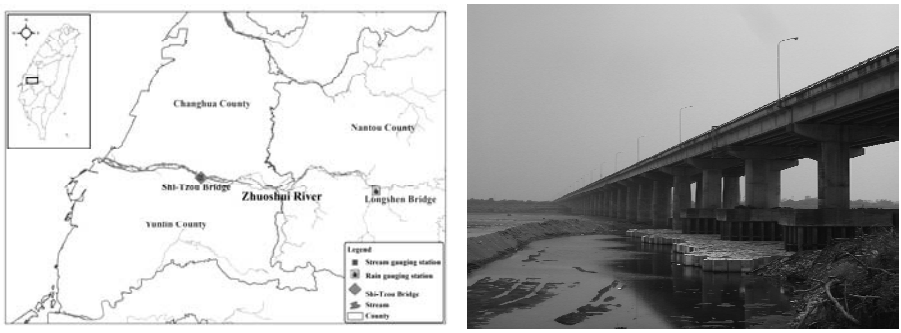


Figure 19 Location map and the photo of Shi-Tzou Bridge (after Lee et al., 2013).



Figure 20 Conditions of monitored piers during Morakot (after Lee et al., 2013).

Because Pier 36 was under retrofit construction when Morakot attacked, foundation of Pier 36 was exposed about 7m below the pile cap as shown in Figure 19. Concrete armor blocks were placed around the pier to protect the exposed pile foundation, and a temporary sand embankment was constructed in the upstream side to divide flow (Figure 19). During Morakot, the pier first experienced river bed backfilling caused by the embankment collapse. Then serious scour occurred when flow condition became scour critical. In addition, quite a lot of drifting tree trunks and debris were found jammed under the pile cap during the typhoon as shown in Figure 20. At end of the event, riverbed surrounding Pier 36 was backfilled by the sediment to about 2m to 3m below the pile cap. Figure 15-21a shows measured characteristic frequencies and flow volume rate increment according to time scale for Piers 36 in pre-defined X, Y, and Z directions. As depicted in the figure, the characteristic frequencies were all at their lower states at the beginning of the event, they were then increased to higher frequency levels assuming that riverbed backfilling had undertaken. When the flow volume rate reached the critical state, i. e. high enough to initiate scour, characteristic frequencies in all directions dropped to lower states; and they recovered to high frequency range when flow volume rate went back down to normal state at end of the event. Figure 16-21b shows frequency domain intensities of Pier 36 versus water levels in time scale. The trend that

measured intensities increased with water levels is clearly observed in the figure, especially during the period when measured characteristic frequencies reached their lower states as shown in Figure 15-21a. When water level went down at end of the event, scale of the measured intensities decreased as well. Figures 17-22a and 18-22b show the monitoring results of Pier 42. As depicted in Figure 17-22a, feature frequencies measured had clearly dropped to lower frequency range when flow volume rate was high; and it recovered to high frequency range when flow volume rate went down. The trend that response intensities increased with rise of water levels and decreased with decent of water levels is clearly observed in Figure 18-22b. Figures 19-23a and 20-23b plot measured frequencies versus intensities of monitored piers during Morakot in flow (Y) direction. As shown in the figure, general trends that the monitored piers gradually lowered characteristic frequencies and increased to high intensity level indicating scour development during Morakot. At end of the event, the pier resumed low intensity level indicating recession of flood and characteristic frequencies steadily increased to higher levels as riverbed surrounding bridge foundation was backfilled and stabilized.

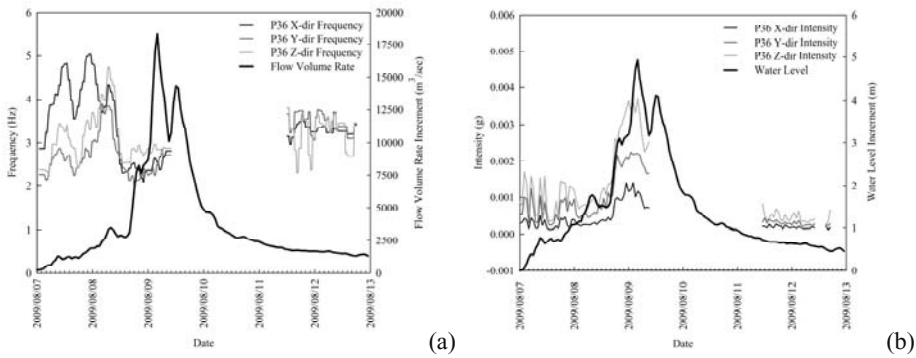


Figure 21 Monitoring results of Pier 36 (after Lee et al., 2013).

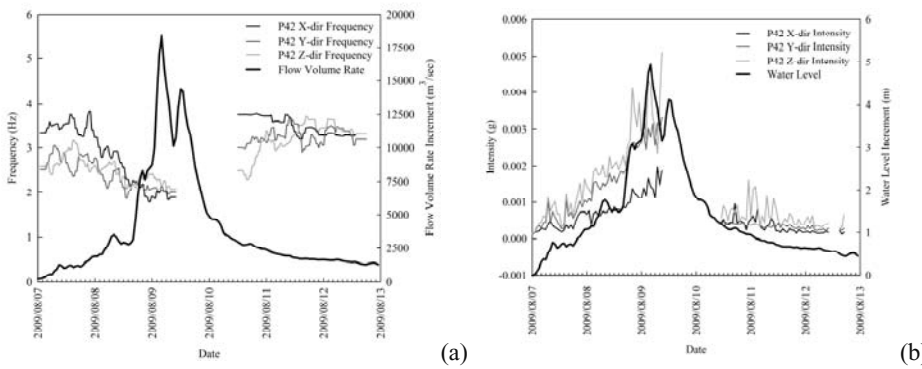


Figure 22 Monitoring results of Pier 42 (after Lee et al., 2013).

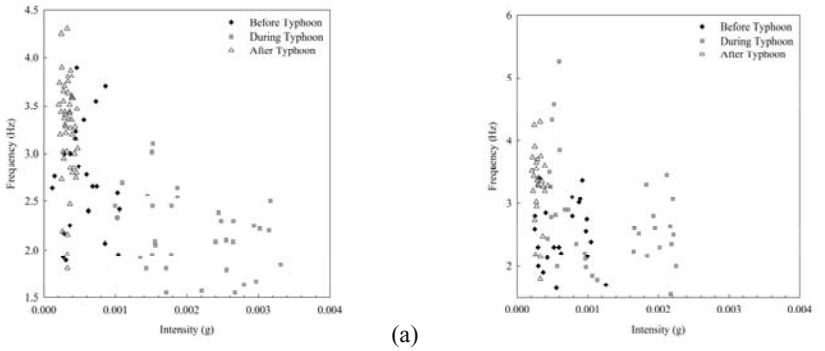


Figure 23 Measured characteristic frequencies versus response intensities (after Lee et al., 2013).

Conclusions

In this paper, forensic investigations conducted on different natural hazards and various performance analyses aiming to assess mechanisms of bridges damages during recent major natural hazards are presented. Typical failure pattern, generic failure causes, and behaviors of bridges under strong ground motion, devastating tsunami, soil liquefaction, and torrential rain induced multi-hazards are carefully studied. As a result of the investigation, the authors found that recent climate change and giant earthquakes have been testing both design and serviceability limits of bridges. Engineers are facing challenges of lack of experiences in mitigation such extreme natural hazards as well as in performance analyses of such infrastructure failures. Improved forensic engineering knowledge in assessing the vulnerability and analyzing performance of bridges under critical natural hazards are in great needs. Progress of this paper provides practical guidelines to engineers conducting investigation and analysis after such extreme natural hazards. Moreover, performance of bridges suffering different natural hazards presented in this paper offers important information for future bridge design code revision or improvement in considering extreme natural hazards.

Acknowledgement

The authors gratefully thank Professor Kenji Ishihara for making collaboration effort of this paper possible. The authors also appreciate researching colleagues who assist the studies and key references cited in this paper.

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Forensic Engineering: Informing the Future with Lessons from the Past
ISBN 978-0-7277-5822-4

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doi: 10.1680/feng.58224.049

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Investigation into highway bridge damage and failures during the November 2009 Cumbria flood event

John Collins MEng(Hons) CEng MICE, Ove Arup and Partners Ltd, Leeds, UK
Mark Steele BEng MSc DIC CEng MIStructE, Ove Arup and Partners Ltd, Leeds, UK
David Wilkes BSc CEng CWEM FCIWEM MICE, Ove Arup and Partners Ltd, Leeds, UK
David Ashurst BSc CEng MICE, Ove Arup and Partners Ltd, Solihull, UK
Prof Bill Harvey BSc PhD CEng FICE FIStructE, Bill Harvey Associates Ltd, Exeter, UK

Abstract
The 2009 floods in Cumbria, north-west England, were the most extreme in local living memory. River-spanning bridges across the county were affected, with damage and collapses at some locations. This paper presents a forensic engineering investigation into the flood conditions experienced by damaged and collapsed highway bridges owned by Cumbria County Council. The investigation highlights the manner in which an extreme event alters the statistical basis for calculating design flood levels. It also addresses the extent to which masonry arch bridges can resist high flows and highlights the importance of an effective bridge management regime.

Introduction
In November 2009 parts of Cumbria in northwest England experienced a prolonged period of very heavy rainfall. This resulted in significant flood events on a number of rivers throughout the county. According to Met Office data, on Thursday 19th November 316.4 mm of rain fell in 24 hours at Seathwaite Farm – the UK's wettest day ever on record(Met Office, 2010). Rain began falling on 17th November and continued without interruption for the next 72 hours. Rainfall which had occurred before the 17th had already saturated the ground, leaving little remaining storage capacity in the ground or in lakes. The consequential flooding lead to widespread erosion of river banks and damage to, or failure of, a number of bridge structures within or adjacent to river channels. In Workington, the collapse of Cloffocks No 2 (Northside) bridge tragically resulted in the death of Police Constable Bill Barker.

This investigation was to establish the mechanisms by which damage or collapse occurred to seven masonry arch highway bridges owned by Cumbria County Council as a result of the flooding. A comparison with undamaged bridges in similar locations was undertaken. Bridges investigated are located in Figure 1. Actions that could have been taken to avoid damage/failure were considered. A detailed understanding of the magnitude of the flood event on the affected rivers was required, in addition to a study of bridge records, anecdotal reports and site investigation post failure.

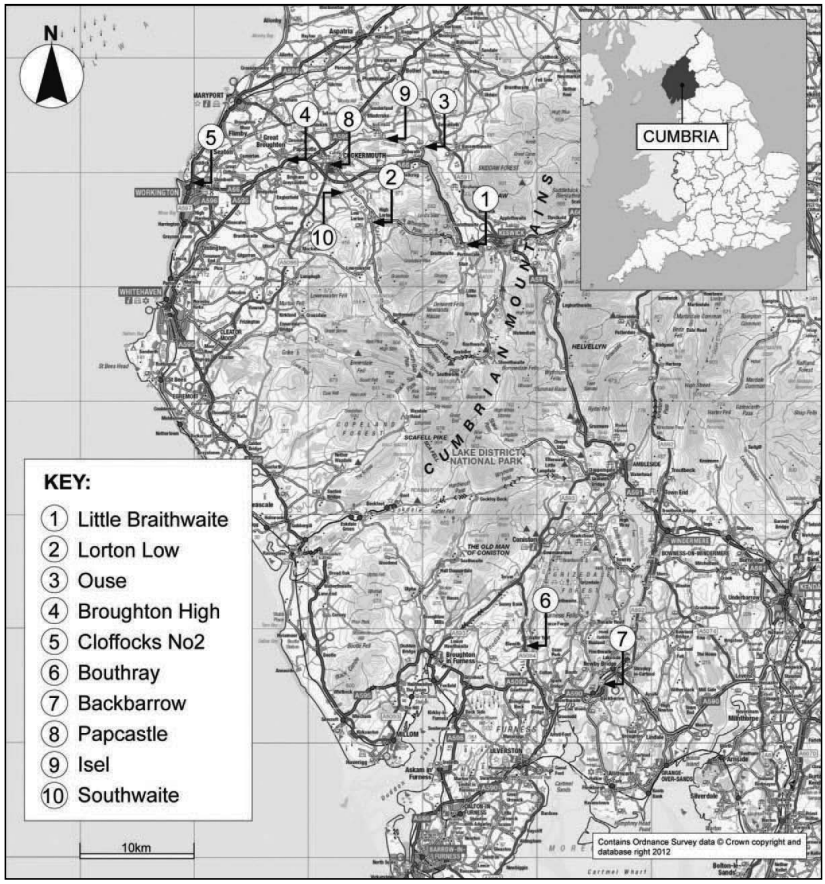


Figure 1 Location of bridges included in this investigation

The November 2009 flood event

Topography

The area of Cumbria affected includes the Cumbrian Fells of the Lake District. The area is characterised by mountainous fells climbing to over 900 mAOD. Between the fells are deep post-glacial valleys, many of which contain natural or manmade lakes. Rivers are hence relatively short and steep, characterised by rapid changes in level with short lag times from rainfall. Storage offered by the lakes creates a damping effect to the response to rainfall of river levels further downstream. The region is the wettest in England, fuelled by North Atlantic low pressure systems.

Fluvial and flood data

The water levels in several rivers in Cumbria on 19th and 20th November were the highest on record(JBA Consulting, 2010). Notably high water levels were recorded on the River Derwent and its tributaries. At Ouse Bridge the River Derwent level reached 3.51 m (relative to a local datum) before the recording range of the gauging station was exceeded and recording stopped. The previous maximum level recorded at the

site was 2.83 m in January 2005. 12 km downstream of Ouse Bridge at Cockermouth, approximately 200 people were evacuated by inland lifeboat or RAF helicopter due to flood depths in excess of 1.5 metres and impassable roads.

Downstream from Cockermouth, and 5 km upstream of Workington, the River Derwent at Camerton measured 4.04 m, compared to the previous maximum on record of 2.93 m in October 2008. Maximum flows at Camerton were unprecedented in records dating back 50 years (Figure 2). The effects of this single, large event on the statistical basis for calculating peak river flows for defined return periods are significant (Figure 3). For instance, at Camerton Gauging station a 1 in 100 year flood before this event would now be considered as a 1 in 12 year event.

Historic data from the Southwaite Bridge gauging station on the River Cocker, 2 km upstream of Cockermouth, was complete enough to allow the return period of the November 2009 volumetric flow to be determined as between a 1-in-400 to 1-in-500 year event. It was noted that at the time of the event, the floods were widely reported to have been a 1-in-1000 year occurrence(UK Parliament, 2010).

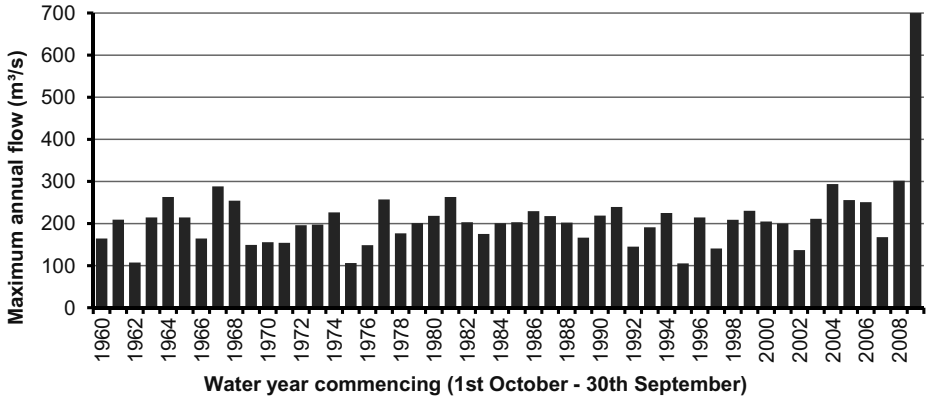


Figure 2 The annual maximum flow recorded at Camerton gauging station on the River Derwent, Cumbria(Environment Agency, 2010).

The damaged bridges

Restricted by the mountainous Cumbrian terrain, the local highway network is largely routed along the bottom of valleys with numerous river crossings. The majority of bridges on the local road network are masonry arch bridges using local stone. Many of the bridges are at least a century old and are owned by Cumbria County Council. The bridges were subject to biennial general inspections. Bridge maintenance is generally reactive in nature, driven by the recommendations of the inspections.

The investigation looked into the collapse of three bridges, and the extensive damage caused to a further four bridges. The condition of the bridges under investigation prior to collapse was determined from Cumbria County Council's records. Typically, these consisted of: a couple of decades' worth of inspection reports, records of recent maintenance work; assessment reports from a 1990s UK-wide assessment programme and underwater scour investigation records. As is typical for historic highway bridges in the UK, no as-built information was available.

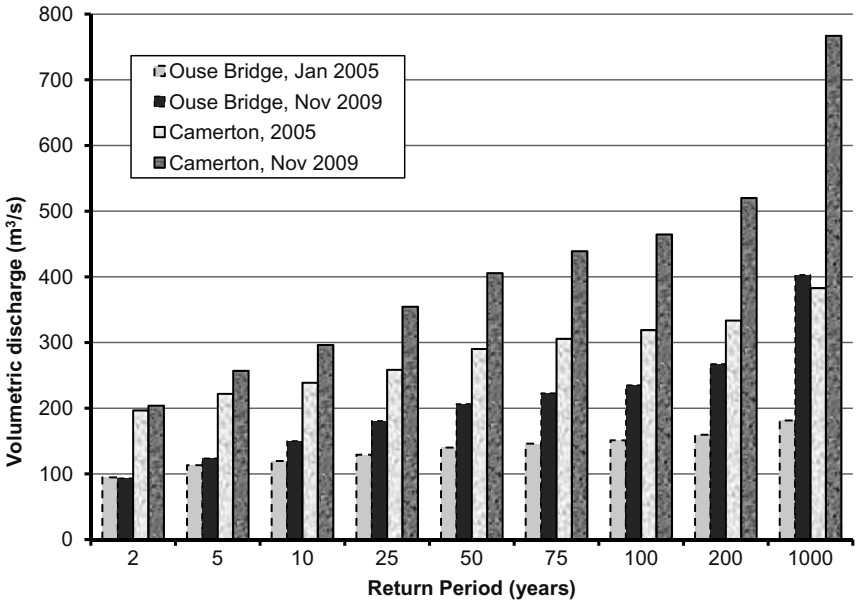


Figure 3 Maximum river discharge associated with defined return periods at Ouse Bridge, River Cocker and Camerton, River Derwent calculated using data available in January 2005 and November 2009.

Collapse or damage to the investigated bridges was considered to have been initiated by one of four effects: scour of the river channel; hydrostatic loading; hydrodynamic / debris impact and stone / mortar removal.

Cloffocks No 2 (Northside) Bridge, Workington

Cloffocks No 2 (locally referred to as Northside Bridge) carried the A597 over the River Derwent in Workington. The bridge formed a critical link between the town's northern and southern parts. Reference to historic maps suggests that the bridge was constructed at some time between 1900 and 1925. It was a three span masonry arch bridge in St Bees sandstone. Two spans at 22.8 m each spanned the Derwent with a central pier. A 3.6 m accommodation arch provided pedestrian access along the southern river bank, and was probably designed to act as an alleviation arch during very high tides.

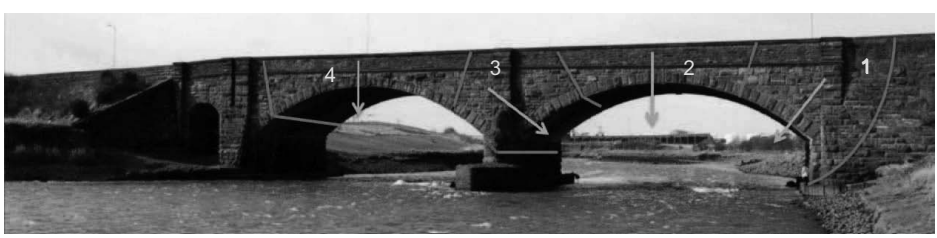


Figure 4 Cloffocks No 2 prior to 2009, looking downstream. Envisaged failure surfaces, sequence and direction of bridge collapse indicated. North bank far right.



Figure 5 Cloffocks No 2, spring 2010 from the north bank.

Prior to collapse, the bridge was in good condition with only minor repointing works recommended. Divers' reports indicated little scour. Scour protection works were present to north and south abutments and the river pier. By the start of the investigation the majority of the structure had either collapsed during the flood event, or had been cleared away since the collapse. The arches, north abutment and pier had completely collapsed but their foundations remained intact. The side span over the river bank had partially collapsed. The collapse of the main spans was reported to have been sudden and without warning. The side span over the river bank was thought to have collapsed shortly after.

As part of this investigation, a topographical survey of the banks, river bed soundings and a ground penetrating radar survey were conducted. This was compared with historic survey information from 1998. At some point between 1998 and the 2010 survey, a deep river channel had developed adjacent to the front wall of the north abutment. However, the north abutment's foundation was left intact. Any affect that scour may have had on the bridge may only have been relatively indirect, possibly causing a small displacement but not resulting in structural failure of the foundation.

Structural modelling using Archie-M demonstrated that the bridge was structurally adequate for traffic loads. Peak water level was just above springing level, making for hydrodynamic, debris impact or mortar scour of the arch ring leading to failure unlikely.

Having studied the arrangement of the remaining rubble post collapse, it was determined that collapse was initiated by failure of the north abutment. This could have been triggered by foundation movement from scour or attrition to the abutment masonry wall. Archie-M analysis demonstrated that the southern span's stability was dependent on the presence of the north span. Collapse of the south span would have occurred instantaneously following the north span's collapse. The partial failure of the southern river bank side span is reported to have occurred at a later time than the main spans. Scour calculations have suggested that scour to a depth greater than the side span substructure founding level was possible though this was not demonstrable on site.

Broughton High Bridge, Little Broughton

Broughton High is a four span sandstone arch bridge constructed in 1835 carrying a minor road over the River Derwent, 10 km upstream of Workington. Three 14.2 m spans cross the river and a 2.93 m span provides agricultural access on the southern bank. The bridge sits on a flood plain, embankments on the approaches forming a constriction to floodwaters. No scour had been recorded prior to November 2009, though significant scour to the banks up and downstream of the bridge occurred during the flood event. Damage was limited to loss of mortar in masonry within the river channel. These parts of the structure had not been inspected since 1996 due to high river levels at the time of inspections. It was not possible to determine whether this mortar loss had occurred specifically due to the flood event.



Figure 6 Left, Broughton High Bridge during the flood event. Right, scour to the river banks following the event.

Ouse Bridge, Bassenthwaite Lake

Ouse Bridge crosses the River Derwent at its outfall from Bassenthwaite Lake. The bridge has two 15.3 m sandstone arches, constructed prior to 1866. The bridge arches and supporting substructure were apparently undamaged during the flooding, though scour holes formed adjacent to the north abutment. Local loss of mortar and blockwork occurred to the north eastern wingwall. This could have been initiated by debris impact, though is likely attributable to local erosion of masonry joints.

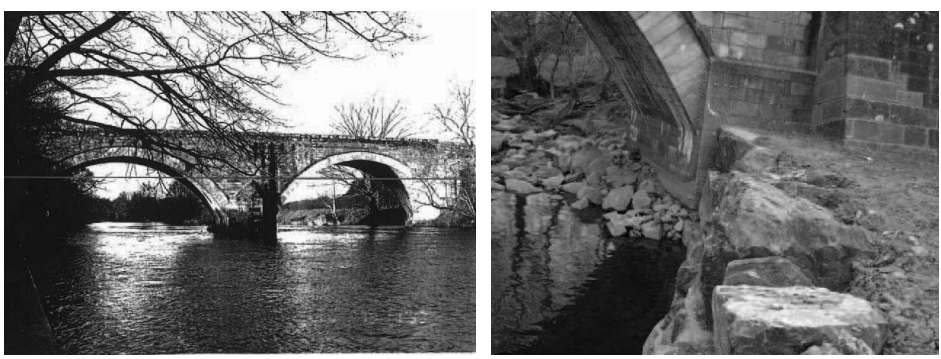


Figure 7 Ouse Bridge, upstream elevation with approximate river level during the flood event indicated (left). Post-flood riverbank protection & wingwall repairs(right).

Lorton Low Bridge, Lorton

Lorton Low Bridge carried an unclassified road over the River Cocker, 7 km upstream of Cockermouth. Each of the two spans were 8.3 m with a 1.5 m central pier (Figure 8). The super and substructure were of roughly hewn slatestone with irregular courses. The arch was standing stone construction. The parapets were sandstone. The arch rings, parapets and central pier were destroyed by the November 2009 floods. The abutments remained intact.

The bridge was assessed in 1996 using the modified MEXE method as suitable for vehicles up to 40 tonnes. A post-collapse Archie-M analysis suggests that the bridge would have been highly utilised in resisting 18 tonne live load. The flat geometry of the arches and relatively little fill material meant that the live to dead load ratio was high and load distribution to the arch was poor. Coupled with the potential for overloading, it was likely that the bridge would have been distressed by loading greater than 18 tonnes. It should be noted that the use of the modified MEXE method was best practice at the time of assessment and is still recommended by BD 21/01(Highways Agency, 2001). The uneven width arch rings joints were deep pressure pointed in the mid-1990s. The root cause of mortar loss was not recorded. Subsequent inspection reports do not make it clear if the ring pointing remained in good quality after these works. If the mortar in the arch barrel was missing or weakened, it would have lessened the ability of the bridge to withstand the adverse effects of the 2009 floods. Given that the abutments remained intact, scour of the substructure appears unlikely. Scour of central pier masonry cannot be discounted.

The hydrodynamic force on the central pier at peak flow and the impact force from a 3 tonne log were calculated. The combined effects of hydrodynamic and impact loads were inadequate to have caused collapse of the central pier or arch rings, but could have caused local damage or parapet collapse. Afflux effects have been estimated by calculation to increase the upstream water level by 1.2 metres. This estimate is approximate as it does not consider the flood waters by-passing the bridge by flowing around it. However, the bridge clearly formed a constriction on the flood waters.

Based on judgement and observation at other sites, bridge failure was likely to have been caused by scouring of the arch or pier mortar, causing removal of mortar and individual masonry units though there was no evidence left on site to support this hypothesis. Mortar or stone loss may have occured from debris in the flow. Once one stone is removed from the arch, more stones could quickly follow leading to local failure. Removal of backfill material could have then lead to a more global collapse.



Figure 8 Left: Lorton Low (upstream face) afflux during floods, upstream level (top line) 72.3 mAOD, downstream level (bottom line) 71.1 mAOD. Right, the bridge in spring 2010.

If the mortar joints were in a poor condition prior to the floods, it is possible that deep re-pointing could have increased their resistance to scour and limited the effect of localised impact damage. Such works would have been of most value if the root cause of mortar loss was also identified and dealt with. This could have been due to rainfall runoff passing through the arch ring due to ineffective waterproofing, or even distress by vehicular overloading.

Little Braithwaite, Braithwaite
This 7 m single span bridge of irregular slates carried an unclassified road over Newlands Beck, 3 km west of Keswick (Figure 9). The bridge had a signed 13 tonne load restriction. Structural modelling as part of this investigation confirmed that this was a sensible load restriction. It was reported that the arch ring had collapsed during the floods. The remaining structure (except the western abutment) and debris was removed from the river channel during the flood event due to the constriction it was causing to the river.

There was little remaining structure to inspect at the time of the investigation. Extensive scour to the eastern abutment took place during the floods. A review of inspection reports in the 16 years before collapse present conflicting and wide ranging remarks on the bridge condition. However, a 1997 inspection reported the poor condition of the arch ring which is attributed to over loading prior to the instigation of the 13 tonne load restriction. Without maintenance works, it was stated that the ring would continue to deteriorate. There are no records of such maintenance works ever having been conducted and the ring was assumed to have further deteriorated in the 12 years since the inspection.

There was a concrete scour slab on the river bed between abutments and there were no historic records of scour. Scour is thought to be an unlikely cause of collapse. Calculations suggest that the parapet had little resistance to hydrodynamic or impact loads. Parapet collapse and removal of exposed arch fill could have precipitated arch ring collapse. As commented at Lorton Low, removal of mortar by local erosion and stone removal, as observed at other sites, could have led to a progressive collapse. However, there was no evidence remaining on site to prove or disprove this theory.

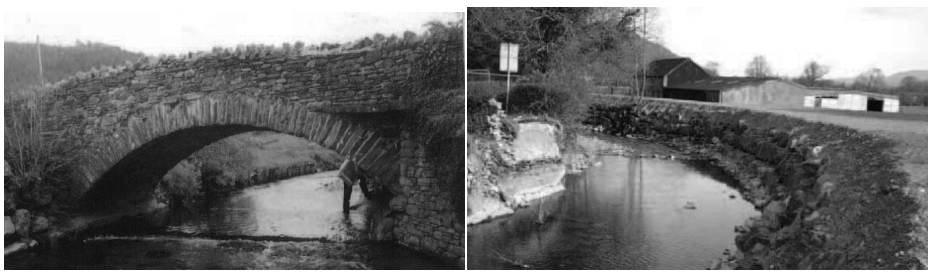


Figure 9 Left, Little Braithwaite downstream elevation, 1993. Right, Little Braithwaite from upstream, spring 2010.

Bouthray Bridge, Water Yeat
Bouthray is a two span masonry arch structure in Lakeland stone, carrying an unclassified road over the River Crake, 500 m downstream from Coniston Water. Minor scour had been noted to protection aprons prior to the flood event. During the

floods, the upstream third of both arches and the central pier were significantly damaged. Undermining of the scour protection apron due to scour occurred during the floods. Judging from the debris remaining on site, progressive collapse then worked its way back through the structure to a longitudinal joint within it from where the bridge was previously widened. Addressing the defective scour protection aprons noted prior to the floods may have prevented flood damage, but this is not certain.

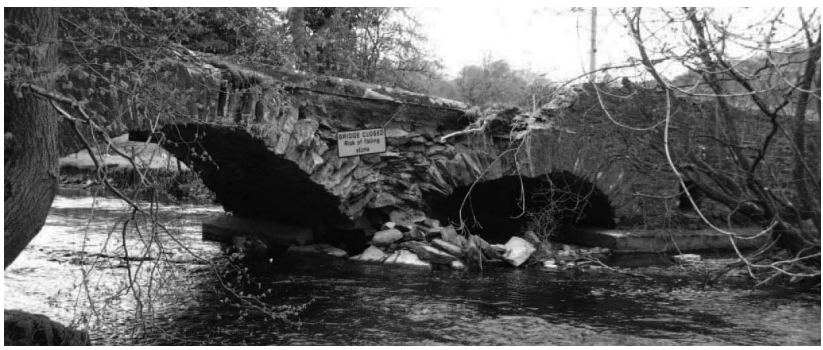
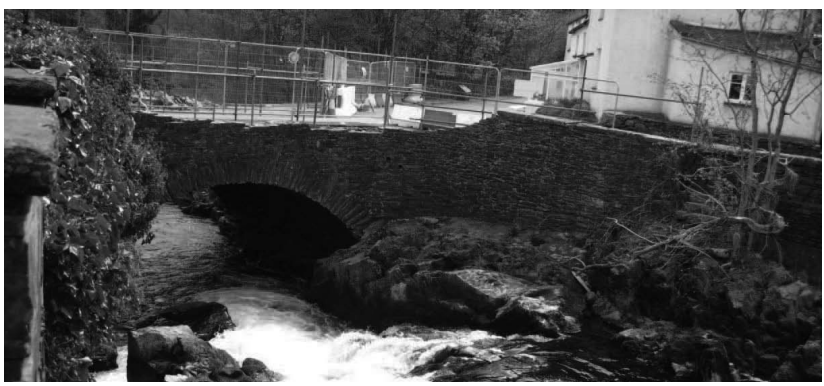


Figure 10 Upstream elevation of Bouthray Bridge following collapse.

Backbarrow Bridge, Backbarrow
The parapets of this 8.1 to 9.0 m single span masonry arch bridge carrying a minor road over the River Leven collapsed during the floods. The arch appeared to have suffered little damage.


Figure 11 Upstream elevation of Backbarrow Bridge

The water level rose to the top of the parapets due to afflux(Umpleby, 2009). Tensile stresses from hydrodynamic and hydrostatic loading exceeded the fracture limit of the parapet and cause extensive cracking as the load was plastically redistributed. These cracks would rapidly propagate in the presence of pressurised, turbulent water and would likely lead to collapse of the upstream parapet. Collapse of the downstream parapet was likely to have quickly followed, hastened by the sudden impact of the water released by the upstream parapet. Strengthening the parapets to current highways standards may have prevented collapse, but would also have

increased the hydraulic forces on the bridge structure, which may have resulted in other consequences.

Comparisons with undamaged bridges

To form a better understanding of the conditions which may have assisted in reducing the effects of the flood on these structures, three undamaged local bridges (Figure 12), which had been subjected to similarly extensive flooding were inspected.

Papcastle Bridge is a three-span steel / concrete composite highway bridge supported on concrete piers and abutments. The bridge was constructed in 1999 over the River Derwent south of Cockermouth. Although the southern river bank had suffered wide-scale scour and erosion, the bridge survived undamaged due to its relatively open elevations and use of resilient materials. **Isel Bridge** is a three-span masonry arch bridge which carries an unclassified road over the River Derwent approximately 4km west of the Ouse Bridge and Bassenthwaite Lake. Flood waters will have reached a similar level to Ouse bridge, though much of the flow bypassed the bridge to the south of it. **Southwaite Bridge** carries an unclassified road over the River Cocker some 3-4km downstream of Lorton Low Bridge. It is likely that this masonry arch bridge survived the flood because it was in better condition than Lorton Low, and the water was more easily able to by-pass the bridge through adjacent low-lying fields.

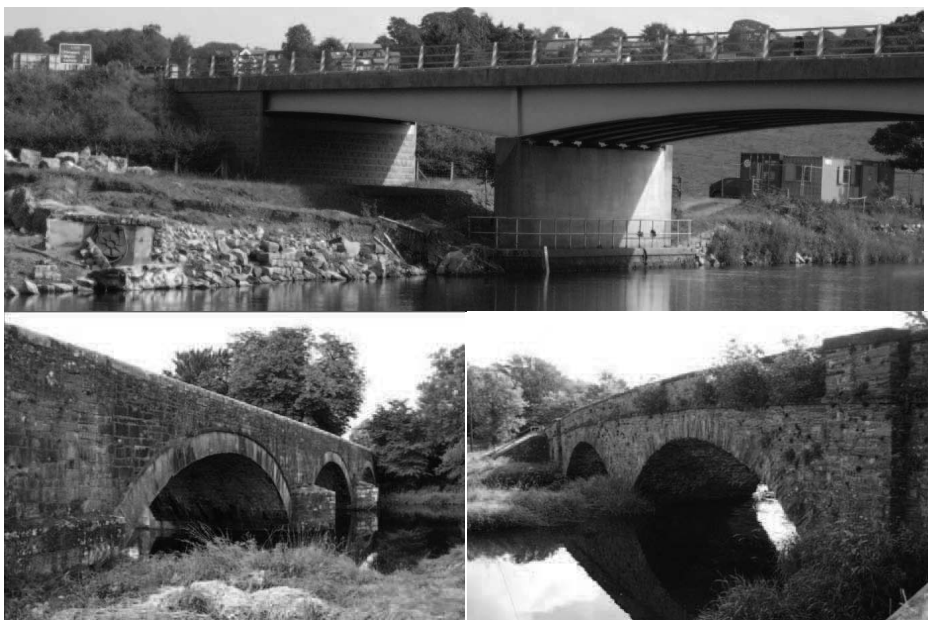


Figure 12 Top, Papcastle Bridge. Bottom left, Isel Bridge. Bottom right, Southwaite Bridge.

Current best practice in bridge management

Roads Liaison Group Management of Highway Structures – A Code of Practice
In an effort to introduce an overarching standard for Local Authority bridge management, in 2005 the Roads Liaison Group published the Management of Highway Structures – A Code of Practice(Roads Liaison Group, 2005). The Code of Practice is far reaching and aims to ensure that bridge stock is primarily safe for use but also fit for purpose. With regards to inspection, general and principal inspections are recommended. General Bridge Inspections take place at intervals not exceeding two years. General Bridge Inspections should be undertaken using the County Surveyors' Society bridge condition indicators. Principal Bridge Inspections take place at intervals not exceeding six years following the previous Principal Bridge Inspection. The inspection will be undertaken within touching distance of all accessible parts of the structure, including underwater. The six year spacing between Principal Bridge Inspections may be increased, provided that a detailed Risk Assessment has been undertaken for each structure. The Code of Practice is an excellent document, raising the importance of inspection and maintenance regimes.

County Surveyors' Society (CSS) Bridge Condition Indicators
The CSS inspection pro-forma(Highways Agency, 2007) is used to record bridge condition by assessing the extent and severity of defects to bridge elements. These are quantified for the extent, and severity. The pro-forma includes a section allowing maintenance or further inspection / testing work to be recorded. When completed comprehensively with an accompanying inspection report, the pro-forma is a good snapshot record of a bridges condition.

Ciria C551 – Manual on Scour at Bridges and Other Hydraulic Structures
Detailed guidance is offered by Ciria C551(CIRIA, 2002) on scour management at bridges. Definitive definitions of scour are offered. Guidance is given in undertaking scour assessments to allow resulting risk to be appropriately managed. A four stage strategy is recommended, including scour depth assessment for each structure in the influence of water for a 1-in-200 year return period. The guide details contraction scour, whereby the confining effect of a bridge, abutments and approach embankments accelerates flow through the bridge opening. Extensive scour results. Careful sizing of bridge openings is required in design. Protection measures are discussed, including scour protection aprons, gabion mattresses and guide banks.

Summary and conclusions

The flood event that occurred in Cumbria in November 2009 was the largest in living memory. The return period of the flood was around 1-in-500 years. The event was associated with atmospheric conditions which produced the highest ever recorded rainfall for the UK. This one flood event has significantly altered the river levels which are statistically calculated for each return period on each of the affected rivers.

It was possible to back-engineer and surmise the events that occurred prior to and during collapse of these seven bridges. Conclusions were formed from the limited physical and documentary evidence that was available given the elapsed time and clean-up works that had occurred between the event and our investigation. However the certainty of the conclusions would have been dramatically increased by the availability of better physical evidence and historic information.

A common source of damage to these bridges may have been mortar removal to masonry from scouring action. Mortar removal caused stone removal and local damage. In some cases, this may have triggered whole structure collapse. This incremental failure mechanism is not commonly considered in bridge assessment and could be worthy of further investigation. Mitigation of the risk of this form of damage could be reduced straightforwardly by effective repointing regimes.

This investigation highlighted the importance of good quality, accurate bridge records. As part of this record, timely, accurate and detailed inspection reports are vital to ascertain bridge condition. It is recommended that the Management of Highway Structures A Code of Practice provides an excellent basis not only for bridge management, but for recording information for a post-collapse investigation.

The scale of the flood was unprecedented and the damage sustained by these structures was a direct result of the scale. Best practice demonstrates that scour assessment is generally considered only for a 1-in-200 year event. Scour assessment and appropriate mitigation works would hence not have necessarily been adequate to have coped with this extreme flood. It was therefore considered unlikely that the damage could have been prevented by a more rigorous inspection, assessment or maintenance regime.

The tragic loss of life and disruption caused by these bridge closures and collapses demonstrates the crucial role bridges play in society. In spite of the unlikelihood of such a flood event, it is suggested that mitigation of the high social, economic and environmental costs may be worth planning for. Strategies could be established in advance with bridge stakeholders including highway operators, local communities and businesses and utility providers. At-risk infrastructure could be identified and early warning criteria and mitigation activities planned.

Acknowledgements

Cumbria County Council - bridge records. Environment Agency and JBA Consulting.

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Forensic Engineering: Informing the Future with Lessons from the Past
ISBN 978-0-7277-5822-4

ICE Publishing: All rights reserved
doi: 10.1680/feng.58224.061

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New York City 51st Street Tower Crane Collapse Investigation

James S. Cohen, PE, MASCE, Associate Principal, Arup, 499 Thornall St., Edison, NJ 08837, PH (732) 623-7480; email: james.cohen@arup.com

Abstract

On March 15, 2008, near the completion of the first round of climbing procedures of an external self-climbing tower crane at 303 East 51st Street, New York, New York, the polyester slings holding the 12,000 lb (5440 kg) collar at the 18th floor failed. The collar fell down the tower, severing the ties connecting the middle collar to the building. The two collars then proceeded to impact the lowest collar with significant vertical and horizontal deflections. The crane fell backwards, resulting in seven fatalities and significant property damage to several buildings. Ove Arup & Partners, PC (Arup) was hired by the New York City Department of Buildings (NYCDOB) to provide engineering and investigative services.

Arup investigated the collapse with regard to possible causes and contributions of the various components and activities surrounding the tower crane. The author was the lead investigator for this multi-disciplinary project in which multiple outside specialists were also involved. Following the investigation the author was the primary witness on behalf of the New York County District Attorney for the ensuing manslaughter trial and later administrative hearing in which the master rigger sought reinstatement of his operator's license.

Due to the breadth and detail of the investigation only a selected number of investigation activities, including those most pertinent to the collapse, will be presented.

Event

303 East 51st Street was a proposed 43-story concrete frame building designed by Garrett Gourlay Architect PLLC on behalf of Kennelly Development Company, L.C. As part of the construction method, an external self-climbing luffing tower crane, Model M440E, manufactured by Favelle Favco Cranes Pty. Ltd. and supplied by New York Crane, was used for construction of the upper levels.

The crane was not in service until the two lower collars, used to secure the crane to the building, had been installed. The crane and two lower collars were installed with the assistance of a mobile crane. These had been placed at the 3rd and 9th floor levels.

The first collar which was to be installed per the manufacturer's instructions was located at the 18th floor.

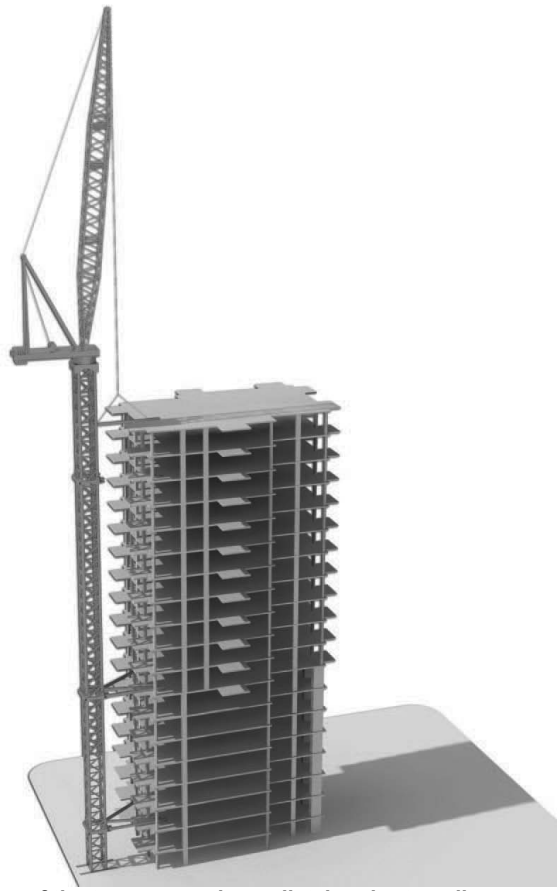


Figure 1 - Configuration of the tower crane immediately prior to collapse.

Each floor-to-tower connection consisted of a structural steel collar assembly surrounding the tower. These were installed at preset heights following jumping, i.e., extension, of the crane. The steel collars were assembled from two sections during which time they would be temporarily held vertically in place using eight chain pulls, four per collar section. The attachment points on the collar for the chain pulls were pre-designated by the crane manufacturer, for which attachment lugs were provided.

Three tie-beams were connected via pins to each completed collar. The far end of each tie-beam was welded to a steel base plate, which was in turn anchored by bolts to the reinforced concrete slab of the building, thus fixing the tie-beam to the building.

At the time of the collapse the team had just "jumped" or extended the height of the tower and was proceeding to install a new collar connection to the 18th floor. Four polyester web slings were being used to temporarily suspend the assembled collar at the time of the tower crane collapse, rather than the eight specified by the manufacturer.



Figure 2 - External climbing tower crane collar and beams configuration, similar to the 51st St. crane. Note: the tower crane is a different manufacturer.



Figure 3 - Chain fall and polyester sling assembly recovered from collapse site.

During positioning of the first tie-beam between the ears of the collar and prior to insertion of the fixing pin, the four slings failed. The collar fell downwards, impacting the 9th floor collar below destroying its connection to the building. Both collars fell and impacted the 3rd level collar. The 3rd level collar beams were significantly deformed due to this second impact with both significant horizontal and vertical deflections.

The tower foundation was a compression only construction, in that there was only frictional resistance to sliding. This choice of foundation had arisen due to the positioning of the tower directly over an open vault. A steel dunnage beam sat upon and spanned across the vault. The tower legs rested within pockets on the dunnage

beam. The tower overturned as its base slid towards the building structure to the north and the crane cab at the top fell to the south.

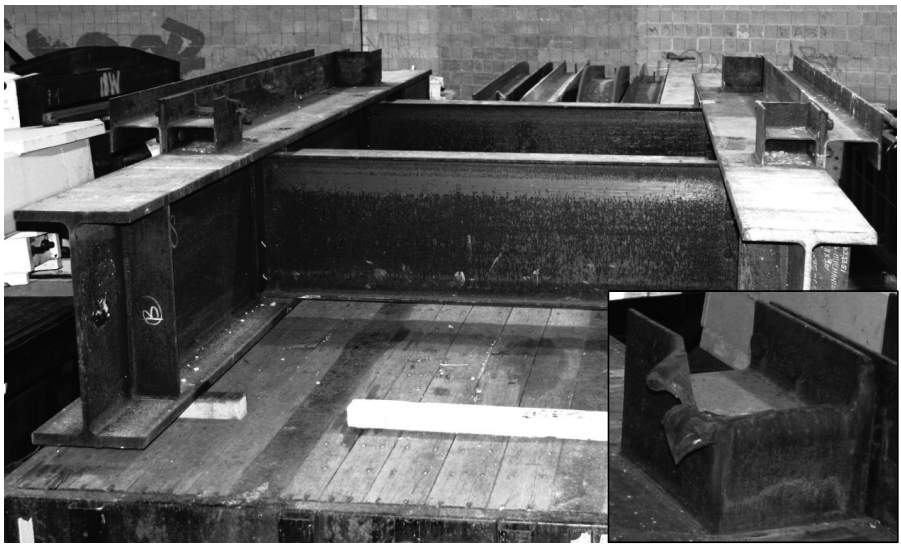


Figure 4 - Dunnage beam recovered from collapse site. Note the four pockets of the tower legs, one of which is notably damaged. Bent tie beams are visible in the background. The inset is a detail of the most damaged of the pockets.

Arup was originally involved in the investigation of the tower crane collapse on behalf of the NYCDOB. The investigation was undertaken without regard to any potential litigation, civil or criminal. Following completion of our investigation and report, as the lead investigator for the project the author was retained through Arup as the investigating engineer expert witness for the NYCDA for the ensuing criminal trial. Additionally, the author was again retained through Arup for the later administrative hearing concerning reinstatement of the master rigger's license.

Investigation Activities

As stated in our report to the NYCDOB, the work undertaken for our investigation included the following activities:

- Document review, including documents supplied by NYCDOB and the New York County District Attorney (DA) and the New York City Department of Investigation (DOI).
- Site visit to the collapse site.
- Site visits to view components of the tower crane and collar
- Assemblies which had been removed from the site prior to our engagement.
- Structural analysis of the tower crane, including both static analysis and non-linear dynamic analysis.

- Structural analysis of the sling system including non-linear dynamic behaviour.
- Materials and metallurgical testing.
- Witnessing of sling tests specified by OSHA, review of raw data from those tests and review of OSHA-related correspondences.
- A review of codes, standards and regulations of relevance to the collapse.
- A review of the permitting process associated with this crane.
- An independent peer review of the crane's support designs connecting the crane tower to the building and to the ground.
- An independent visual assessment, including microscopic examination, of the polyester slings used for supporting the collar assemblies recovered from the collapse site.
- Review of the report prepared by the U.S. Department of Labor, Occupational Safety and Health Administration (OSHA) dated September 2008. The report is titled "Investigation of the March 15, 2008 Fatal Tower Crane Collapse at 303 East 51st Street, New York, New York". The report was prepared by Mohammad Ayub, PE, Directorate of Construction at OSHA, Washington, DC.

Initial Theories and Approach

At the time the investigation commenced, we were unfamiliar with the parties involved, the construction means and methods, the building design or with any other aspect of the incident. Accordingly, there were many possibilities open as to potential causes. As the collapse itself represented an unusual event, any proximate cause(s) or combination of causes would therefore also be a rare occurrence.

Among the possibilities which were investigated included: seismic event, weather, tower design, building design, concrete and steel material properties, missing components, foundation design, tower component failure, collar component failure.

The simplest items were able to be discounted at an early stage. The US Geological Survey was the source for seismic information and the National Climactic Data Center was a reliable source for weather data. Visual observations ruled out any possibility of the tower (the primary vertical supporting steel lattice structure of the tower crane) being the cause of the failure. Review of the damage field generated by the falling crane precluded any cause resulting in lateral or torsional movements of the tower crane during collapse. Site observations indicate that there were no contributory causes arising from the building. Remaining items of interest were those involving the tower crane foundation, collars, crane operations, collars and tie-beam assemblies, and overall stability concerns.

Certain component parts were identified as being of special interest, with particular emphasis initially placed upon the base plate welds connecting the collar tie-beams to the building. Due to the breadth of potential initial causes for the collapse and range of possible contributing factors, attention was initially given to all facets of the tower crane, including design, fabrication, assembly and usage in the initial review of data,

as well as potential issues surrounding the cause and consequences of potential weld defects in the base plates.

Following the quick removal of certain possible causes of the collapse, the investigation therefore began with identification of failed components through direct observation of the available tower crane components and failure site with a view to establishing priorities for investigation, concentrating on the base plate welds and associated components. As additional information was obtained either through direct observation, document review or analysis, these priorities were adjusted.

Industry accepted standards were used as the basis for establishing accepted industry practice. Codes and standards selected for review included the New York City Building Code, selected sections of the American Society of Mechanical Engineers Standards B30 Safety Standard for Cableways, Cranes, Derricks, Hoists, Hooks, Jacks, and Slings, American Welding Society Structural Welding Code D1.1. Other publications used included those of the Web Sling and Tie Down Association, the Wire Rope Technical Board and certain military standards on wire rope.

Documentation related to the tower crane failure was collected and reviewed. Again, as the range of possible contributory factors to the collapse was initially vast, the review of documentation was similarly broad. As part of the documentation, a review of the permits prepared and submitted was carried out.

As hypotheses were proposed as possible explanations of the observed phenomena, it became necessary to formulate experimental studies via computer modeling to test these hypotheses. Results of these analyses needed to be consistent with both the observed phenomena and documentation as well as consistent with related studies underway simultaneously in order to dependably determine causes of failure. Both prioritized areas of interest and failure hypotheses were adjusted as analysis results and other additional information were obtained.

Computer analyses therefore included both macroscopic studies, encompassing the tower crane structure in its entirety and localized studies involving detailed analyses of smaller components. The global approach was used in the investigations of general stability to the tower and in the development of design criteria for the various components which supported the tower. These were initially limited to static analyses of the tower crane. As the investigation progressed plastic deformation / large displacement / dynamic computer analyses were implemented. Due to the extremely long durations which such computer runs required these were necessarily selective rather than all encompassing. Smaller models investigating particular components, such as welds, tie-beams and polyester web slings were more easily manipulated and adjusted as additional information inevitably arrived.

Theories that encompassed wider domains were then developed to bind these hypotheses surrounding the smaller events into a coherent structure. This in turn assisted in reaching the final conclusions.

Within the context of this investigation was the need to maintain and clearly be able to demonstrate objectivity. This included sensitivity analyses of unknown or ill-defined parameters to establish a lack of bias in the selection of data. This practice also allowed for a measure of the reliability of these data to be established. Parameters which were allowed to vary included the initial impact velocity of the 18th floor level collar on the 9th floor level collar, friction coefficients between the dunnage beam and foundation, polyester web sling tensions and base plate fillet weld ductility.

Expertise was sought to supplement knowledge and experience in certain specialized areas. These areas included metallurgy, polyester materials and tower crane practice.

Each element of the investigation, including but not limited to dynamic analysis modeling, development of test protocols, and document review was subjected to peer review to confirm its validity.

Crane On-Site Design Review

This review included the design of all components not supplied by the tower manufacturer, including the foundation dunnage beam, tie-beams and tie-beam connections to the tower crane collars and the building. The crane engineer's design submission was found, for the most part, to be in accordance with industry "best practice", save for some noteworthy departures. More specifically:

- Adequate documentation of the design (i.e., drawings and calculations) was provided;
- The tower crane make/model that was selected (i.e., the M440E) was appropriate for the job;
- The support and erection scheme for the tower crane (i.e., collars, horizontal ties, base support conditions, etc.) were appropriate, however the magnitude of the height of free standing tower in the temporary condition (prior to installation of additional tie-ins) was not justified in the calculations provided;
- The calculations address industry standard load-cases/loading scenarios and are generally accurate with the major exception that slewing moments do not appear to have been considered for the in-operation mode. However, the reviewer determined that the design of the tower's attachments had sufficient robustness to resist these slewing moments.
- The design drawings clearly show the details of welds required to connect the tie-beams to the steel base plates. Furthermore, these welds are sufficient as designed to withstand the anticipated lateral loads, both those calculated by the crane engineer (Stroh Engineering) and those calculated by the reviewer. It is considered beyond the expectations of "best practice" to design these welds for impact loads of a magnitude similar to that which is believed to have occurred at the site the day of the collapse (i.e. a 12,000 lb. / 5440 kg weight falling a distance of 100 feet / 30.5 m).

Nonlinear Static Analysis

Static structural modeling of the crane tower was undertaken to provide a better understanding of the overall static stability of the tower when subject to various forces and moments under different lateral support conditions.

These analyses demonstrated that the tower was stable against elastic buckling between collars and above the uppermost collar. This was consistent with field observations that showed no damage to the tower segments themselves other than a connection failure near the top following impact during collapse.

The analyses also demonstrated that, although the legs nearest the building would start to lift up, they would not slide. As such, an overturning failure would not be predicted. This was inconsistent with observations. However, it was also clear that the static analysis would be unable to capture key aspects of the event, including, in particular, the dynamic effects of the linear and angular momentum of the crane.

Dynamic Analysis: Collar Integrity and Tower Stability

Several non-linear dynamic finite element analyses were completed to simulate the behavior of the 9th floor collar assembly when it was impacted by the 18th floor collar. Although the model demonstrated that the impact would have caused an almost immediate failure in the base plate welds and, additionally, that the tie-beams would detach from the 9th floor slab, the collars would nevertheless slow down until vertical equilibrium was reached.

Post collar impact studies were therefore undertaken. The global tower stability analysis used an enhanced model of the 9th floor collar impact analysis to capture the global behavior post collar impact. The foundation was modified to include contact between the base of the tower and the dunnage steel.

After the 9th floor connection is compromised, the stability of the tower was found to be dependent on whether or not the dunnage steel displaced. Analysis demonstrated that the horizontal force below the dunnage was great enough to overcome the frictional resistance due to the self-weight of the tower; hence the tower was predicted to fall. This was consistent with the observed behavior of the dunnage foundation.

Polyester Slings

Polyester slings were used as part of the chain fall assembly to suspend the collar at the 18th floor immediately prior to collapse. As part of the collapse investigation, material analysis of the slings, dynamic structural analysis, review of OSHA test results and review of pertinent codes and standards related to the use of the polyester slings were conducted.

Polyester web slings were used both for lifting the collars into place as well as for securing them temporarily in a vertical position while the two collar halves were bolted together and positioned to their final vertical position using the chain falls. The

slings used for securing the collars in a vertical position on the tower were manufactured from two-ply, two-inch wide polyester webbing. Three of the slings used on the tower were manufactured by one manufacturer, the fourth sling by another. As part of the manufacture of the slings from the polyester webbing, a load bearing splice is typically formed at the base of the eye, at which location there may be a 3-ply construction in an otherwise 2-ply sling.

There was a failure to follow manufacturer's explicit instructions with regard to the selection of lifting and chain block points. As a result, the collar was suspended from the tower by choking the slings around the legs of the crane mast and, additionally, the requisite number of support points was not used. A photograph, coincidentally taken by an independent professional photographer just one hour before the tower crane collapse, showed the polyester slings choke hitched around the tower legs as well as improper selection of the lifting points. The bearing of the sling within one of the "V" grooves was also evident.

Observations of the sling parts retained by OSHA revealed that the polyester web slings were exposed to frictional contact at their bearing points arising from the unprotected edges of the tower legs around which they were choke hitched. Specifically, the melt of the polyester web slings fibers at the failure lines is strongly indicative of an abrasion melting of the fibers. This was corroborated by the test results carried out under OSHA's directions. This method of attaching the slings violated industry accepted standard practice in addition to the codes and standards which were reviewed.

There was no indication that the loads being applied to the polyester web or wire rope slings were known with any degree of certainty. As indicated below, the high variability in stretch would render the ability to accurately assess the tension under load without a load indicating device impossible.

There was evidence that at least one sling, was previously exposed to long periods of ultraviolet radiation sufficient to bleach both the outer fibers of the sling and the manufacturer's label. This sling also showed evidence of prior usage, including damage to the sling. This observation was corroborated by OSHA.

During the test program carried out under the direction of OSHA, all slings failed significantly below their anticipated actual capacity. Per the reviewed standards, the rated capacity of a polyester web sling, regardless of other considerations including angle of choke and angle of loading, should incorporate a factor of safety of 5. When in a choke hitch there is an additional minimum 20% reduction in rated capacity. Therefore, as the slings tested had rated capacities of between 5,000 (2270 kg) and 5,100 lb (2310 kg) in a choker configuration, actual load capacities should have been approximately 25,500 (11570 kg) and 25,000 lb, (11340 kg) respectively, in the choker configuration. Actual test results showed breaking strengths of the new undamaged slings as low as 7102 lb. (3220 kg).

In addition to the dynamic analyses described above, a nonlinear study of the polyester sling collar support system was also performed. The non-linear analyses examined the dynamic amplification effects of one sling failing and the load being redistributed into the remaining three slings. Both even and uneven load distribution scenarios were studied.

Of the analyses performed two of the three analyses based on an uneven load distribution predicted complete failure of the sling support system as the result of the failure of one sling. Variations in results arose from application of the differing OSHA test data in each respective analysis.

Although the specific condition of the slings on site including their strength, ductility and load distribution is not known, the analyses demonstrate that it is indeed possible that the failure of one sling would precipitate the failure of the remaining three. More importantly, the analyses demonstrated the need for careful control of these conditions when using synthetic web slings.

Summary Of Investigation Results

Based on our investigation, we identified the following relevant factors in the collapse of the tower crane:

1. The collapse of the tower crane was initiated when the polyester web slings supporting a steel collar at the 18th floor level failed, allowing the collar to fall.
2. Improper usage of the polyester web slings resulted in the failure of the slings. Specifically:
    a. The number of supports provided by the slings did not meet the crane manufacturer's requirements – only half of the required eight supports were used.
    b. The positioning of the slings on the collar as installed was inconsistent with the crane manufacturer's instructions.
    c. The method of attaching the slings to the tower was not in accordance with accepted industry practice and standards.
    d. One of the slings had been previously used, rendering its condition such that its load capacity would have been reduced.
3. As demonstrated by structural analysis, the failure of one sling in the arrangement as installed. could lead to a sudden and uneven redistribution of load to the remaining three slings. This effect could be sufficient to lead to a sequential failure of the remaining slings, thus allowing the collar to fall from Level 18.
4. As demonstrated by structural analysis, the unanticipated loads arising from the dynamics of the collar falling from Level 18 caused failure of the collar connection at Level 9, which in turn allowed slippage of the dunnage steel at the base of the crane resulting in the overturning of the tower.

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SECTION 2:

FORENSIC INVESTIGATION CASE STUDIES – BRIDGES

Forensic Engineering: Informing the Future with Lessons from the Past
ISBN 978-0-7277-5822-4

ICE Publishing: All rights reserved
doi: 10.1680/feng.58224.075

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Railway Bridge Failure during Flood in the UK and Ireland: Learning from the Past

Jeremy Benn FREng MA MSc CEng CWEM FICE FCIWEM MASCE MIEI
Chief Executive, JBA Group, Broughton Hall, Skipton, BD23 3AE, UK

Abstract

Review of past railway structure failures due to hydraulic action (usually in flood) provides useful evidence to inform design and inspection regimes.

There have been 15 fatalities and perhaps 4-5 times that number of injuries which can be attributed to structure failure during flooding on the GB railway system since the 1840s. The resulting economic damage is estimated to be at least £287 million (2004 figures).

Bridge failure due to flooding is most commonly associated with 'extreme' but not necessarily 'very rare' floods; the average event rarity associated with catastrophic failure is 1 in 160 years, but the range of 200 - 250 year return period includes most flood related failures. The high incidence of summer/early autumn flood events leading to failure mainly as a result of localised high intensity rainfall on small catchments is of particular note. These events are likely to be at a time of reduced vigilance for flood management.

Undermining of abutments and piers by scour is the most common form of failure of bridges and these may not be adequately predicted by existing assessment procedures. The remaining failures can be attributed to six other failure sequences which are currently not adequately addressed within the existing procedures. Of these six, debris collection resulting in the exacerbation of local scour and the location of structures in rapidly responding catchments are considered the most significant.

Based on the results of this research and practice elsewhere it is recommended that the design flood for scour assessment and scour protection design should be based on a 200-year return period flood event. A higher value (of 1,000-years) may be appropriate for structures with a particular high consequence of failure such as high speed lines, critical transport routes, where no adequate warning can be provided, or where flood waters could impound behind embankments and be released suddenly.

The methods of analysis developed for this study have been used to investigate the causes of two recent water-related bridge failures: the Lower Ashenbottom viaduct in Lancashire (2004), and Malahide Viaduct (2009). In both cases, serious loss of life and injury were only narrowly avoided. This paper describes how it was possible to establish the underlying causes of failure and the collapse mechanisms.

Introduction

Railways need to cross watercourses and floodplains and do so by a variety of engineered structures including embankments, viaducts, bridges and culverts. On the operational railway in GB there are over 10,000 such structures.

Such structures are vulnerable to hydraulic action, such as scour which can undermine foundations, additional loading causing failure of decks and voussoirs, and the shifting of river channels undermining approach embankments and retaining walls. Most but not all such failures occur during flood events, although some are the result of progressive failure.

In addition to bridge/culvert failures there have been many embankment/cutting slips and pipe/drain failures as a result of heavy rain and flooding. These failures appear to be more widely spread geographically during times of severe bad weather but there appears to be no incidence of fatalities resulting from them.

The first recorded casualty from a flood-related failure occurred in the early years of 'Railway Mania' when the driver of a train was drowned when a bridge collapsed at Tonbridge in 1846. In total there are 15 known fatalities from flood related collapse, the worst single incidents being the June 1914 incident at Carrbridge in Scotland and the Glanrhyd disaster in October 1987 resulting in 5 and 4 deaths respectively and dozens of injuries.

In terms of financial consequences, an annual loss of at least £1 million a year can be suggested on the basis of a statistical average of at least one bridge failure every 2.5 years.

While this paper concerns railway infrastructure, it should be noted that road structure failure is probably as common – with 20 bridges damaged or collapsing in the November 2010 floods in Cumbria and resulting in the death of an on-duty police officer.

Rail Safety and Standards Board Research

In 2003, the Rail Safety and Standards Board commissioned research on the incidence and causes of catastrophic failure of GB railway infrastructure due to water (hydraulic) action. The work was completed in 2004 and consisted of:

- Creation of a database of 129 historical scour or flood-related failures.
- A review of the information in the Network Rail database of bridges/culverts at risk from scour and flood damage.
- A forensic assessment of the causes of failure of railway structures due to water action.

For 69 of the 129 failures it was possible to obtain enough information to make a judgement on the courses of the failure. Sources used included Rail Accident Investigation Unit (Ireland) / Rail Accident Investigation Board (GB) / Board of Trade reports, information in Network Rail and National Railway Museum archives, and rainfall records. Computer modelling was used to recreate river flow conditions and to undertake structural and hydraulic analysis.

The information gathered in the RSSB research is the basis of this paper together with data from three recent railway structure failures due to hydraulic action: at Lower Ashenbottom (2002), Malahide (2009) and Feltham (2009).

Causes of Failure

Table 1 summaries the dominant cause of failure of railway structures due to scour from the RSSB study.

Table 1: Failure classification of 69 water-related failures to railway structures (1845 – 2012)

(Source: RSSB, 2004 and unpublished data)

Dominant Cause of Failure	Number
Pier scour	23
Abutment scour	14
Floating debris	10
Other debris	10
Embankment scour	8
Channel modification (e.g. dredging)	4

The most common form of failure of bridges is undermining of abutments or piers by scour. While these have occurred in all types of rivers, a significant number of these failures were in steep gravel bed rivers which suggests that the additional energy in such environments leads to a particularly aggressive environment and where the bed material is erodible.

A feature of the failures due to abutment scour are the number of single span structures on smaller watercourse crossings. It was common for the bridge to be built alongside the river and then it was diverted through the new structure and the old river channel infilled. This has created an artificial double bend in the river with the outside of the bends being more susceptible to scour (Reed, 2004).

The incidents of embankment scour are dominated by failure of embankments on wide floodplains where flow can pond (see also Whitbread, et.al., 2000).

Accommodation bridges accounted for 10% of the total bridge failures. These structures are generally not currently assessed for flood or scour risk unless they are known to be in the floodplain.

There were four incidents of flood failure where channel modification was a dominating factor:

- Ness Railway Bridge (February 1989) – possibly exacerbated by navigation dredging downstream.
- Colne Brook, Wraysbury (May 1988) – exacerbated by gravel abstraction.
- Wooler Water (Haugh Head) (August 1948) – greatly exacerbated by gravel dredging in the river channel reducing river bank stability.
- River Medway at Penshurst (Jan 1846) – gravel extraction in the floodplain provided additional flow routes.

Incidents of failure where floating debris contributed to the failure included:

- Lower Ashenbottom Viaduct (June 2002)

- Bridge 3/48, Feltham, (November 2009)

There are several incidents, however, where multiple structures have failed during a single flood episode.

Effectiveness of existing assessment procedures

In order to test the adequacy of existing scour assessment procedures, the 69 structures for which detailed information was available were analysed using data that would have been available pre-failure. Two methods were used: Assessment of Scour at Highway Bridges (Highways Agency, 2006) and Hydraulic aspects of bridges: Assessment of the risk of scour at railway structures (Bettess, 1993).

Both methods performed similarly. In 50 of the 69 cases, they provided a 'high priority' score suggesting that the structure was at particular risk of failure. However, in only 20 cases was the failure mechanism correctly identified in the assessment.

Particular findings from this exercise were that:

- There is insufficient emphasis on the role debris can play in scour failure.
- Existing assessment procedures appear to be particularly poor for culverts and accommodation bridges.
- The risk or priority score is very dependent on the foundation (i.e. depth and type – details of which are often subject to considerable uncertainty).

The influence of flooding

The existence of high river levels is a dominant although not entirely exclusive factor in failure due to hydraulic action. The very rare floods appear to be so destructive that it is hard to imagine any reasonably economic bridge protection measure that would withstand the associated forces, other than to ensure that the bridge/culvert opening is wide enough to accommodate these floods.

The majority of the failures are from single geographical incidents. Where multiple failures have occurred they have been geographically close and in the same river valley. For example, the Moray floods of September 1915 destroyed 16 rail bridges and culverts belonging to the Highland Railway in the Spey and Findhorn valleys. The August 1948 flood event in the Scottish Borders and the Cheviots can probably be ranked as the 'worst' from the viewpoint of disruption to railway operations, with 9 bridges failing – 7 of them being on the East Coast Mainline between Grantshouse and Reston.

There is no readily apparent geographical pattern to the incidence of the failures. However, there are 5 identifiable 'clusters' where failures have occurred in the same river catchment for different years:

- The valley of the Eye Water between Grantshouse and Eyemouth (multiple failures in both September 1846 and August 1948).
- The Esk Valley between Glaisdale and Whitby (failures in August 1866, July 1930 and September 1931).
- The Upper Spey valley at Carrbridge (failures in June 1914 and July 1923).

- Upper Severn valley upstream of Newtown in Wales (2 failures in February 1868 and one in June 1936).
- Moray in Scotland (Spey and Findhorn valleys) – 16 bridge/culvert failures in September 1915, and previous unspecified events.

Figure 1 shows the seasonal distribution of major catastrophic events. The high incidence of summer/early autumn flood events leading to failure mainly as localised high intensity rainfall on small catchments is also of particular note. These events are likely to be at a time of reduced vigilance for flood management and a particular risk must be for the washout of a relatively small bridge/ culvert just before a train arrives as a result of a localised thunderstorm on a small steep catchment.

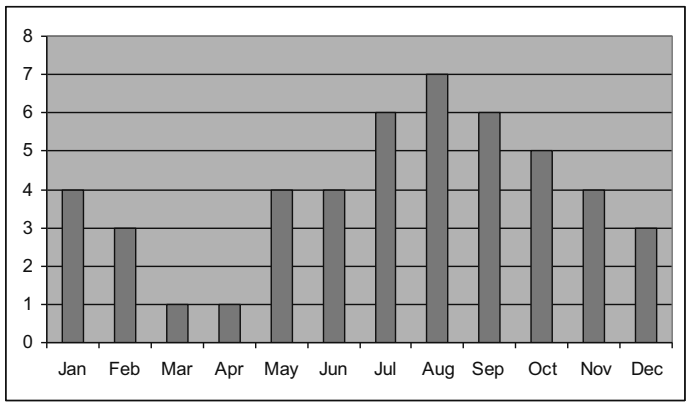


Figure 1: Incidence of failures of railway structures due to hydraulic action (1846-2004)

Source: RSSB, 2004

Design Floods/ Acceptable Risk

Bridge failure due to flooding is most commonly associated with 'extreme' but not necessarily 'very rare' floods. The average event rarity for the failures analysed in the RSSB dataset was about 160 years. A 200 to 250-year return period flood therefore appears to be the sort of event, which includes most of the failures that are purely flood related.

However, the estimation of return periods for past events can be very uncertain. For assessment purposes, a design flood of 200 - 250 years therefore appears appropriate. For planning purposes, a return period of 100 to 200 years for flooding is now widely used as an acceptable level for 'low to medium risk' developments such as housing and a 1,000 year for high risk infrastructure such as hospitals (see National Planning Policy Framework for England, Technical Advice Note 15 for Wales and Scottish Planning Policy for Scotland). For structures with a particular high impact of failure such as high speed lines, critical transport links, high value structures, locations where no warning can be provided, or where flood waters could impound behind embankments and be released suddenly, a 1000-year return period may be more appropriate.

However, for new or replacement structures, a slightly different approach can be taken. The design life of a structure is commonly between 30 and 150 years, depending on the size and

nature of structure being designed. A structure would normally be designed to withstand a flood flow of a given magnitude, the design flood, which has a certain probability of occurring. The probability is normally expressed in terms of return period, where a particular flood flow is likely to occur on average once in N years. It is not normally appropriate - although in practice it is often done - to use the design life of a structure as the return period of the design flood. The danger of doing so is illustrated by considering a structure with a design life of 100 years, which is designed for a 100-year return period flood. This would have a 63% chance of experiencing a flood of that magnitude or greater over its life.

May et.al., 2002 recommended a more rigorous approach to deciding what probability of the design flood being exceeded over the structure's design life is judged to be acceptable. The probability of exceedance, Pr, the design life, L_y, and return period, N, are connected by the following formula:

$$Pr = 1 - (1 - (1 / N))^{L_y}$$

from which it can be shown that the relationship between the return period and the design life varies with the acceptable degree of risk as follows:

$Pr = 0.50$ $N = 1.5\ L_y$ (approx.)

$Pr = 0.20$ $N = 4.5\ L_y$ (approx.)

$Pr = 0.10$ $N = 9.5\ L_y$ (approx.)

Thus, for example, if a 20% chance (Pr = 0.2) of the design flood being exceeded during the 50-year design life of a structure is judged acceptable, then a design flood with a return period of N =50 x 4.5 = 225 years should be used. Due to safety factors included in the foundation design, a small exceedance of the design flood would not normally be expected to result in the failure of a bridge; this can make it more difficult to judge what constitutes an appropriate probability of exceedance for use in design.

In the UK, a flood return period, N, of 100 to 120 years is often used for the design of new structures, although a range from about 30 years for minor rural roads to 150 years or more for motorway or trunk roads is also typical. In the United States, bridges are generally designed to withstand a 100-year flood with a load factor of about 1.5 to 2.0 against collapse; a separate check is also made to ensure that the bridge will survive what is termed a ‘superflood', with the load factor for the structure remaining greater than 1.0. The superflood is normally taken to be the 500-year return period event. The stability of foundations at existing bridges is also checked for the superflood. The risk equation indicates that, for a typical bridge with a 100-year design life, US practice can be considered as equivalent to accepting an 18% probability that the load limit of the bridge might be exceeded during this period.

Ideally, ‘acceptable’ levels of risk must also take into account the impact of possible damage or failure. For instance the acceptable probability of an event is different if there is risk to life compared to say, the risk of damage to a structure requiring remedial work but which does not impair its load carrying capacity. This consideration of consequence is often considered in other industries and countries (Swiss Federal Roads Authority, 2002).

In practice there are many uncertainties in flood estimation and risk analysis, and despite the impression given by the various tables and formulae, the determination of suitable design flood events is still a matter of considerable judgement. While scour depths and hydraulic forces do generally increase with return period, the incremental increase for reasonably severe and severe floods is often small. This is due to a ‘flattening of the flood flow versus

depth/velocity' curve and hence scour depth once out-of-bank flow occurs. Hence the 'outcome' in terms of required mitigation measures for a 100, 200 or even a 500 year event is usually the same.

In the context of linear assets such as railways, the overall risk profile also needs to be considered at the route level. When a flood occurs, several bridges on the same line may damaged or destroyed. Alternatively several lines may be impacted where flooding is widespread or where multiple lines cross through the same catchment. In both cases the spatial dimension of the flood risk and asset location needs to be taken into account in the risk assessment and management plan, and can in some cases lead to fewer management responses. For instance once a route is closed, the need to visit all bridges of concern may become less urgent.

Predicting Floods

Floods with the capability to damage railway structures can be expected with some regularity. However, the prediction of the floods that could cause damage is extremely difficult for the following reasons:

- The science of flood forecasting is still far from exact and has a large degree of uncertainty. Rainfall forecasting is unlikely in the short or medium term to provide a means of taking effective mitigation measures as it is currently available at too large a spatial scale. It is also not possible to distinguish between rain that will cause damaging floods and rain that will lead to river levels that will not pose a risk to railway infrastructure. Prediction of flow or water level is more promising but this will require installation of suitable monitoring equipment at vulnerable structures and within the catchments upstream.
- The risk of damage to a structure depends on many local factors and not just the occurrence of a flood. Amongst the most important of these are debris build up and modifications to the river channel in the vicinity of the structure.
- Many of the most potentially vulnerable structures lie in small, upland catchments where extreme rainfall and floods are particularly difficult to forecast with sufficient lead time to allow any kind of 'real-time' response action such as line closures.

For small catchments (with areas less than 25 km^2) or where the time to peak of flooding is less than 4-6 hours then physical measures of scour and flood protection must be given priority. Use of flood forecasting as part of flood action planning will be most reliable in larger river catchments (where in much of the UK a service is provided by the Environment Agency or the Scottish Environmental Protection Agency).

Case Studies

To illustrate the techniques used for forensic examination of flood-related failures, two case studies are presented:

Case Study 1 - Lower Ashenbottom Viaduct

Lower Ashenbottom Viaduct is located on the preserved East Lancashire Railway approximately 2.5 km south of Rawtenstall town centre in Lancashire. Sometime between Friday 14th June 2002 and early afternoon on Saturday 15th June 2002, the central pier of the viaduct carrying the railway over the River Irwell partially collapsed. Luckily no train services were scheduled that day and the failure was noticed and the line closed before any trains crossed the bridge (Photographs 1 & 2).

Calculations undertaken by JBA Consulting (unpublished) indicate that scour resulting from the flood flow of June 2002 for the pier without any debris would not have exceeded the estimated foundation depth. Also, the estimated severity of the June 2002 flood (25-50 year return period) does not suggest that flow conditions were worse than those during other floods which the Viaduct has previously survived without serious damage. No other structure on the River Irwell is known to have failed or suffered damage as a result of the June 2002 flood.

Scour depth calculations using gauged river flows demonstrated that the flood alone was insufficient to undermine the central pier. Another factor must have contributed and exacerbated the scour to cause undermining. An adequate factor of safety against overturning and sliding of the pier even under debris loading appears to have been available. Flood levels did not reach the bridge deck and hence any possibility of deck flotation and loss of support to the pier from the dead load can be discounted.

Further analysis showed that debris collection on the central pier (especially a large tree trunk) would have doubled scour depths through creating additional turbulence and enhanced local flow velocities.



Photograph 1:

Lower Ashenbottom Viaduct - central river pier after the flood of June 2002

(photograph courtesy of Bury Metropolitan Council Engineering Services)



Photograph 2:

Lower Ashenbottom Viaduct - reconstructed central river pier

(photograph courtesy of JBA Consulting)

Case Study 2 - Malahide Viaduct

On 21st August 2009 a section of the Malahide Viaduct on the main Dublin to Belfast railway line collapsed shortly after an Iarnród Éireann passenger service passed.

The incident triggered an investigation into the reasons for collapse and a study into the design and reinstatement of the viaduct. A number of reports relating to the collapse (RAIU, 2010) and the design (Holloway, 1968) and historic information (Doyle, 2000) are available.

At the time of the accident, the Malahide Viaduct piers were formed on a grouted rock armour weir, with stones intermittently placed along this weir to maintain its profile. Extensive post-incident survey, computer model, and physical model studies were undertaken to help establish the causes of the failure.

The immediate cause of the collapse of Pier 4 was as a result of the undermining of the weir that surrounds and supports Pier 4 through the action of scouring. This was the result of a combination of factors:

- An inspection carried out on the Malahide Viaduct three days before the accident did not identify the scouring defects visible at the time.
- A scour inspection undertaken in 2006 did not identify the Malahide Viaduct as a high-risk structure to the effects of scouring.
- Iarnród Éireann's likely failure to take any action after an independent inspection carried out on the Malahide Viaduct in 1997 identified that scouring had started at the base of Pier 4 and that the rock armour weir was "too light for the job".
- The historic maintenance regime for the scour protection along the Malahide Viaduct appears to have ceased in 1996, resulting in the deterioration of the defence.
- The weather and sea conditions on the day of collapse were fairly normal. The cause of collapse was triggered by accumulative scouring action over a long time period, rather than from an isolated river flood or a surge tide, although there was a possible contribution of dynamic loading from passing trains to the loss of undermined pier masonry.

The key lessons to be drawn from this incident were the need for (a) an understanding of the function of past engineering works to counter hydraulic action and (b) for regular inspection and monitoring.

Conclusions

Comparison of the fatality and injury statistics from hydraulic failure with those across the whole of the rail industry (e.g. Evans, 2003) show that water action related accidents do not rank as the most significant. However, the railway industry should follow best practice, which in turn should be based on forensic analysis of past incidents. While it may not be economically sensible to prevent all flood failures, it is important that the risk to railway users and staff is mitigated where possible. It is therefore timely that the widespread dislocation experienced in rail operations through the autumn/winter 2000 floods in England and Wales, summer 2007 in England and summer 2003 in Scotland, and the results of this research, encourage the railway industry to ensure flood risks are acknowledged and managed.

Forensic analysis of past failures shows that existing assessment procedures can be improved upon at little additional cost. Also the collection and recording of information on damage incurred and water levels reached after a flood will be valuable in developing effective management procedures.

Programmes of checks, inspection and maintenance are a critical concern, regardless of the 'design' standard of flood resilience adopted. The N-year flood at a particular site is as likely to happen next year as in any other year, and climate change appears more likely to increase than decrease risks.

Because of the large number of river crossings on the GB railway network, and the partial spatial dependence in extreme rainfalls and flooding, the annual risk probability of a N-year flood somewhere on the GB railway network is many times greater than 1/N. When an extreme event occurs, it is quite likely to affect multiple structures and could affect several railway links.

The risk to railways is not that a big flood will occur but that a big flood will cause bridges, culverts and embankments to fail. In analogy to other areas of river flood defence, there are three main issues:

- To design and maintain structures to a high standard (and higher than currently used), so that failures are tolerably infrequent;
- To take precautionary action – to reduce impacts – when a major flood threatens.
- When a flood occurs, several bridges in the same area may be damaged or destroyed, and hence the safety issue from the point of view of a route rather than a single asset.

A feature of bridge failure due to scour is that appreciable pier or abutment movement often precedes collapse. This suggests scope to consider a generic real-time monitoring system to detect abnormal pier/abutment movements.

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Forensic Engineering: Informing the Future with Lessons from the Past
ISBN 978-0-7277-5822-4

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doi: 10.1680/feng.58224.085

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Tamar Bridge - Investigation of Surfacing Defects, Design and Specification

Iswandaru Widyatmoko CEng MCIHT FIAT, Associate, URS, Nottingham, UK
Richard Elliott MIAT, Technical Director, URS, Nottingham, UK

Background

The Tamar Road Bridge was constructed in 1961, alongside and immediately north of Brunel's Railway Bridge across the river Tamar. At the turn of the century, a programme of major strengthening and widening was undertaken, including the replacement of the original concrete deck with steel and the addition of two steel decked single lane cantilevers on either side. The steel decks were surfaced with a mastic asphalt surfacing system in 2001, comprising the key components/processes as shown in Table 1.

Table 1: Bridge Surfacing Structure

Thickness	Bridge Surfacing Structure
38 mm	10mm nominal aggregate size Mastic Asphalt incorporating 50% TLA (Trinidad Lake Asphalt) plus 14mm precoated chippings
1.5 – 3 mm	Rubberised Bitumen Underlay (RBU) comprising 25-30% of 80/120pen bitumen and 70-75% of limestone filler, with an addition of 1.3-1.6% (by weight of bitumen/filler mastic) of Pulvatex
-	Adhesive primer: Aquamac Super Penetration (north/south cantilevers and main deck south) or Bostik 9225 (rest of the structure)
13 mm*	Steel deck

Note: *The cantilever decks have variable thickness but are usually 12mm thick, except at the following locations: (a) Both approach spans and dog leg around towers, where the deck is 15mm thick and; (b) Dog leg around Plymouth main tower, where the deck is between 15 and 18mm thick.

An overview of the bridge deck and the trafficking arrangement is illustrated in Figure 1. There are four trafficked lanes in the main bridge decks and North Cantilever (NC); the South Cantilever (SC) is used as a pedestrian path (footway) and cycle track (cycleway). This traffic arrangement indicates that the NC and main deck Eastbound (EB) carriageways could potentially be overloaded by slow moving and/or standing vehicles approaching the Toll Booths, whilst traffic on the main deck Centre (CT) and Westbound (WB) carriageways can be expected to move faster and more freely. The main deck centre lane accommodates 2 way traffic flow, depending upon the nature of the flow and/or the time of day.

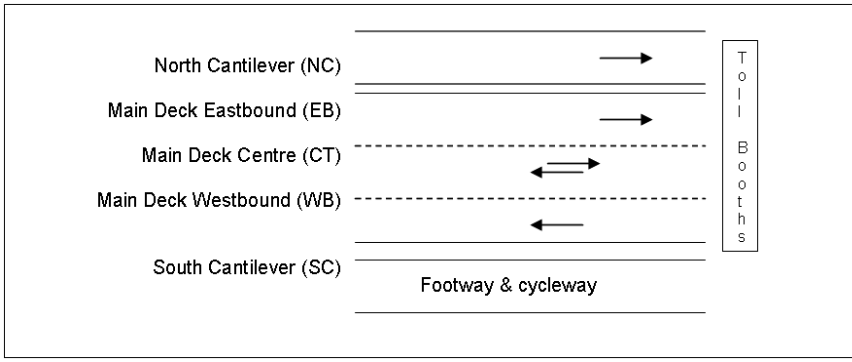


Figure 1: Layout of Traffic Arrangement

Methodology

A thorough investigation was carried out involving site inspection, reviewing available data, laboratory testing, designing surfacing material and recommending remedial actions. Samples were recovered from site for detailed laboratory assessment, including stiffness, tensile strength and shear modulus, of the existing mastic asphalt surfacing and the waterproofing layer. These suites of testing provided a sufficiently in-depth fingerprint of the residual mechanical properties of the recovered materials.

Finite Element Models (FEMs) were subsequently developed specifically for this bridge deck structure to analyse movements of the deck, including the cantilever and the main decks, for a range of traffic levels and speeds, considering the mechanical properties determined from the laboratory tests. The analysis included prediction of the remaining life of the existing mastic asphalt and the projected life of the recommended replacement waterproofing and surfacing material. The effect of super single wheel loading on various proposed materials was also assessed, combined with advice on the treatment interfaces between separate rips of the mastic asphalt surfacing system.

Based on the outcomes from the above, alternative material options and improved surfacing designs were recommended for maintenance and remedial works.

Site Inspection

The cantilevers were surfaced in February/March 2001, the main deck eastbound (EB) in June/July 2001 and the main deck centre and westbound (WB) in the period August to October 2001. The work described in this paper was carried out between 2007 and 2009.

Possible original construction and early-life deterioration defects (including poor profile, variations in chipping density and shrinkage movement at kerbs) were noted, and there was evidence that some material had been replaced at the time. However the defect that precipitated the current investigation was considered to be of more recent origin and comprised multiple cracking in the EB lane approaching the Toll Booths. At a site inspection in November 2007, the defects were broadly categorised into those requiring treatment either

relatively immediately or in the near future, and those that were inherent features of the system in place and could (or would have to be) tolerated for the design life of the system:

A) "Structural" defects requiring immediate or future treatment:
- Multiple cracking in EB Lane 1 between east pier and Toll Booths;
- Longitudinal and transverse joints requiring sealing;
- Longitudinal cracks (major);
- Separation of the surfacing from the kerb;
- Movement/settlement cracks at discontinuities/changes in construction;
- Minor cracks (less than 500mm long and 6mm wide);
- Blisters.

B) "Functional" defects:
- Variation in chipping density and embedment;
- Variation in (predominantly) transverse profile, from rip to rip;
- Variation in longitudinal profile (substrate related).

Following the observation from the site inspection, samples were subsequently removed from the bridge deck for further testing.

Laboratory Testing

Slabs of existing mastic asphalt surfacing materials and waterproofing layer were removed from the deck during repair work. Core samples were subsequently taken from these slabs and they were subjected to the following tests at the URS laboratories in Nottingham:
- Indirect tensile stiffness modulus (ITSM) of the mastic asphalt to BS EN 12697-26;
- Indirect tensile strength of the mastic asphalt to BS EN 12697-23;
- Shear modulus of the waterproofing layer to BS EN 14770.

The stiffness modulus and tensile strength test results of the mastic asphalt are summarised in Table 2.

Table 2: Summary of Stiffness and Tensile Strength Test Results of Mastic Asphalt

Core Reference	Stifness Modulus (MPa) at			Tensile Strength (MPa) at		
	-10°C	20°C	30°C	-10°C	20°C	30°C
Average of 3	20210	3430	1950	3.1	2.4	1.2

The stiffness values in Table 1 are considered to be lower than those expected for mastic asphalt material with 50% TLA. The stiffness modulus of such mastic asphalt at 20°C is expected to be greater than 7000 MPa (typically around 9000MPa). This indicates that the existing material had suffered a significant, and unexpected, reduction of up to 60% of the initial stiffness value of typical mastic asphalt, or that the material supplied was not as specified. Reduction in stiffness modulus results in reduced load spreading ability of this layer, leading to higher stress in the waterproofing layer.

The waterproofing layer contained a blend of rubberised bitumen and fine aggregate. A small sample of this material was taken and placed in a dynamic shear rheometer (DSR) for shear modulus testing; the results are summarised in Figure 2.

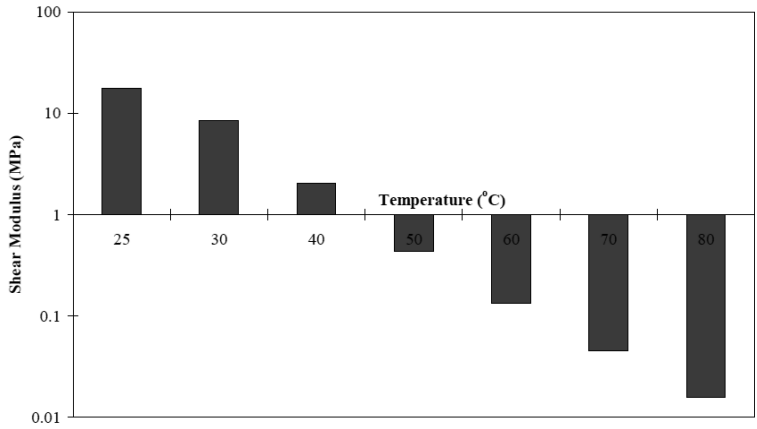


Figure 2: Shear Modulus (0.4Hz) of the Existing Waterproofing Layer

The results shown in Figure 2 indicate a rapid reduction in shear modulus of the waterproofing material as the test temperature exceeds30^{o}C; the measured values were at least 100 times lower than that of the mastic asphalt surfacing (which itself was already considered to have low stiffness). The observed significant difference in stiffness/shear modulus values could become a critical weakness under slow moving, heavy, traffic loading. These results also suggest the waterproofing layer had inherently poor resistance to shear flow/deformation.

Remedial Options

The existing surfacing material was not considered to have adequate performance in the Plymouth approach and sidespan to carry the residual future traffic which was anticipated for the remaining surfacing design life and consequently it was recommended to be completely removed and replaced by a better performing bridge surfacing system. Three remedial options were proposed, specifically: (1) polymer modified mastic asphalt (PMMA) surfacing; (2) polymer modified gussasphalt (GA) surfacing, and; (3) epoxy asphalt (EA) surfacing. Methyl methacrylate based waterproofing layer was recommended for use with either option (1) or (2) whilst epoxy based waterproofing layer was recommended for use with any option. Each of these surfacing systems had to be designed to meet the performance criteria at a level that was deemed adequate to carry the future design traffic at the Tamar Bridge.

To account for material response at different traffic speeds, the shear modulus values shown in Figure 3 were adopted for further analysis. These values were determined by establishing a correlation between traffic speed and loading time of the laboratory test data, and subsequent mixture stiffness calculation, using the well known method proposed by van der Poel (1954). Respective values for the existing material (mastic asphalt and RBU waterproofing) are also presented for reference purposes.

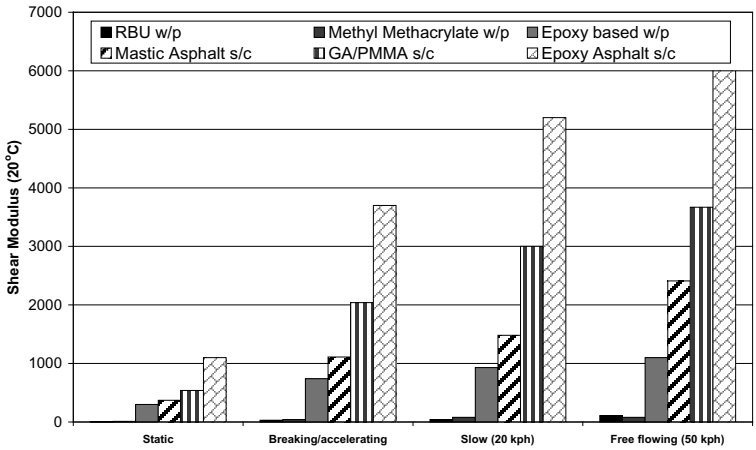


Figure 3: Shear Modulus (at 20°C) of Surfacing Material (s/c) and Waterproofing Layer (w/p)

Finite Element Model

Finite Element Models (FEMs) of the bridge / pavement structure were developed to assess the potential localised distresses within the asphalt surfacing at 20°C temperature under traffic load effect, in order to identify the key performance parameters of the asphalt to carry the predicted future traffic loading against different distress modes. FEMs were prepared for the main and cantilever decks. In each model, a section of the orthotropic steel deck, waterproofing and asphalt surfacing was modelled in order to investigate the effects of the wheel loadings. The effects of global loads (e.g. wind loading) was not considered. The analysis of the deck and surfacing was carried out using a three dimensional ANSYS v.11 FEM of a section of orthotropic deck. The model allowed composite action between the deck and the mastic asphalt. The results of this analysis were used to verify that local (as distinct from global) deck behaviour would not unduly stress the surfacing, causing premature distress.

Several surfacing systems were considered in the analysis. For the main and cantilever decks: the transverse and longitudinal girders and the troughs were modelled using 4 noded elastic shell elements. The steel deck plate, waterproofing and asphalt surfacing were modelled using 8 noded elastic structural brick elements. Boundary conditions were applied to each model (main deck and cantilever), in order to simulate the restraint to the deck provided by adjacent parts of the structure. The material properties assumed were based on the different material types and the effects of varying traffic speeds on the surfacing; the material properties of the deck constituents were assumed to be linear elastic, homogenous and isotropic, and; the geometric properties of the deck constituents were assumed to be non-linear.

Applied loading was based on a pair of axles, with the loading arrangement as follows:

- Single axle load: 116kN (the axle consists of two 58kN wheels);
- Pressure of the wheel contact with the pavement: 0.7 MPa;
- Contact area: 0.083m^2;

- Distance between wheels on axle (i.e. transverse to deck): 2.6m;
- Axle spacing (i.e. longitudinal to deck): 1.8m;
- Braking/traction (horizontal load): 23.2kN (two 11.6kN loads).

The braking/traction force represents 20% of the normal force and was applied to one axle only in the longitudinal direction. The horizontal shear force represents additional load to account for vehicle braking and acceleration. The applicable value for a particular situation will depend on the nature of the site and traffic magnitude (for example, the effect of horizontal force is more significant with increased axle loading and/or reduced traffic speed); in this case, 20% was considered to be appropriate.

A total of 17 models were analysed using the FEM analysis to estimate the maximum tensile strains within the asphalt surfacing, and maximum shear and tensile stresses at the surfacing/waterproofing interface, at different traffic speeds, encompassing existing and proposed new surfacing systems. Outcomes from this analysis were subsequently fed into the pavement design.

Pavement Design

Three main criteria were considered to check the performance of the various surfacing systems as follows:

- The maximum tensile strain within the asphalt surfacing material, to limit fatigue cracking under repeated loading.
- The maximum shear stress to limit shear failure, either within the surfacing causing surface deformation, or at the interface between the surfacing and the waterproofing layer.
- The maximum vertical tensile stress, to limit debonding between the surfacing and the waterproofing layer and further cracking within the surfacing.

Traffic Data

The total traffic data from each category and each individual year were calculated. The data for each year and category type were divided by 365 [days] to give an Annual Average Daily Flow (AADF). The total yearly traffic data were used to calculate the 'growth rate' between one year and the next. From the traffic data provided, HD24/06 (DMRB, 2006) uses the OGV1, OGV2 and bus traffic data in order to calculate future design traffic in 'million standard axles' (msa), i.e. 80kN standard axle loads. However, supersingle 116kN axle loads were used for Tamar bridge surfacing design and consequently the equivalent traffic damage due to this axle load was also predicted. OGV1 and OGV2 denote Other Goods Vehicle 1 and 2 respectively. Cars do not cause significant damage to the pavement and were not included in the future design traffic calculations. The calculated AADF for OGV1, OGV2 and buses in 2005 were 665, 285 and 11 respectively.

The past traffic (2001 to 2009, inclusive) and future 20 year design traffic (starting from 2010) were calculated to be 4.23msa and 13.90msa respectively, for 80 kN standard axle loads. These correlate to residual and future 20 year design traffic for 80 kN standard axle loads for the existing and new pavement of 9.67msa (which was the difference between 13.90msa and 4.23msa) and 13.90msa, respectively. These values are equivalent to the design traffic for 116 kN axle loads for the existing and new pavement of 2.18msa and 3.14msa respectively.

Maximum Allowable Horizontal Tensile Strain in Asphalt Layer

The maximum asphalt tensile strain criterion is used to limit fatigue cracking under traffic loading. The number of load applications expected during the surfacing life is used to calculate the allowable strain, incorporating the material fatigue characteristics. The maximum strains calculated under a 116 kN axle load using the FEM were subsequently compared with the allowable strain in order to specify the minimum asphalt design stiffness.

The normalised "site" fatigue lines for various materials are presented in Figure 4, which clearly illustrates the improved fatigue properties of the new materials (GA/PMMA and EA). These lines were derived from laboratory fatigue test data of the respective materials. SF1 and SF2 are the shift factors applied to the respective laboratory fatigue characteristics to account for different material quality and in situ conditions, respectively; the latter were derived from the work by Brown et al (1985) but modified for bridge deck application. The allowable horizontal strains and the respective predicted lives for the various combinations are summarised in Table 3.

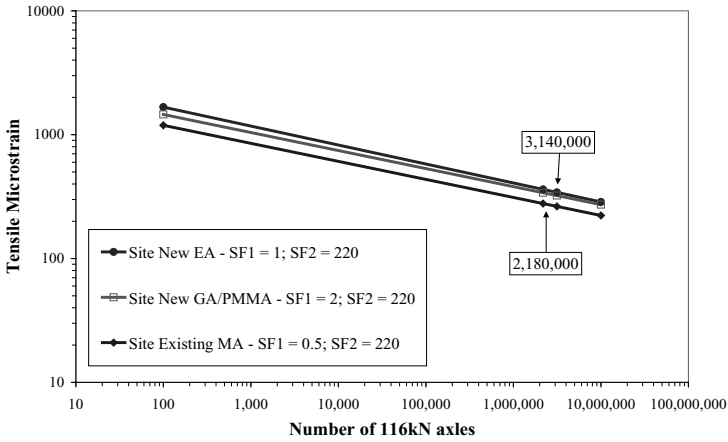


Figure 4: Normalised "Site" Fatigue Lines for Various Materials

Table 3: The predicted life for each surfacing systems

Surfacing System	Traffic Speed (kph)	Horizontal Tensile Microstrain			Predicted Life (years)	
		Allowable	FEM Deck	FEM Cantilever	Deck	Cantilever
Existing MA	Static (0)	278	571	680	0.1	0.0
Existing MA	Braking/Accelerating (0 – 20)	278	365	440	3.1	0.9
Existing MA	Free flow (50)	278	220	260	>20	>20
GA/PMMA	Static (0)	322	440	470	2.4	1.5

GA/PMMA	Braking/Accelerating (0 – 20)	322	320	330	>20	17
GA/PMMA	Free flow (50)	322	175	195	>20	>20
EA	Static (0)	341	395	410	5.0	3.8
EA	Braking/Accelerating (0 – 20)	341	175	235	>20	>20
EA	Free flow (50)	341	<100	170	>20	>20

Maximum Vertical Tensile Stress at Asphalt/Waterproofing Interface

The maximum vertical tensile stress may cause either debonding at the above interface or cracking within the asphalt or waterproofing material. Results from the FEM analysis and the respective values expected for the existing and new systems are summarised in Table 4.

Table 4: Tensile Adhesion Strengths and FEM Calculated Tensile Stresses

Waterproofing	**Tensile Adhesion Strength (N/mm^2)**	**Maximum FEM Tensile Stress at Waterproofing Interfaces (N/mm^2)**	
		Main Deck	**Cantilever Deck**
Existing – RBU	0.95 – 1.05*	0.92 – 1.03	4.83
New - methyl methacrylate	6.52**	1.18 – 1.20	0.81 – 1.07
New – epoxy resin	2.5 – 7.35		

Note: *from in situ testing to DMRB BD47/99. **manufacturer's data.

The above results show that the existing RBU waterproofing material does not have sufficient tensile adhesion strength, whilst the new alternative materials show higher strength and are expected to withstand stresses induced by traffic loading

Maximum Horizontal Shear Stress at Asphalt/Waterproofing Interface

The shear stresses under surface loading may cause failure at the interfaces between the asphalt/waterproofing and the waterproofing/bridge deck, or within the waterproofing layers. The results from the FEM analysis and the respective values expected for the existing and new systems are summarised in Table 5.

Table 5: Shear Adhesion Strengths and FEM Calculated Tensile Stresses

Waterproofing	**Shear Adhesion Strength (N/mm^2)**	**Maximum FEM Horizontal Shear Stress at Waterproofing Interfaces (N/mm^2)**	
		Main Deck	**Cantilever Deck**
Existing – RBU	n/a*	0.27 – 0.35	0.42
New - methyl methacrylate	3 – 4 **	0.10 – 0.20	0.35 – 0.49
New – epoxy resin	2.5 – 7.35***		

Note: *good quality material typically 0.7 N/mm^2; ** manufacturer's data. ***assumed to be similar to tensile adhesion strength.

There were no shear adhesion test data available for the existing material; the results in Table 5 show that the new material has sufficient strength to withstand stresses induced by traffic loading.

Analysis

The current speed limit for vehicles crossing the Tamar Bridge is 30mph (approximately 50 kph). However, it is not practical to regulate traffic speed. For surfacing design purposes, it was therefore considered appropriate to use a conservative assumption of a slow moving speed of 20 kph, with a small proportion of traffic at stand still and/or braking/accelerating, as the basis for assessment of a 'running lane'.

The following assumptions were adopted during traffic analysis:

- For the North Cantilever (NC), where traffic is predominantly non-commercial vehicles, it was assumed that the number of commercial vehicles was 30% of the total traffic;
- For the Main Deck i.e. Eastbound (EB), Main Deck Centre (CT) and Westbound (WB) lanes, it was assumed that the number of commercial vehicles for each lane was 70% of the total traffic.

The FEM analysis results for the existing surfacing system indicate that:

- The main deck material would not be able to carry the design traffic if the speed of traffic was less than 20kph. The residual life was estimated to be around 1.5 million standard axles or approximately 3 years of trafficking (at these speeds), much less if the traffic is static. If the traffic was running at a speed of not less than 20kph, the main deck surfacing material was predicted to just meet its design life;
- The cantilever deck material would not be able to carry the design traffic if the traffic speed was less than 20kph, with a residual life of less than a year;
- In addition to the above, however, the risk of the surfacing material debonding from the deck was a significant negative factor with respect to surfacing life, since the tensile adhesion strength was either comparable with (in the case of the main deck), or less than (in the case of the cantilever deck), the (FEM predicted) maximum tensile stresses induced by traffic.

The FEM results for the alternative PMMA/GA surfacing over methyl methacrylate waterproofing systems, and the alternative EA surfacing system, indicated that the main and cantilever deck material would be able to carry the design traffic under moving traffic condition (above 20 kph). The alternative waterproofing materials, methyl methacrylate and epoxy resin, showed improved strength compared with the existing rubberised bituminous underlay. These new materials can be expected to have sufficient strength to withstand shear and tensile stresses induced by traffic loading. Assuming the new materials were produced and laid to strict quality control procedures, the longest service life may be achieved by the EA surfacing system. However, it is recognised that there are practical and workmanship concerns that needed to be adequately addressed if such materials are to be successfully implemented.

Recommendations

Assuming new materials are produced and laid to strict quality control procedures, the results showed that the longest service life may be achieved by the EA surfacing system. However if traffic speed can be regulated to be kept above 20kph, then GA/PMMA could provide a more cost effective solution over the 20 year design life. Whichever system was adopted, it was

recommended that the existing material should be stripped down to the steel deck, followed by application of a new protective primer (as appropriate) prior to placing the selected new surfacing material system. As with any construction work, good quality control during construction (e.g. joint detail) is essential.

In reality, it was recognised that there was always the likelihood of braking and standing traffic in the pavement as cars approached the Toll Booths, and standing traffic would be a challenge for any of the three systems. It was therefore recommended that whichever option was chosen, a contingency should be allowed in the Operations and Maintenance budget for future (limited) remedial works in these areas. Finally, it was concluded that the likelihood of achieving the 20 year design life would be higher if a thicker GA layer (i.e. 55mm or greater) could be accommodated for the Tamar Bridge resurfacing.

The work described in this paper was done between 2007 and 2009 and was incorporated as a supporting document to the AECOM surfacing strategy report which investigated the whole project aspect of the surfacing strategy, in particular taking account of the potential structural feasibility of any surfacing thickness increase. Associated structural design checks were then undertaken together with various project option costings. A proposal was put forward to the Tamar Bridge Board and the most urgent phase of the resurfacing work on Plymouth approach and sidespan was completed by Autumn of 2011. The chosen system was a sprayed methyl methacrylate waterproofing with a 55mm layer of machine laid Gussasphalt in line with the recommendations from the URS investigation. Following the re-surfacing work the ride quality over the surfacing has noticeably improved and surfacing performance to date has been good.

Acknowledgement

This paper was based upon work commissioned to URS by AECOM on the behalf of Tamar Bridge Board and Cornwall Council. The authors would like to express their gratitude for the Clients' permission to extract the relevant information. The views expressed in this paper are those of the authors and not necessarily those of the organisations or the clients they represent.

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Forensic Engineering: Informing the Future with Lessons from the Past
ISBN 978-0-7277-5822-4

ICE Publishing: All rights reserved
doi: 10.1680/feng.58224.095

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Assessing the risk to railway structures from train impact

John Lane, Rail Safety and Standards Board, London, England.

Kevin Thompson, Rail Safety and Standards Board, London, England.

Background

The ability of railway structures to withstand impact loads from derailed trains is key to the safety of passengers on trains and stations and people using the structures (pedestrians and occupants of road vehicles for example). Where design decisions are required these should be made with an understanding of the underlying risk.

Potentially, such forces can be extremely high if the full mass of the train is mobilised at the point of impact at line speed. However, for most practical situations, the likelihood of an impact and its magnitude being representative of the worst case scenario (maximum mass impacting a structure at maximum line speed) in the event of a derailment is low. In such cases it is reasonable to provide a level of robustness that can be achieved in design with an acceptable risk to the safety of people. Furthermore the option of guaranteeing safety is not a realistic proposition, as even a small improvement in safety is likely to be achievable only at a very high cost. The costs of the works necessary to improve safety, can be grossly disproportionate to the enhancement of safety that can be achieved.

Design standards provide values of impact forces that are intended to achieve a minimum level of robustness for railway structures that are located adjacent to and span above the track. Where their application would result in an impractical or uneconomic design, those responsible for the management of railway infrastructure are sometimes faced with the dilemma of deciding what is a reasonable impact force to use in design. Logically this must depend on the likelihood of derailment, the speed, mass and frequency of passing trains, and the likelihood of an impact in the event of a derailment and consequential collapse of the structure. However, it is often unclear what impact scenarios are represented by the values provided in design standards and how they might be changed to reflect the operational conditions at the location in question.

The over-riding requirement in UK law is to manage risk to a level that is as low as is reasonably practicable (ALARP). The test for reasonable practicability has been outlined in case law which states '*a computation must be made…. In which the quantum of risk is placed*

on one scale and the sacrifice involved in the measures necessary for averting the risk (whether in money, time or trouble) is placed in the other' (Edwards vs National Coal Board 1949). Therefore it is possible to specify design requirements by considering the likely frequency and consequences of train derailments at the location for which a structure is being designed and the cost of providing additional structural integrity to withstand an impact.

The Construction Design and Management Regulations (CDM) 2007[1] place responsibilities upon designers that require them to avoid foreseeable risks to health and safety by eliminating hazards and reducing the risk of remaining hazards; however the general rule of reasonable practicability still remains.

This paper will examine and comment upon the application of a suitable methodology for the design of structures adjacent to and above the track, taking account of current risk data from the RSSB Safety Risk Model and application of guidance in the RSSB publication 'Taking Safe Decisions'.

Typical design situations

In principle, wherever structures are adjacent to or span above, the track, they are vulnerable to impact from a derailed train. In practice, the vulnerability of a structure depends on a number of factors including the:

(a) likelihood of a derailment,
(b) consequences of a derailment,
(c) proximity of the supports to the track,
(d) robustness of the supports and the supported structure,
(e) degree of support protection provided,
(f) passing speed of the train,
(g) track alignment in the vicinity of the structure,
(h) occupancy of buildings related to the structure,
(i) number of pedestrians and road vehicles using the structure, and
(j) occupancy of passing trains.

Structures that might be vulnerable include:

(a) station buildings adjacent to the track,
(b) other buildings and structures adjacent to the track, and
(c) structures (bridges or buildings) spanning the track.

RSSB has received a number of applications for deviation from design standards, most of which have involved situations at stations where the structure supports are located close to the track and the costs of providing a compliant design have been unreasonably high. Instead of focussing only on the design of the structure a more holistic view of the risk should be taken, by considering the system-wide risk. Factors that will influence the risk at a given location are listed above and include the purpose of the structure, the number of people on trains and at stations at risk of harm, neighbours close to the structure site, and the potential for development close to the railway. A key consideration for vulnerability at stations is whether the structure support is located within a platform and if so the degree of protection

against impact that might be available from the platform construction. This can be a significant factor at stations where the degree of passenger and staff occupancy can be high.

This holistic view can be achieved by following the principles set out in 'Taking Safe Decisions' (see section 6).

Standards used for design

Now that the Eurocodes have been implemented as the UK design standards for structures, Part 7 of Railway Group Standard GC/RT5112[2], requires design to be based on the Eurocodes and their National Annexes where available. Reference is made within GC/RT5112 to BS EN 1991-1-7[3] and its National Annex[4] for the value of the forces (actions) for impact due to a derailed train.

The BS EN 1991-1-7 and UK National Annex design requirements, are based on the recommendations set out in UIC leaflet UIC 777-2R[5]. These recommendations provide a classification system for structures based upon the type of traffic and line speed. The structure class is dependent upon whether it is a building or a bridge and the degree of occupancy.

Table 1 – Structures classification system

Class of structure	**Type of traffic**	**Line speed (km/h)**
A	Mixed traffic	120
B	Passenger	300
B	Freight	160

Design values for impact are recommended in BS EN 1991-1-7 and its National Annex, for Class A and B structures. Where the design impact values for Class A and B structures (representing situations where high speed impacts are likely) cannot reasonably be achieved at an affordable cost, it would be appropriate to assess the values for design impact force on the basis of a risk assessment.

Table 2 - Table 4.4 from EN1991-1-7

Distance "d" from structural elements to the centreline of the nearest track (m)	Force Fdx[a] (kN)	Force Fdy[a] (kN)
Structural elements: d < 3m	To be specified for the individual project. Further information is set out in Annex B.	To be specified for the individual project. Further information is set out in Annex B.
For continuous walls and wall type structures: 3m < d < 5m	4000	1500
d > 5m	0	0
a – x = track direction; y = perpendicular to track direction		

Based on experience to date, circumstances will arise for a particular project where it is not reasonably practicable to achieve the design values stated in the standard. In such circumstances, risk assessment is a valuable tool for demonstrating that the design solution has reduced the risks to a level that is ALARP. Design for the potentially maximum force from a derailed train (i.e. maximum mass at maximum speed) is unlikely to be economically achievable in many design situations which arise in practice. The design values in BS EN

1991-1-7, have been shown to be representative of light impacts at speeds below the maximum line speed.

Designs should take account of known or anticipated future changes in railway traffic, to ensure that the risks remain ALARP over the design life of the structure.

Application of the Safety Risk Model

The precursor to train impact events, is train derailment. Fortunately, train derailment is a rare event and therefore the risk of impact with structures adjacent to and above the track is also a rare event. There are approximately 7 passenger train and 19 non-passenger train derailments in GB per year, most of which remain close to the operating envelope of the train (ref RPB v7). This occurs along a track length of 31,000 kms over which there are 11,146 overbridges, of which 2,498 are steel or wrought iron structures more than 100 years old. In addition, 2,880 footbridges span the track, of which 791 are located at stations with an average age of 100 years.

The derailments of concern are those that go significantly beyond the operating envelope at speed. Such events are even rarer, the most recent in GB being at Grayrigg on 21st February 2007, where faulty switches and crossings (S&C) caused the train to derail, whilst travelling at a speed of 92 mph, then to leave the track, and then to roll down the embankment. At Potters Bar (10th May 2002), derailment of the train due to faulty S&C, occurred whilst the train was travelling at 100mph and the fourth vehicle became jammed between the platform and the platform canopy. During its passage along the platform, the fourth vehicle had encountered and demolished, amongst other things, an overhead line equipment (OLE) mast on the Down platform ramp, several lighting columns on the Down platform, and part of a waiting shelter on the Up platform.

RSSB provides independent knowledge, analysis, technical expertise, information and risk management tools for the benefit of the rail industry across a whole range of subject areas. This includes the collection and analysis of safety intelligence on behalf of the industry and the on-going development of the Safety Risk Model (SRM). The SRM is the most comprehensive railway safety risk model available worldwide.

Figure 1 is based on data from the SRM and shows the decline in both the risk and frequency of derailments in GB, where it has halved over the last ten years. Passenger train derailments are of most concern as the occupants of the train are at risk from falling structures. However, freight trains are not without risk, particularly if dangerous goods are being conveyed. In addition, any occupants of the structure should be considered, such as pedestrians, road vehicle occupants, and workers for example.

The causes of derailments are a significant factor which can influence the consequences of a derailment event. The most frequent cause on GB railways is landslide, causing approximately 15% of passenger train derailments (ref RPB v7). Landslides affect the potential for derailments in cuttings, which may be spanned by crossing structures. Derailments where switches and crossing are implicated are also frequent and therefore have a bearing on design decisions. 31% of derailments occur at switches and crossings due to a variety of causes such as mechanical defects, signals passed at danger, human errors and signal failures.

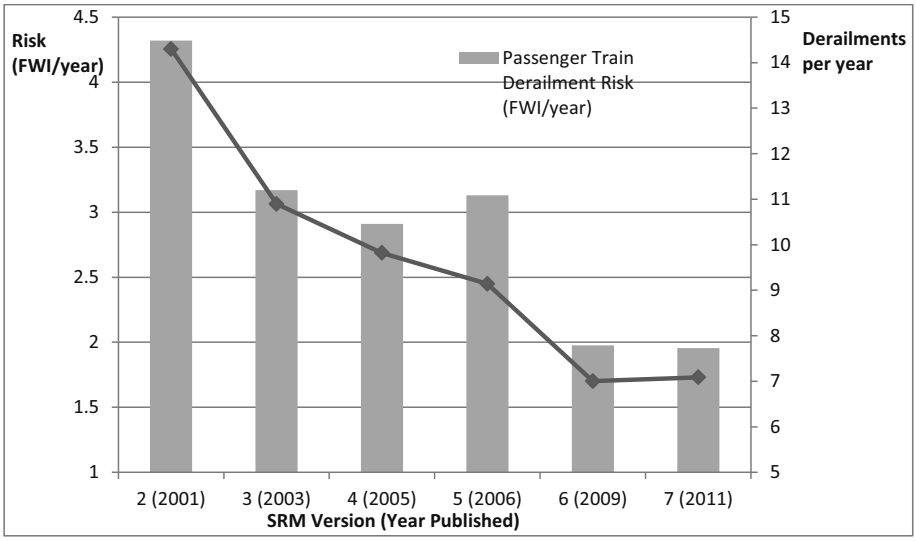


Figure 1 Changes in GB passenger train derailment risk (2001 – 2011)

Information on derailments across Europe is available in the UIC Safety Database Activity Reports, the most recent, at the time of writing, being published in December 2011 and reporting on incidents that occurred in 2010. In 2010 it reports that there were 126 significant derailments, an increase on previous years. Three-quarters of these were caused equally by rolling stock, infrastructure failures and human error. 44% of European derailments occurred at switches and crossings, a higher proportion than in GB.

There is a great disparity between the derailment rates across Europe, the worst experiencing over 4 derailments per million train kms, compared with the best at less than 0.5 derailments per million train kms.

Methodology for assessment of risk

Where the design requirements based on the values in design standards are not reasonably practicable to achieve, the risk assessment methodology described in Appendices F and G of UIC Leaflet 777-2R, 2nd Edition, September 2002, 'Structures built over railway lines – Construction requirements in the track zone', can be used to ensure an appropriate level of safety at reasonable cost. The method provides a basis for assessing the risk of fatalities arising from train derailments resulting in impact with structures, collapse of structures and secondary collisions with trains on the adjacent track, but does not consider the wider system risks that could be taken into account, such as operational limits for example. A basis for taking system-wide decisions is described within section 6.

The key geometrical parameters considered within the risk assessment methodology are shown in Figure 2. An assumption implicit within the methodology is that collapse of the structure is assumed not to occur where the speed of the derailed train is less than 60 km/h. The methodology involves the application of an 'event tree' analysis which considers the joint probability of critical events occurring in parallel. The risk value obtained from the analysis is then used, together with an assessment of the number of fatalities, to calculate the

cost of preventing fatalities. A figure of £1.763M was used for the cost of preventing a single fatality.

The forces from design standards may be used to calculate the nature and cost (C_{tot}) of the works necessary to resist them, or alternatively to reduce their effect. In reality, the resultant force applied to the structure will be dependent on the energy absorbed by the structure (a function of the stiffness of the structure and the train), the speed of impact and the mass of the train associated with the impact. Simple energy based methods can be used to estimate the design forces as an alternative to using the values provided in standards. Otherwise it is always possible to undertake a complex analysis involving the dynamic behaviour of the structure and the train. This is not a subject which is covered within this paper.

For the purpose of demonstrating application of the UIC risk assessment methodology, four scenarios concerning a bridge structure spanning the track have been considered, assuming various values of the distance 'a' (2.0m at stations and 3.0m at non-station locations) between the face of the supporting structure and the centreline of the adjacent track carrying the passing trains (i.e within the hazard zone).

The following circumstances are considered in each scenario:

1. Derailment of a passenger train travelling at 120 km/h on a double-track section, with switches and crossings (S&C) present on the approach to the station, with no protection provided to the columns of the supported structure, and with a potential need for investment in works to prevent the impact or to protect the supports of the affected structure from impact.
2. Derailment of a passenger train travelling at 120 km/h on a double-track section, with switches and crossings (S&C) present on the approach to the station, with protection provided to the columns of the supported structure by the station platform construction.
3. Derailment of a passenger train travelling at 200 km/h on a double-track section of plain line, with switches and crossings (S&C) present on the approach to the structure, with no protection to the columns of the supported structure and the need for potential investment in works to prevent or protect the supports of the affected structure from impact.
4. Derailment of a passenger train travelling at 200 km/h on a double-track section of plain line, without switches and crossings (S&C) present on the approach to the structure, with no protection to the columns of the supported structure and the need for potential investment in works to prevent or protect the supports of the affected structure from impact.

The results of the risk assessment are expressed in terms of the ratio of the cost associated with prevention of loss of life in the event of an impact (R_{bpm}) and the investment cost required to prevent impact or to protect the structure (C_{tot}). The annual costs of prevention/protection are assumed to be spread over the design life of the bridge structures (125 years). The results are summarised in Table 3.

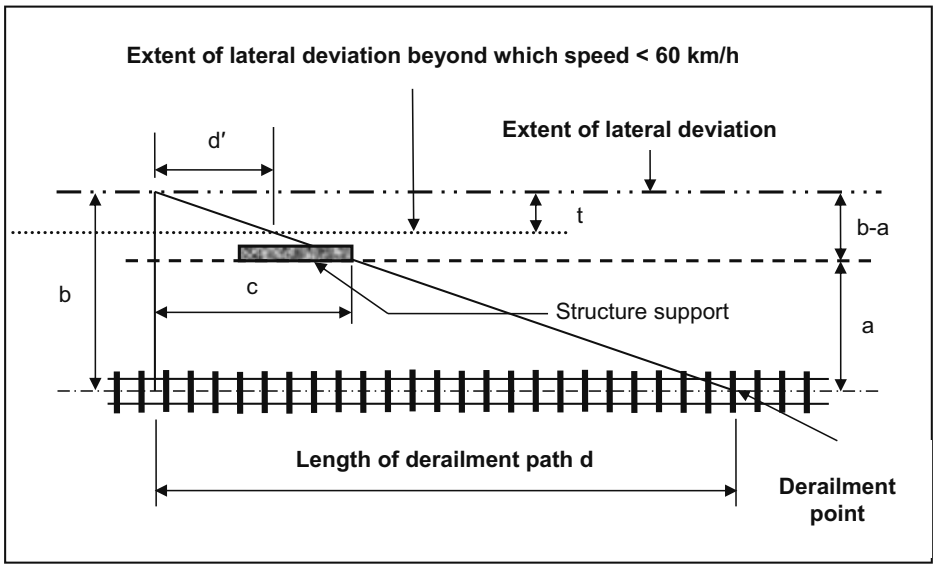


Figure 2 – Geometric parameters for risk assessment

For scenarios 1, 2 and 3, the greater costs of human life are primarily influenced by the more frequent derailment rate where S&C are present and where there is a high number of passengers on the train and potentially waiting at the station. Additionally, for scenario 3 a significant factor is the high speed derailment of a train with a high number of passengers on board. This was the case for both the Eschede and Potters Bar incidents, where the presence of S&C played a significant role in initiating derailment of the train. In all cases the costs of preventing loss of human life (the benefits) is greater than the costs associated with the provision of prevention/protection, and may therefore be considered to justify the investments that are proposed for the works.

Table 3. Results of risk assessment

Scenario	Cost associated with loss of human life (R_{bpm})	Annual cost of prevention/protection (C_{tot})	Ratio (R_{bpm} :C_{tot})
1	£33,100	£28,383	1.17
2	£33,100	£14,191	2.33
3	£79,000	£29,340	2.69
4	£7,903	£29,340	0.27

For scenario 4, this is not the case as the costs associated with prevention of loss of human life are significantly less than the costs of the works necessary for prevention of impact or protection of the structure, due to the significantly lower derailment rate where S&C are not present. In all cases the risks may be considered to be ALARP although for ratios close to 1.0, stakeholder validation may be considered to be appropriate.

Ultimately the decision about whether to invest money on works to reduce the likelihood of loss of life, will be made on an economic basis taking account of the assessed risk, the

associated costs, and the options for mitigation, including operational measures. For a particular project, the risks and costs can be established with greater certainty as the circumstances are known. The risk assessment can then consider the specific operational conditions that affect the safety of structures at risk from impact, and also take into account the holistic range of safety issues that affect the safety of the railway as a system (see section 6).

Guidance on taking safe decisions

In 2008 RSSB issued railway industry guidance called 'Taking Safe Decisions – how Britain's railways take decisions that affect safety'[7]. The purpose of the document is to outline an established agreed common approach for tackling decisions that affect safety; the robustness of the design of structures being a case in point. The document sets out a framework that can be followed that leads to a decision being made. This is shown as figure 3 below:

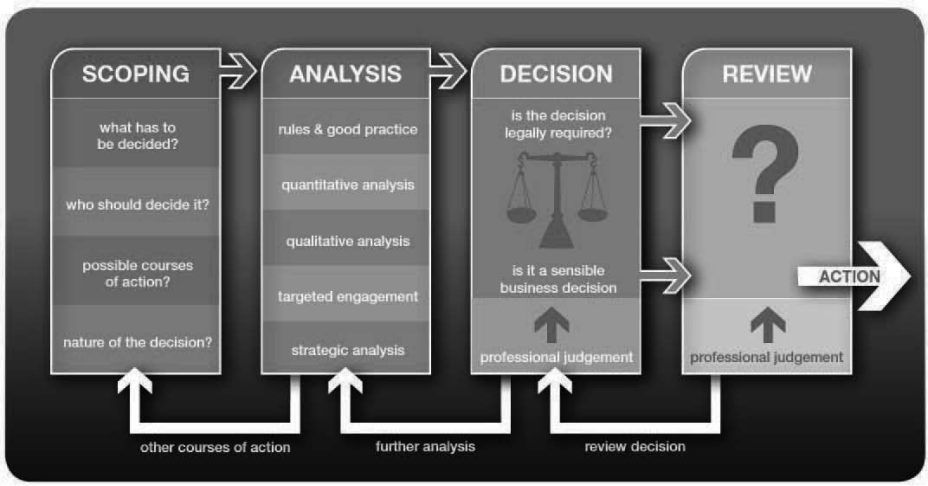


Figure 3 – Safe decision framework

The framework has four sequential steps with feedback:

- Scoping: what is the nature of the decision, who should take it and what are the possible courses of action?
- Analysis: how do rules and good practice, quantitative and qualitative analysis, targeted engagement and strategic analysis help me to reach my decision?
- Decision: based on analysis and understanding of the problem, what decision should be taken and for what reason?
- Review: does the decision make sense? Over time, what does experience tell me about the decision taken?

The first step requires all the possible causes of action to be considered. The options available might include, check / guard rails, derailment protection, protecting the structure, redundancy of support structures, provision of structural integrity, and a 'do nothing' option.

It is essential that the approvals strategy is understood at the outset, including what review processes are required, who will make the final decision and how will it be approved?

'Taking Safe Decisions' recommends that decisions be made upon the basis of established rules and good practice; this might involve compliance with the existing standards or using existing designs. However, it allows for changing risk controls which are no longer considered to be reasonably practicable. This is where it might be appropriate to challenge the existing standards which may be based upon a pessimistic view of the risk.

In 'CDM' terms the client has a key guiding role as to how they would like the design to evolve. Their approach might be to follow established patterns, or alternatively they may wish to challenge existing precedents, particularly where there is an opportunity to save costs. The McNulty[8] review of the railways has identified the need for the industry to save money and this represents a potential cost saving opportunity.

Once the scope of a safety decision has been agreed, analysis will be required on how cost beneficial each option will be; this will involve;

- Estimating the residual risk for a structure with no protection or added structural integrity in place;
- Estimating the change in risk by the introduction of each option.
- Comparing the risk saving for the life of the structure (converted to a monetary value using the current 'value for preventing a fatality' (VPF) with the cost of implementing and maintenance of each option.
- Selecting the option or options that provide the greatest value for money, which may be a 'do nothing' option.

In the case of protecting structures from derailments the emphasis will be on the change in consequences that is envisaged from differing levels of structural integrity.

Historically, where there is the potential for multi-fatality accidents, there has been a tendency to only accept 'gross disproportion' as an acceptable means of taking safe decisions. However, there has been no clear guidance as to what is meant by this and as a result this has the potential to drive unnecessary cost into the industry. Instead, figure 4 below shows that as the cost benefit moves away from 'one' there is increasing evidence to implement or not to implement. Where the value is close to 'one' we recommend that supporting analysis and the outputs of targeted engagement with key stakeholders, should be taken into account.

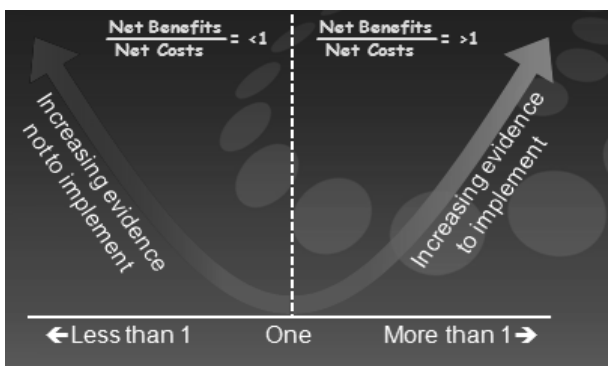


Figure 4. A basis for making safe decisions

Conclusions

This paper has examined the application of the UIC risk assessment methodology for impact on railway structures. It has taken account of risk data from RSSB's Safety Risk Model (SRM) and the guidance in the RSSB report 'Taking Safe Decisions'.

It has shown that, in some cases it is not possible to satisfy the prescriptive approach in design standards for specific values of impact force. However, within the railway environment, risk data is available from the RSSB SRM, enabling more informed decision making. As the collation and modelling of safety risk data matures, the data provided will become more reliable and be more able to support design decisions for impact, that fall outside of the standards regime.

The UIC risk assessment methodology provides a basis for quantitative risk assessment, but does not consider the potential benefits that can arise where system-wide safety is taken into account. Where the UIC method has been used, the approach outlined in RSSB's report on 'Taking Safe Decisions' can be used to validate the design decisions taken.

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Forensic Engineering: Informing the Future with Lessons from the Past
ISBN 978-0-7277-5822-4

ICE Publishing: All rights reserved
doi: 10.1680/feng.58224.105

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Calculating Collapse: Analytical Approaches for Investigating the Cause of the I-35 West Bridge Failure

Liling Cao, Thornton Tomasetti, New York, NY, USA

Elisabeth Anna Malsch, Thornton Tomasetti, New York, NY, USA

Francesca Brando, Thornton Tomasetti, New York, NY, USA

Gary F. Panariello, Thornton Tomasetti, New York, NY, USA

John Abruzzo, Thornton Tomasetti, New York, NY, USA

Abstract

The I-35 West Bridge in Minneapolis Minnesota collapsed on August 1st, 2007 at 6:04 pm killing 13 people and injuring many more. A nearby security camera captured a video of the collapse. In order to determine the cause of the collapse, a set of analysis results using finite element models and hand calculations were combined into one non-linear model to simulate what occurred on the day of collapse. The collapse model contains the non-linear material properties, loading scenario and findings from the inspection history of the bridge. As such it captures the actual demands upon and actual capacities of the bridge prior to collapse. This combination of a priori and a posteriori knowledge was used to drive the analysis and trace the actual behavior at its various bifurcation points. The results from the analytical model compare well with the video, which helps to verify the collapse initiation theory. The completed collapse model proved to be a valuable tool for comparing various theories as to the cause of the collapse, and confirmed that the theory presented by the National Transportation Board does not tell the whole story.



Figure 1: I-35 Bridge collapse

Introduction

The I-35 West Bridge consisted of three continuous spans with a total length of 1,065 feet over the Mississippi river in Minneapolis, Minnesota (Figure 1). The bridge was supported by roller bearings at Piers 5, 6 and 8 and fixed bearings at Pier 7. The bearing supports were designed such that they allowed the bridge to expand and contract freely as the temperature changed. The main bridge trusses were comprised of built-up box sections and I-sections connected with gusset plates at each panel point[1]. The truss depth varied from 60ft at pier support to 36ft in the mid-span (Figure 2). The bridge was designed in the early 1960s and was opened to traffic in 1967.

Three major retrofits were performed during the life of the bridge to address various maintenance concerns. An additional two-inch thick wearing surface was added in 1977 to increase the concrete cover over the deck's reinforcing bars. The curbs and railings were replaced in 1998 to enhance the safety and reduce water infiltration[2]. These two retrofits increased the dead load of the bridge by twenty percent. During the summer of 2007 the deck expansion joints were being repaired, and the wearing surface was in the process of being replaced, when the bridge collapsed.

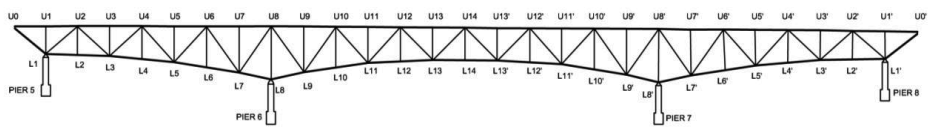


Figure 2: Elevation of main truss

Calculations performed subsequent to the collapse showed that the gusset plates at U10 and the corroded gusset plate at L11 in particular were under-designed by half[3]. Investigations into the cause of the collapse performed by the National Transportation Safety Board (NTSB) and others have focused primarily on the under-design of the U10 gusset plate and the effect of the dead and live loading on that connection on August 1st. The compressive load at the connection to U10 gusset plate of the L9-U10 member in particular was the focus of the

official investigation performed by the National Transportation Safety Board in the US[2]. However, none of the gusset plate theories explain why the bridge continued to stand for hours after the heaviest loads were placed on the bridge.

Another untested fragility of the bridge was its capacity to carry temperature loads. Over the years, bridge inspectors had cataloged the deterioration of the roller bearings (Figure 3). However, no analysis had been performed to ascertain if the bridge could withstand the large temperature swings common in Minneapolis. The combination of the increased weight of the bridge due to the retrofits, the construction vehicles and material stationed on the bridge and the temperature load effect proved to be catastrophic for the lower chord member that spanned from L9-L11. Buckling of the L9-L11 member initiated the collapse of the I-35 West Bridge.

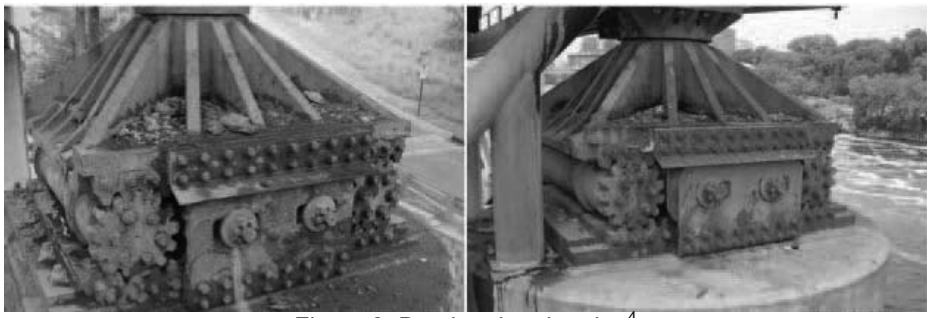


Figure 3: Bearing deterioration[4]

Collapse Simulation

Non-linear analyses pose challenges beyond those encountered for linear simulations. A non-linear model can be incredibly sensitive to very small details. This is especially true for when stability calculations are required. Also, collapse models in particular often have bifurcation points where two or more mathematically valid behaviors are possible for the same set of input conditions. A verifiable and accurate collapse model needs to contain the information gathered from the actual collapse event. The actual failure surfaces predict the failure capacities more accurately than any model could. The simulation results are then compared with the actual collapse shapes to check that the model and the theories and assumptions that drive it are reasonable.

The information about the I-35W Bridge, gathered before and recovered after the collapse, is essential to determining the cause of the collapse. Translating and compiling the many thousands of available documents into the language of the collapse model proved a challenge[5]. In the case of the I-35 West Bridge, a static bridge model was developed in SAP2000 to understand the static demands on the members. Critical pieces of evidence were then identified and detailed analysis of these critical locations was performed using Abaqus[6]. The analytical results were subsequently compared to the recovered members and used to drive the nonlinear and dynamic collapse simulation, performed using LS-DYNA[7], past various points of numerical instability (Figure 4).

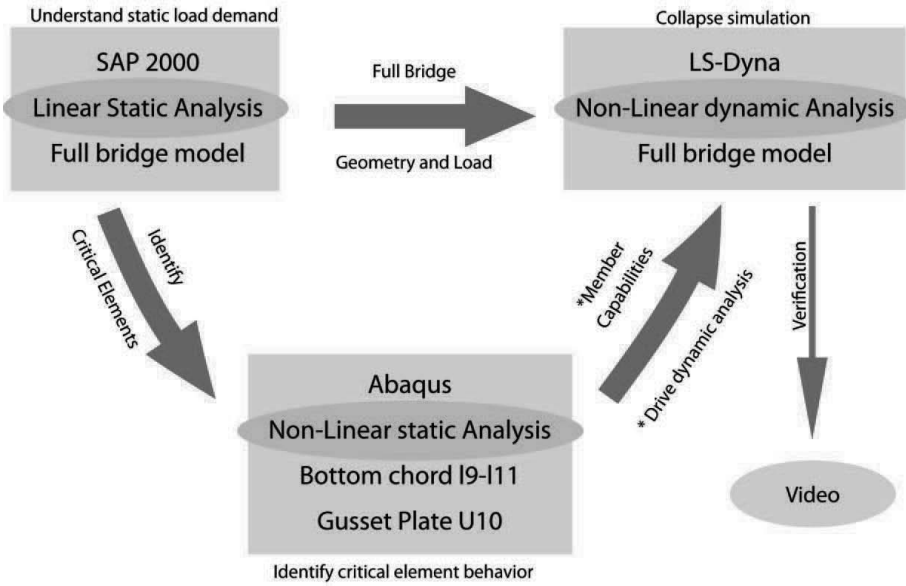


Figure 4. Analysis procedure

Bottom Chord Buckling

As the roller bearings gathered corrosion and debris ever larger forces were required to cause the bearing to move[8]. As a result, seized bearings turned a functionally non-redundant truss into an indeterminate structure. This led to a significant increase of forces in the bottom chord as the temperature increased which were not accounted for in the original design.

Local weather station readings show a 43 degree Celsius [110 degree Fahrenheit] change in temperature from winter to summer of 2007. The LS-DYNA and SAP models showed a significant increase of the axial force in the bottom chord member L9-L11, in particular, due to this temperature variation. The temperature in combination with the in-situ construction load, and the additional retrofits, more than doubled the original design load and exceeded the compression capacity of the member (Table 1).

Table 1: Loads and capacity of bottom chord L9-L11 on August 1st, 2007

Load Cases	Total August 1st Loads		
	Total Dead Load (kips)	August 1st Live Load (kips)	Seasonal Temp. Change (kips)
Load Values	690	80	1090
	770		
	1860		
Original Design Load	**919**		
Calculated Compressive Capacity per AISC code	**1700**		
DC Ratio	1860 / 1700 = **1.1**		

To evaluate the bottom chord member post-buckling behavior, a 3D model of the truss bottom chord member L9-L11 was developed in Abaqus using solid elements assembled to form a rectangular, perforated built-up section (Figure 5). The gravity loads on August 1st and the temperature load immediately prior to collapse[2] were applied in the analysis. Spring elements with the stiffness calibrated using the global SAP model were employed at either end of the member to simulate the effective boundary constraints from adjacent members. Nonlinear finite element analysis was performed using the arc length control to overcome the numerical instability caused by the bifurcation at the critical compression load.

The buckled shape with the local hinge formed near L11 from the Abaqus model matched well the collapse evidence of the south bottom chord as shown in Figure 5 and Figure 6. The compressive force-displacement curve generated from the nonlinear analysis (Figure 7) indicated that there was very little post-buckled capacity in the member after the inelastic hinges formed. The local buckling capacity matched the hand calculations performed per the AISC code (Table 1) and reconfirmed that the demands on the lower L9-L11 chord on August 1, 2007 exceeded the buckling capacity of that member.

Buckling of the bottom chord was the fuse that triggered the instability of the global system and initiated the collapse. Detailed simulation of the bottom chord behavior helped to accurately capture the failure progression. The Abaqus axial-displacement response was used to compute the capacity of the non-linear hinges imposed on the LS-DYNA model in the locations where they occurred in the actual collapse.

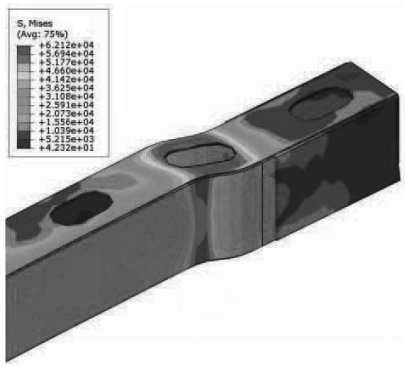


Figure 5: L9-L11 buckling with a local hinge formed near L11 in Abaqus



Figure 6: Photo of L9-L11 deformation at the plastic hinge

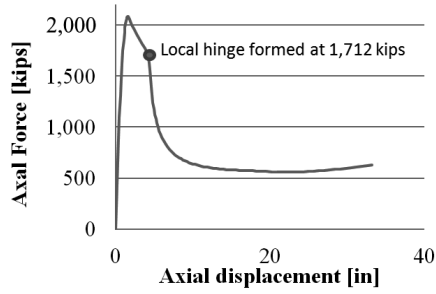


Figure 7: Axial force-displacement curve

Load Redistribution and U10 Gusset Failure

As the L9-L11 lower chord member buckles, loads are redistributed to the truss members. The load redistribution is calculated using both a linear-static SAP model and the nonlinear-dynamic LS-DYNA model. In particular members near U10 experienced a drastic increase of the forces. This increase resulted in a significant overstress of the U10 connection. SAP and LS-DYNA models predicted 'n'-shaped bending at the U10 gusset with tension on the top and compression at the bottom. The 'n'-shaped failure matches the evidence retrieved from the U10 connection after collapse (Figure 8, Figure 9).

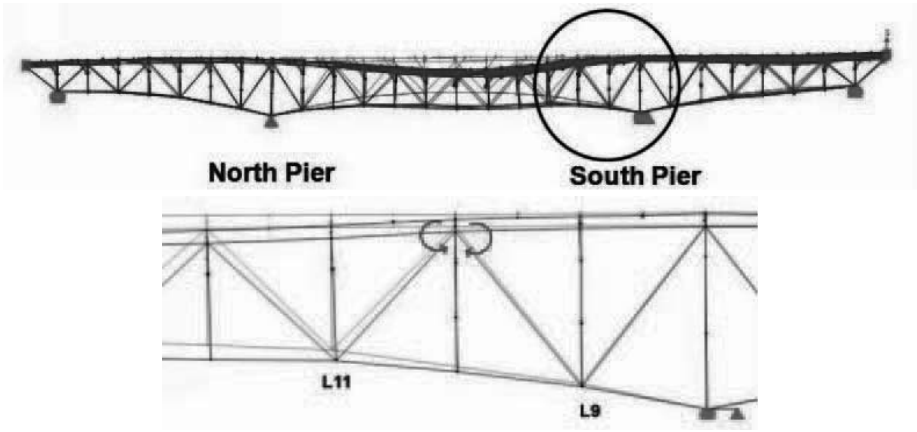
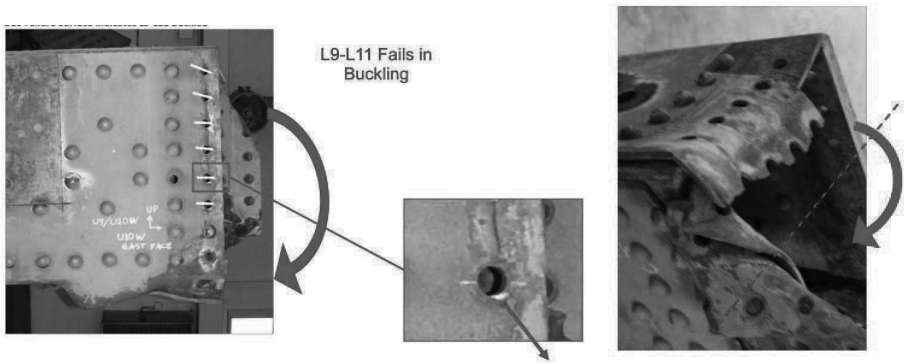


Figure 8: 'n' shaped bending at U10 connection



a. Side splice plate rivet bearing b. top splice plate bending

Figure 9: U10 connection bending

The LS-DYNA model revealed that the dynamic increase of the moment and axial forces was within a range of 1.1 to 3.3 times the original forces prior to the buckling (Figure 10 and Figure 11). The dynamic effects were more pronounced in the moment response than in the axial due to the damping effect from the concrete deck which acts compositely with the structure in the LS-DYNA model. A similar dynamic increase of axial forces would be expected if decks were non-composite. The LS-DYNA simulation theoretically overestimates the composite nature of the system because the expansion joints and the connection between the deck and the floor truss are not explicitly modeled.

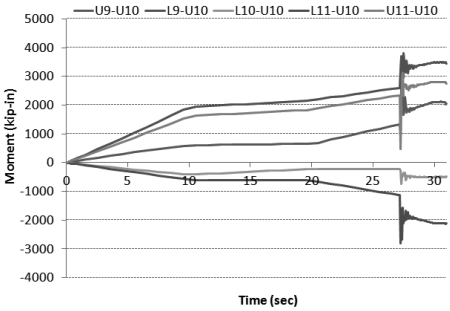


Figure 10: Moment responses of U10 members

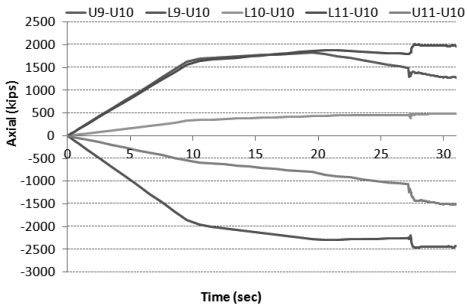


Figure 11: Axial responses of U10 members

To investigate the stress conditions at the U10 gusset plate, a 3-D gusset plate connection was modeled in Abaqus. The model consisted of truss members near U10 sandwiched by half-inch gusset plates on either side of the members and connected using rivets. All the members were modeled with homogeneous solid elements, and elastic-perfectly-plastic material models were applied to the elements. The increased axial and moment loads from the LS-DYNA model due to bottom chord buckling were applied to individual member ends. The gusset model verified 'n' shaped bending failure of the gusset with tensile rupture near the upper plate, which matched the evidence of local plate deformation and rivet hole bearing location from the recovered connection (Figure 12, Figure 9).

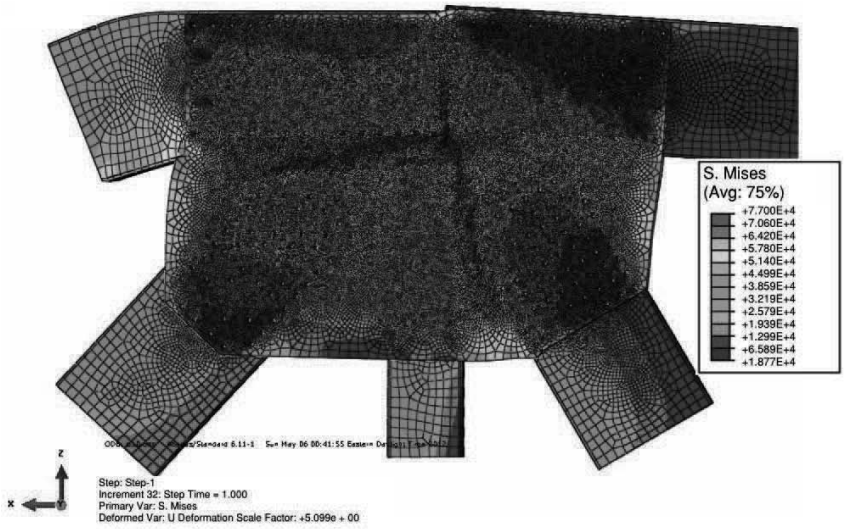


Figure 12: Von Mises stress of U10 gusset plate after bottom chord buckling

The U10 gusset plate model in Abaqus complemented the hand calculations performed to ascertain the capacity of the in-situ failure surface. The rupture strain of the upper gusset was computed as well as the rupture strengths of the incoming members, U8-U10, L9-U10, L10-U10, L11-U10 and U10-U12. These values were then imposed on the LS-DYNA model to drive the collapse analysis. The rupture strengths of the top chord U8-U10 and the diagonal

L9-U10 were adjusted to capture the fragility of the U10 gusset connection as determined from the collapse evidence and from the buckling fragility that the NTSB calculated[2].

LS-DYNA Collapse Model

The full 3D bridge model was developed using frame and shell elements in LS-DYNA. The main trusses were modeled with two types of beam elements, Hughes-Liu beam with cross section integration to capture the stress-strain conditions in the truss members and Belytschko-Schwer resultant beam to simulate the plastic hinge formation in critical elements. The concrete deck was modeled with shell elements.

The collapse model in LS-DYNA contained the dead load, vehicle load, construction load as well as the temperature gradient on August 1st, 2007. The seized bearings at Pier 6 were modeled as a roller combined with the lateral spring element, which was calibrated to reflect the actual stiffness of the pier. Gravity loads were gradually added to the model followed by a temperature change.

The bottom chord buckling behavior and the fragility of the U10 connection were modeled based on the detailed Abaqus models and hand calculations described previously. Nonlinear analysis with large displacement was performed with the implicit-explicit solver switch turned on. As such the model was able to capture both static effect and dynamic fluctuations from the load changes and sudden member failure (Figure 13).

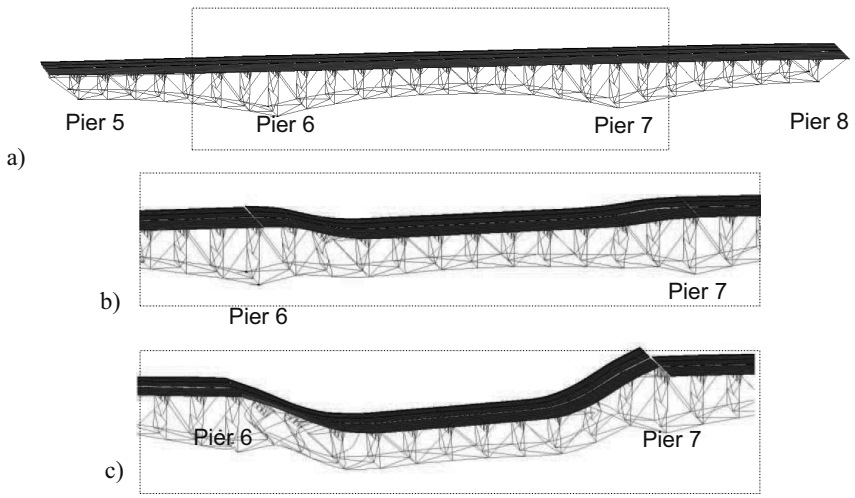


Figure 13: LS-DYNA collapse model of I-35 Bridge a) model overview b) initiation c) collapse.

Collapse Simulation and Evidence Verification

An integration of priori and posteriori knowledge combined to form a robust analysis tool. The various collapse theories were imposed on the model and tested. The LS-DYNA bridge model was able to simulate the various sequential failure mechanisms that occur as a result of the various collapse theories. The collapse initiated by the buckling of the L9-L11 chord most closely matches the material evidence collected after the August 1st, 2007 collapse. The

animation of the model results compared well with the videos[9] recorded by a surveillance camera at the south bank of the river (Figure 14).



Figure 14: Overlay of the collapse model on the video still after L9'-L11' buckled

Due to the combination of the seized bearings, temperature load effect and minimal post-buckling capacity of the truss member, the buckling and local plastic hinges of the bottom chord at the south end triggered the global instability and subsequent failure of the south gusset at U10, which occurred by means of rupture failure in the top chord U8-U10 at the U10 connection, followed by the south diagonal L9-U10 failure. As the collapse progressed, the north end bottom chord buckled followed by the north end U10 failure. During the collapse, the lateral force produced at the frozen bearing was large enough to overcome the resistance, leading the bridge to move forward and slide off Pier 6. The sudden impact on the bottom chord L7-L8 was predicted by the collapse model and is confirmed by the impact marks discovered in a recovered lower chord member (Figure 15).



Figure 15: Impact mark on L7-L8 near pier 6

NTSB Theory

Simulating the NTSB theory using a similar approach confirmed that the gusset plate initiation theory is not likely. Buckling at the U10 gusset results in 'u' shaped bending not 'n' shaped bending (Figure 16). The inelastic buckles in L9-L11 at the south end of the bridge are not explained by the NTSB's theory and the buckling of the L9'-L11' member at the north end seen in the video is not expounded. Furthermore, the NTSB theory does not cause the bridge to collapse unless the L9-U10 member overlaps with the top chord in a manner that is possible in a computer simulation but impossible in real life (Figure 16).

Conclusion

The tragedy of I-35 West Bridge required a comprehensive investigation of the failure mechanism and the cause of the collapse. A vast database of evidence was sorted and critical elements contributing to the collapse were selected for different levels of analysis ranging

from detailed finite element models to hand calculations. A nonlinear bridge model was developed in LS-DYNA and enriched with the local element analyses. A combination of priori and posteriori knowledge was acting as a powerful tool to drive the analysis past the bifurcation points in the dynamic collapse simulation. The simulation confirms that the evidence does not support the NTSB conclusion that a lateral instability at the under-designed U10 gusset was the fuse that resulted in the collapse. Instead, the model confirms that collapse initiated by the buckling of the bottom chord best matches the bridge collapse video and local failure behaviors.

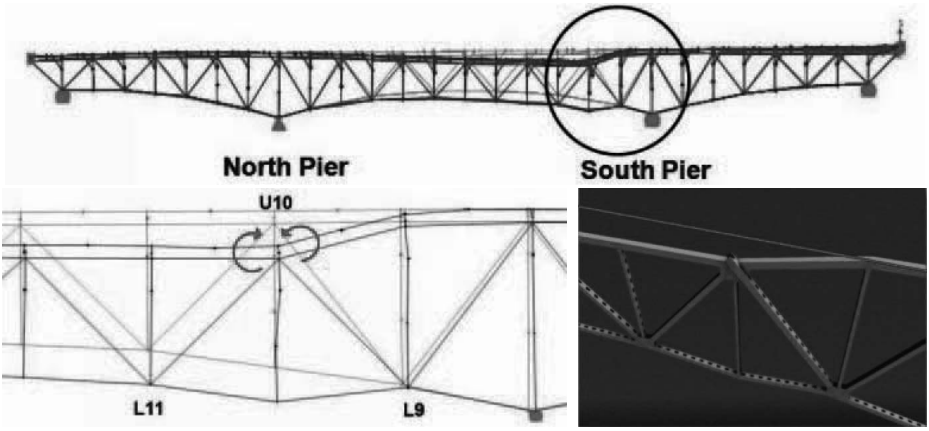


Figure 16: 'u' shaped bending and U10 failure from the NTSB theory

Acknowledgement

The investigation into the cause of the I-35 West Bridge collapse was performed for Robins, Kaplan, Miller & Ciresi, L.L.P., a national law firm with offices in Minneapolis, Minnesota. Firm partners recruited and oversaw a consortium of 17 law firms that agreed to provide pro bono legal services to the survivors of the collapse.

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Forensic Engineering: Informing the Future with Lessons from the Past
ISBN 978-0-7277-5822-4

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doi: 10.1680/feng.58224.115

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Assessment and repair of a fire-damaged pre-stressed concrete bridge

Rob Wheatley, **BEng (Hons) CEng MICE** Atkins, Epsom, United Kingdom.
Noel Gibbin, **MEng CEng MICE** Atkins, Epsom, United Kingdom.
Mark de Melo BEng (Hons) MSc CEng MICE, Arup, Solihull, United Kingdom.
Mauricio González-Quesada MSc , Arup, Solihull, United Kingdom.
Keith Harwood MA CEng MICE, Arup, Solihull, United Kingdom.

Keywords: Fire Damaged Concrete, Concrete Bridge Investigation and Repair, Bridge Assessment, Pre-stressed Bridge, Dean's Brook Viaduct.

Abstract
Limited guidance about the performance and repair of pre-stressed bridges can be obtained from research projects and guidance documents available regarding concrete structures damaged by fire. This paper reviews the methods used to investigate, assess and repair a concrete bridge, of pre-stressed precast beam construction, damaged by fire in April 2011. The aim is to increase understanding of the processes required to test, assess and repair a fire damaged bridge.

The repair of the Dean's Brook Viaduct supports the confidence expressed in TR68 regarding the resilience of prestressed concrete structures in the face of fire damaged. The experience showed that early involvement of all parties is invaluable in reducing timescales and improving the running of the project, and careful definition of the testing regime is important to the success of repairs. It is suggested that the cost of replacing key infrastructure would merit further study into repair strategies and standardised testing necessary to assess fire damaged structures.

Introduction
On 15th April 2011, a fire in a scrap yard damaged the south span of Dean's Brook Viaduct on the M1, in the Greater London Area. After the fire was extinguished, the deck soffit displayed significant signs of damage, including blackening of the concrete surface, spalling of concrete and exposure of reinforcement in several areas, showing both links and pre-stressing wires.

A reinstatement project was initiated after the fire and was substantially completed towards the end of 2011. A number of research projects and reference documents were considered for guidance towards the assessment and remediation of the fire damage. In particular, extensive reference was made to the Concrete Society's (2008) Technical Report No. 68 – "Assessment, Design and Repair of Fire-damaged Concrete Structures" (TR68).

This paper presents a case study of the reinstatement of Dean's Brook Viaduct. It describes lessons learnt and suggests areas for further research. The investigations

into the damage caused by the fire and the stages undertaken in reinstating the bridge are discussed, including testing, assessment, design and repair.

It is acknowledged that this paper makes extensive reference and use of documents prepared by the testing company RSK STATS Ltd., the concessionaire Connect Plus, and Connect Plus Services. Atkins were the designers and Arup were the independent checkers in the project.

Description of Dean's Brook Viaduct

Dean's Brook Viaduct carries the M1 motorway over a minor road between Junction 2 and Scratchwood Services. The road forms a service link to Scratchwood Services and access to a number of businesses which are located adjacent to and underneath the bridge. It was a fire associated with two of these businesses that brought Dean's Brook Viaduct to national media attention and initiated the assessment and repair project. Culverted beneath the access road and the M1 is the original Dean's Brook.

The bridge is owned by the Highways Agency (HA) but currently operated and maintained by Connect Plus Ltd, the Special Purpose Vehicle company commissioned to design build and operate the M25 widening project on behalf of the HA. Connect Plus comprises Balfour Beatty, Skanska, Egis and Atkins. In addition to the M25 the operational area of Connect Plus includes many of the connecting motorways, including the M1 into central London.

The bridge was constructed in 1964 as part of the Hendon Urban Motorway project to a design by W S Atkins. It has 3 skew spans of 15.2m, 23.5m and 15.2m with 17° skew. The superstructure consists of a reinforced concrete (RC) slab composite with prestressed concrete beams and in situ RC edge cantilevers (see Figure 1). The abutments are RC cantilevered walls supported on spread footings. Five oval RC columns make up each of the two intermediate supports. The deck is supported on elastomeric bearings at the abutments and is continuous and monolithic with the intermediate piers. The structure also includes regular narrow transverse diaphragms cast at approximately 2m centres and broad transverse diaphragms at the abutments and piers.

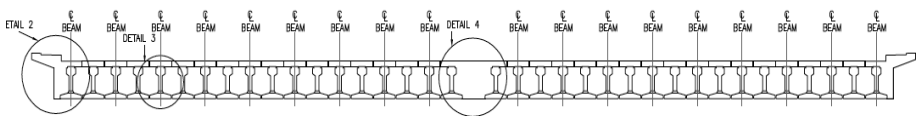


Figure 1 - Typical deck cross section

Prior to the fire, regular inspections had found the structure to be in good condition and assessments demonstrated adequate capacity to HA requirements. Access to the side spans had been seriously limited by the presence of the business tenants. Security fencing had been erected almost to the bridge soffit and inspectors had not recently gained access inside, so inspection was limited to viewing what was visible from publicly available land.

The incident

A fire was first reported at the bridge at 04:12 on Friday 15th April 2011. It was started by arson in a scrap metal yard and was centred on a flammable material store. The fire spread to the adjacent skip hire business where numerous skips and a skip lorry

were located underneath the bridge. The skips were stacked up to three high. The proximity of the skips to the soffit was to have a material effect on the damage caused to the bridge (see Figure 2).



Figure 2 – Immediate aftermath of the fire

The London Fire Brigade focused on avoiding explosion of the gas cylinders in the flammable material store. As priority was given to keeping the cylinders cool, the fire under the bridge was left to burn exposing the bridge to elevated temperatures over a prolonged period. The Brigade estimated that the fire could have reached temperatures of up to 800°C. Pieces of concrete were observed falling from the soffit around three hours after the start.

The immediate response

The fire was timed, albeit inadvertently, for maximum effect on the public. On that weekend two FA cup semi finals were to be staged at Wembley and was also the London Marathon. With the motorway closed there was intense media interest and close ministerial attention.

Once the area was handed over by the Fire Brigade, temporary propping was installed at 2m centres and the motorway progressively reopened. The bridge was inspected on the Monday; an initial assessment of the damage proposed neglecting any strength from the bottom layer of the prestress wires and assuming 75% of the ultimate strength for the second layer of wires. These figures were used to support the initial calculations and provide assurance that the road could be opened to 3-lane running in each direction at reduced speed (50mph) and no HB loading.

Project setup

The reinstatement project was undertaken in line with the stages described in TR68:

- Preliminary inspection and initial assessment of damage
- Testing and detailed assessment
- Design of repairs to structural elements
- Implementation of structural repairs

The strategic location of the road and the political profile meant that there were significant pressures to condense the repair programme. The stages did not all happen sequentially but ran concurrently and ideas were developed in parallel until evidence became available to refine the decisions affecting the next stage.

A core team was set up quickly by Derek Hughes of Connect Plus with Russ McNeill (Balfour Beatty) appointed as project manager. The HA team comprised David Keane as Project Sponsor and Mike Gibbs as Technical Approval Authority. In addition to Atkins and Arup, RSK STATS Ltd were appointed to undertake the material investigation. The HA Departmental Nominee was Hyder.

The investigation and reinstatement design required technical approval in accordance with BD 2/05 (HA, 2005) and the unusual circumstances warranted an independent check (category III). It was evident from the start that the complexity and importance of the challenge called for collaborative working and that involvement from all parties at an early stage would be beneficial in raising concerns early and reaching mutual consensus in order to allow a rapid design process. The HA were particularly keen that the scope of the testing was fully agreed by designer and checker in order to avoid repeated rounds of site investigation.

Workshops involving all participants were the main space for brainstorming, discussion and joint decision making. They were used to review the testing strategy and interpret test results. To illustrate the dynamic nature of the design workshops, data from the on site stress testing was telephoned directly to one of the meetings. Compared to the usual process where design progresses sequentially, this process encouraged and improved the flow of information between all parties. Early on, the advantages and disadvantages of the initially large number of options for repairing the damage were discussed. The improved visibility of the reasons for choosing a particular strategy brought early agreement and less risk of disagreements or unforeseen problems that would delay progress. It was particularly beneficial to the checker to have knowledge of and influence the ongoing design philosophy; and to the designer to have the advice and direction provided by the HA technical approval team at the earliest opportunity.



Figure 3 – Exposed reinforcement

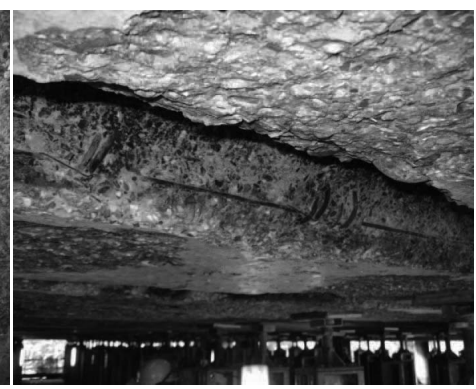


Figure 4 – Spalled soffit and sloughing of flange corners

Initial visual inspections and light hammer tapping survey showed that the damage had been limited to the south span of the bridge across most of its width. A typical illustration of the damage to the bridge soffit following the fire is shown in Figures 3 and 4. Prestress wires had been exposed in places and there had been extensive

spalling. In many cases substantial lengths of the beam edges had spalled, exposing the edge of the link reinforcement and the outer prestress wire.

Assessment of the damage

Detailed material testing of the structure identified several of the expected signs of damage pointed out in TR68. The concrete displayed pink discolouration of aggregates, hollow areas and micro-cracking above the pink aggregate horizon. Sloughing of flange corners was observed on several beams (see Figure 4). The damaged areas correlated closely with the position of the skips and skip lorry where peak temperatures would have been expected.

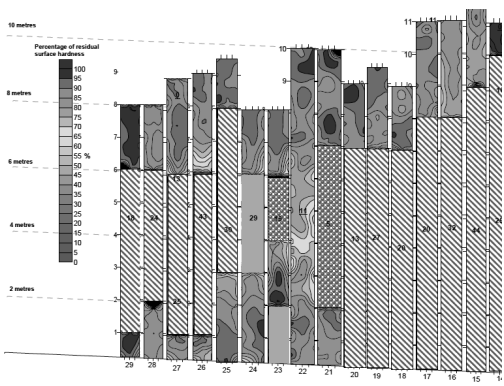


Figure 5 - Interpretation of the Schmidt Hammer test results (Hatched areas spalled, scale shows percentage of surface hardness retained relative to undamaged beams)

Further to the visual inspection, non-destructive Schmidt Hammer testing of the soffit was done. The Schmidt Hammer test was particularly useful in identifying areas likely to have been affected by the fire and presenting the results in a clear and informative way (see Figure 5). The survey also suggested that the effects of the fire on the surface of the concrete extended beyond the areas suggested by the visual and tapping surveys. Having identified the areas of greatest damage detailed material tests were used to determine the appropriate properties that would be needed for analysis of the fire damaged structure.

With regard to other elements of the bridge, the cathodic protection which covered the face of the south abutment was destroyed by the fire. The abutment itself was protected by the presence of this layer and did not show signs of major damage. The possibility of structural damage to the piers and the central and northern spans was also discarded. The vehicle parapet located above the east edge beams suffered extensive damage. The front faces of the elastomeric bearings were also affected, those closest to the fire sources being found to be charred to a depth of up to 3mm.

Material testing

The fire brigade estimated that the fire had reached temperatures of up to 800°C. The preliminary inspection showed significant spalling of concrete and exposure of pre-stressing wires. Concrete and steel suffer important changes in their properties at temperatures above 300°C (The Concrete Society 2008). Reinforcement performs better, but still suffers a long term reduction in strength above 500°C. TR68 also indicates that pre-stressed elements with damaged wires will generally be demolished and replaced. Clearly demolition would have a significant impact on road users so there was considerable interest in ascertaining the actual condition of the concrete, reinforcement and prestressing that could be safely used to include in a

repaired structure. The following testing regime was agreed in order to generate the information required for the detailed assessment of the structure and design of repairs:

Table 1 – Purpose of tests

Purpose	Tests
Depth of fire damage	Petrographic investigation.
Strength of concrete	Compressive testing of cored samples.
Condition of reinforcement and pre-stressing wires, residual tensile strength	Laboratory testing of samples taken from exposed and unexposed areas to determine, hardness, chemical composition, microscopic examination tensile testing.
Loss of pre-stress	Natural frequency testing (non-destructive). Measurement of deformation after cutting (destructive).
Condition of elastomeric bearings	Visual inspection and specialist survey by manufacturer.
Extent of fire affected surface	Schmidt hammer testing
Other verifications	Depth of carbonation, cement type Concrete PH and chloride content

Concrete

A major consideration in planning the testing of the beams was the close centres (25mm) of the prestressing wires. It would not be possible to take a concrete core without cutting a number of the 7mm diameter wires, but cutting the wires would reduce the available strength. Sampling was carefully designed to provide sufficient material data but avoid unduly damaging the bridge. 50mm diameter cores were selected to reduce the number of wires severed and four cores, approximately 200mm length, were cut vertically upwards into the web of the beams, cutting through both layers of pre-stressing wires (see Figure 6). They were positioned in areas of particularly severe fire damage. The data obtained were petrographic descriptions, thin-section microscopic petrographic examinations, compressive strength, chloride content, depth of carbonation and pH value. Shorter cores, which did not affect the second wire layer (80mm long), were obtained from every beam that exhibited evidence of fire damage. These cores were used to obtain petrographic descriptions, chloride content, depth of carbonation and pH value.

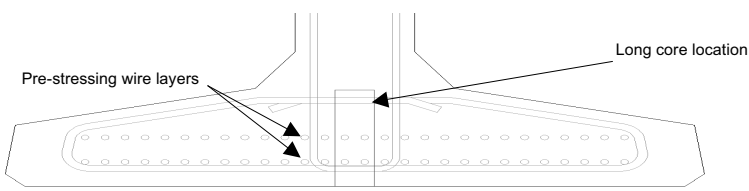


Figure 6 – Location of concrete cores within lower beam flange

The results from the petrographic examinations suggested that in places the original surface of the beams had been subject to temperatures in excess of 600°C. Furthermore, large portions of the beams, the edge beam and the cantilever slab had

been heated in excess of 300°C. Despite this, the petrographic analysis showed that the depth of penetration was not as deep as first feared when visually inspecting the bridge. The unspalled concrete provided an efficient thermal barrier that prevented large temperature rises at the level of the prestressing wires. An example of the output from the petrographic tests can be seen in Figure 7.

The petrographic tests showed that the temperature reached during the fire is an upper bound and that even weakened concrete can act as a protective barrier to limit the temperature reached by the reinforcement. They also highlighted the need to carefully choose the location of petrographic testing in order to use its results for prediction of pre-stress loss as well as concrete damage.

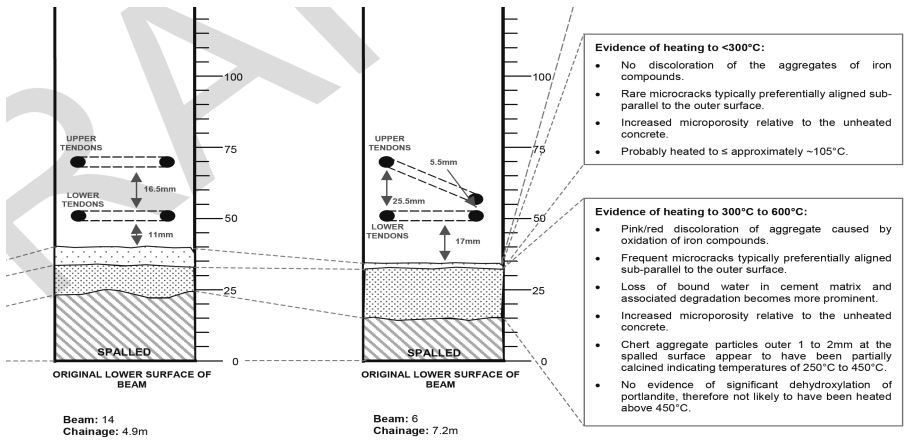


Figure 7 - Typical petrographic examination findings

Reinforcement and Pre-stressing Wires

Steel samples were taken and tested in the laboratory for tensile strength, chemical analysis, hardness and microscopic examination. The results demonstrated that the reinforcement and pre-stressing wire material properties had not been significantly affected by the fire. Metallurgical tests showed that decarbonisation of the prestress wires was generally limited to 0.1mm from the edge of the wire and the worst case reinforcing bar was 0.3mm of decarbonisation. Hardness and tensile strengths were unaffected. Whilst the standard tensile tests gave full stress-strain plots for the reinforcing bars they were not able to give the same level of data for the prestress wires, as the gauges were removed to avoid damage when the sample broke.

The post fracture measurements of elongation were not sufficiently accurate to give a reliable assessment of the elongation above the 0.2% proof stress. Hence it was decided to adopt a conservative limit for the strain in the wires taken from the recorded strains and adjust the ultimate limit state calculations to keep within a wire strain limit of 0.0085 and as well as the usual concrete strain limit of 0.0035 (HA 1995).

Remaining Pre-stress

TR68 does not mention any potential tests to determine the remaining force in a prestressing wire. However, it does make clear that significant losses of pre-stress are to be expected if wires are exposed to temperatures in excess of 300°C. Loss of pre-stress was considered a critical piece of information. Two different non-standard tests were specified: measurement of wire deformation after cutting (strain relaxation) and measurement of the natural vibration frequency.

Attempts at direct measurement of the force in the wires were complemented by an indirect determination of the plausible losses by assessing the temperature likely attained by the wires, based on the results of the petrographic testing. The petrographic tests shown in Figure 7 indicate that where there was available cover 50mm may be enough to avoid heating of the prestressing wires beyond 100°C. TR68 indicates that such temperatures would provoke at most a loss of 10 to 15% of force and a negligible loss in strength, which correlated well with the results from the tests. Areas where the wires were exposed directly to the fire showed significant loss of prestress. Two areas in particular showed that the remaining prestress was reduced down to 9.6kN a loss of 60%. Samples taken just 1m further along the beams where the wires were not exposed and cover remained, did not show the same degree of loss of prestress. This indicated that the losses were very localised at the exposed wires. The degree of prestress loss was again verified during the repair operations. A simple tap test on the exposed wires created a distinct, audible differentiation between fully stressed and partially de-stressed wires.

Assessment and design of repairs

Assessment and design of repairs was carried out in line with DMRB standards BD 44/95 (HA, 1995) and BD 24/92 (BS 5400-4) (HA, 1992). The proof that the beams still retained much of their initial prestress despite exposure to a five hour fire enabled a repair strategy to be developed. The capacity to carry load was reduced but remained well above the required strength. Serviceability limits were reduced and it was agreed that Class 2 crack requirements (HA, 1992), could be adopted at the interface between the original beam and the repair. The repair would not be prestressed. The damage to the bridge, was extensive but, largely owing to the significant spare ultimate capacity the strength of the deck, was sufficient to carry the imposed load. The repairs were therefore designed to restore the durability of the beams, abutment and bearings.

Hydro-demolition was selected as the preferred method of removing the damaged concrete and sprayed concrete reinforced with structural mesh chosen for the reinstatement. Balvac were engaged as specialist sub-contractor and were involved at an early stage in planning the phasing of reinstatement work and assisting with specifying the repair material. The typical repair detail is shown in Figure 8. The lighter shade shows the maximum allowable extent of breakout. The removed concrete would be replaced up to the darker shade after the fixing of the extra reinforcement, thus providing the required additional cover to reinforcement. The prestressing wires also had to remain anchored during the remedial works to prevent uncontrolled exposure of the lower wire layer. Therefore the repair methodology also included for limitations on the breakout of concrete at the anchorage.

Full time supervision of the works was provided by an engineer with experience in fire damaged structures, particularly to ensure that the depth of the breakout

removed all the damaged concrete but did not extend into the undamaged areas. This proved to be particularly valuable in ensuring that there was a high level of confidence in the thoroughness of the repair.

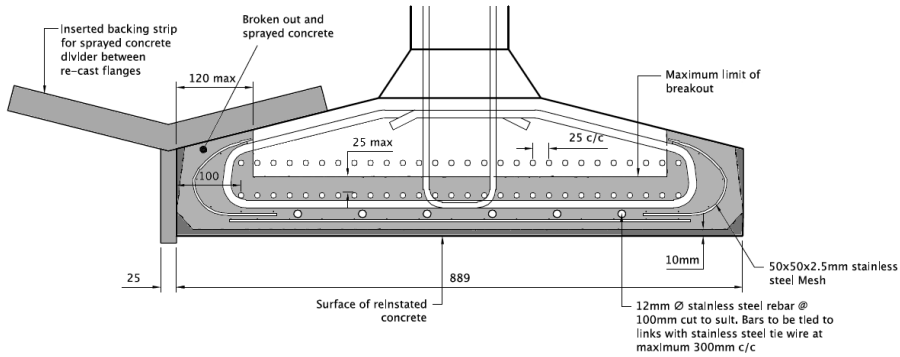


Figure 8 - Typical repair section

Consideration was given to the effects of traffic over the bridge during repair works, since it was considered very important for the bridge to remain partially open at all times.

For the breakout of the concrete it was agree that the props could be removed from one beam, which meant that the particular beam would be at its weakest during this stage. Once the concrete had been removed, the props were reinstated with a permanent mortar pack between the prop head and the beam. The repairs were phased to ensure sufficient separation between the repair works and the running lanes above, through traffic management. Where repairs were close to running lanes work was restricted to night time and monitoring was used to ensure that any vibrations were within the permitted limit of 5mm/ s^2. Generally recorded values were around 1mm/ s^2 with peak figures of 3mm/s^2. The bridge soffit before and after the repairs is shown in figures 9 and 10.



Figure 9 – Soffit before repairs



Figure 10 – Soffit after repairs

Conclusions

Limited guidance about the performance and repair of pre-stressed bridges can be obtained from the research projects and guidance documents available regarding concrete structures damaged by fire. The lessons learnt from the methods used to investigate, assess and repair Dean's Brook Viaduct, damaged by fire in April 2011 are summarised below:

1. The experience obtained during the repair of the Dean's Brook Viaduct supports the confidence in TR68 to the resilience of pre-stressed concrete structures in the face of fire damage. Where cover remained it was enough to prevent significant loss of pre-stress even after exposed to the fire. Where wires were directly exposed to the fire the losses were significant but localised.

2. The capacity between a Class 1 (decompression) Serviceability Limit State design and an Ultimate Limit State assessment is a useful source of reserve strength.

3. Early involvement of all parties relevant for the development and approval of the project at an early stage proved to be invaluable in reducing timescales and improving the running of the project.

4. Careful definition of the testing regime is key to the success of a concrete bridge repair following fire damage. Careful consideration should be given to the different uses of tests, for instance the usefulness of petrographic testing to confirm the temperature-depth profile through the concrete.

5. The nature of this project highlighted the lack of a standardised test for determination of remaining pre-stress in wires in service. It is suggested this is a field that would benefit from further study.

6. The strategic implications and costs involved with rebuilding an essential piece of infrastructure will make it worthwhile developing further testing and repair options for prestress structures.

Acknowledgements

The authors of this paper would like to thank the Highways Agency, RSK STATS Ltd., Connect Plus and Connect Plus Services for allowing the use and reference of documents prepared by them during the reinstatement of the Dean's Brook Viaduct.

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Forensic Engineering: Informing the Future with Lessons from the Past
ISBN 978-0-7277-5822-4

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doi: 10.1680/feng.58224.125

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The Role of Monitoring in Diagnosis of Bridge Faults

Bill Harvey Bill Harvey Associates Ltd, Exeter, UK
Hamish Harvey Bill Harvey Associates Ltd, Exeter, UK

Abstract

There is a deeply ingrained habit among engineers of leaping to conclusions about diagnosis. This is partly fuelled by the perceived difficulty and expense of monitoring work and its, all too frequent, failure to deliver worthwhile results. It is important always to ask first what you truly want to know and whether it can be measured. Sometimes it is possible to make simple measurements first to help with this decision. Measurements often bring surprises and it is also important to allow the data to speak freely. That is to maintain an open mind about potential behaviour.

The paper describes two projects in which relatively simple measurement has revealed unexpected behaviour. A masonry bridge appeared to have separated brick rings but this was proved not to be the case. Bearings in a large metal bridge were subject to plan rotation which may have contributed to deterioration.

Reference will also be made to situations where the measurements being taken could not possibly provide value. The reasons for such work going ahead will be discussed.

Introduction

In primary school science classes, the question most firmly driven home is:

"Is it a fair test?"

Engineers, at least in the UK, are not good at asking that question. Perhaps an example would be useful. The crack in Figure 1 is a metre or so from the edge of a bridge. If it widens at all it will do so through wearing away of the material not through moving apart of the sides. This high grade installation will show nothing useful. Measuring out of plane movement would be better but only if it could be done fast enough to pick up the range of live load movements.

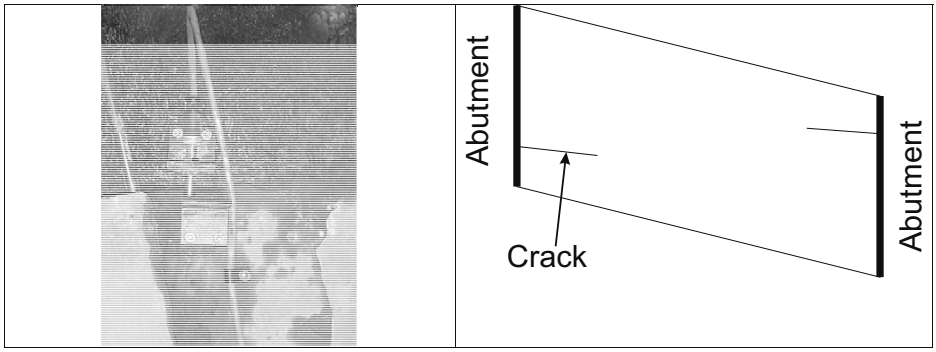


Figure 1 Monitoring a crack in the mid soffit of a masonry bridge

Often very high cost measuring is done without proper consideration of the frequency of measurement required. If axles cross a bridge in 1 second a frequency of at least 10Hz is required to be sure of finding the limits of movement.

One error that is particularly common is the assumption that stress can be measured. Strain may be a surrogate for stress but only changes in strain can be measured and often only very locally. An attempt to understand the stress patterns in an old riveted plate girder must take into account the nature of the girder and of the loads. Thus a welded strain gauge of 50mm gauge length attached to one plate of a built up riveted flange is likely to show predominantly local effects. If the rivets are at 100mm centres, a gauge length of 500mm might be more appropriate and some form of framing system might be needed to ensure that the strain is effectively averaged over the width. Once again, this is a particular case, though we do not propose to identify it.

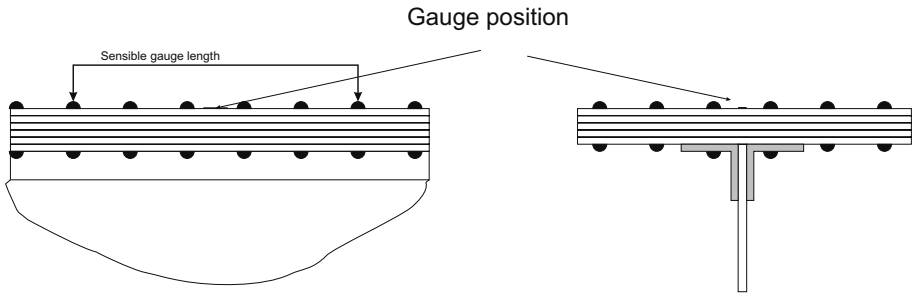


Figure 2 Girder strain measurements

In another case, accelerometers were used to measure x,y and z displacements on 5 piers of a masonry railway viaduct (Figure 3). Measurement “events” were triggered by movement and ran until the movement subsided. Accelerations were integrated and only peak values of displacement were recorded. The measurements were taken at different heights above ground which was itself sloping, yet the data recovered was used to declare that the foundation of one pier was failing, despite the fact that the only movements recorded were longitudinal to the bridge and the peak values were only 0.3mm (recorded as 300micro-metres).

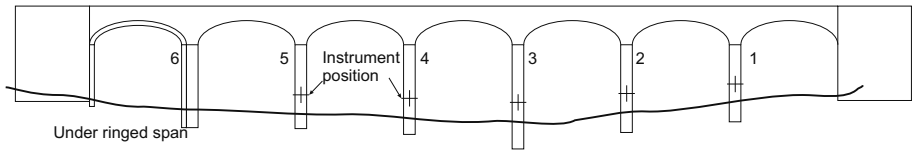


Figure 3 Viaduct instrumentation

A final example of problem cases is the attempt to measure displacements in a system using an automatic total station. The total station was reading perhaps 100 targets which might take 10 minutes. On a bridge over a tidal waterway carrying frequent trains there would be no correlation between different measurements.

Case studies

Damaged bridge bearings

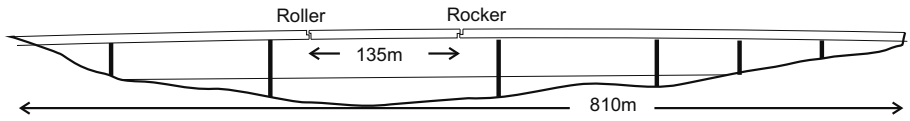


Figure 4 Cleddau bridge elevation

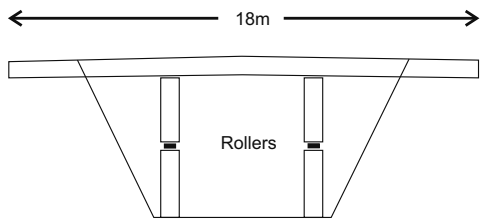


Figure 5 Cleddau bridge cross section

Cleddau bridge is an 800m long box girder over seven spans (Figure 4). The longest span is 230m and has joints at 67.5m each side of the centre creating a suspended span. One end of this sits on rocker bearings, the other on rollers which must allow the full temperature movement of 600mm. The box is 6m deep and has a top flange 18m wide and a bottom flange only 6m wide (Figure 5). The webs slope outwards. The depth of corbel required to support the bearings means that they had to be positioned in-board of the edge of the bottom flange and the rollers are thus less than 6m apart. Damage to the rollers had been apparent for some time before the full extent became obvious in the winter of 2011. The owners agreed that monitoring was necessary.

Nature of damage

The bearings carry about 500 tonnes each, though the full design load is rather more. The rollers are 203mm diameter and 440mm long with 20mm wide, 20mm high flanges to engage with the edge of the bearing plates and transmit wind loads. Rack and pinion gears, as required by various codes of practice, were intended to ensure continuous and parallel rolling.

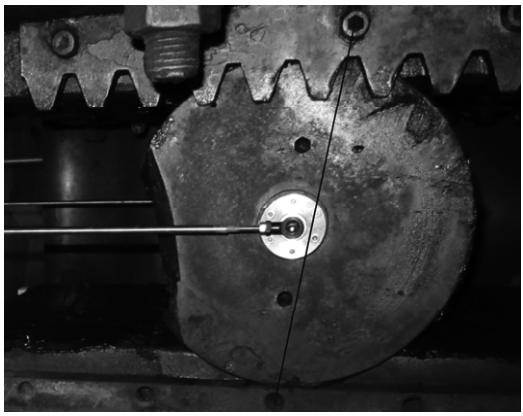


Figure 6 Damaged roller with sensor mounting

At the time of inspection, several of the pinions had broken free, one flange was badly damaged and a deep area of corrosion was found in the bearing plates (Figure 6). Note how the roller is off centre and the flange is embedded in the spacer bar below where there was originally 3mm of clearance.

Damage development

Following the gathering of data since discovery, it seems that corrosion was the primary cause of all the other damage, though other, unconsidered, behaviour may also have over stressed the bearings. A total of 8mm of material had been lost locally between the roller and the lower plate in the area where the bearing sits in the depth of winter. This corrosion caused the gears to over-engage which fractured the mounting bolts. It also meant that the top and bottom radius of the roller became different forcing an uneven movement in rotation.

Once the gears had broken free, a number of other things might have contributed to the bearings rotating in plan, but there can be no doubt that the damage to the flange was caused by such plan rotation.

Monitoring measurement

We wanted to be sure from hour to hour and day to day that the bearings were running free and so allowing the bridge to expand and contract without interruption. In a sense, such measurement could take the place of the gears, checking that the bearings were running rather than forcing them to. A minimalist approach to this requirement would have been simply to measure the movement of the joint on the bridge centre line. In general terms, once monitoring is set up, the marginal cost of making more or better measurements is very modest. We chose to attempt to measure the position of the joint outboard of each bearing and also to measure the position of each end of each roller. Over time, this would build up a picture of movement and ensure that the rollers were remaining sensibly parallel and at mid-point in the joint. After some debate, we settled on a 1 second cycle for measurement, producing 86k readings in 24 hours. On 6 gauges, this meant over 0.5M items of data each day to be measured and returned to a web store where they could be processed for observation.

A major difficulty was fixing gauges effectively to the centre at the end of the rollers. We built a unit that fitted into the centre drill hole used for making the bearings. It was held in place by a ring of small magnets. The gauges used were 300mm travel linear potentiometers for the rollers and 750mm travel wire pull potentiometers for the joint as a whole. To correlate with these measurements, temperatures were measured on the inside face of the box at mid span near the centre of the top, bottom and two webs. All the electrical instruments were positioned inside the box girder where they were well protected from the weather.

Running graphs were produced of:

- Temperature v time
- Joint movement v time
- Roller movement v time
- Bearing misalignment (difference in position between ends of a roller) v time
- And Bearing misalignment versus position.

A typical graph is shown in Figure 7. Reproduced in black and white some of the detail is difficult to identify. In the temperature curve the smooth rise is the deck. The double peak is in the west web as the sun goes down and the more mobile of the remaining two is the East web. In the second plot, the four roller traces move half as far as the two joint traces and the sets are almost coincident.

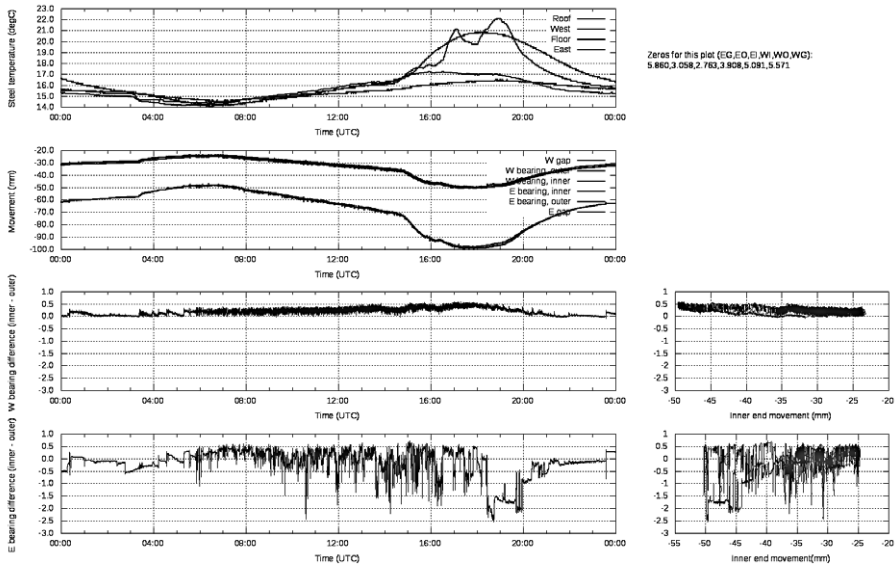


Figure 7 Web chart for 16th July 2012

The first issue of interest is the multiple spikes visible at different scales in each of the graphs. The charts at bottom right show that the rollers change direction of skew when the overall movement changes direction. This produces an overall loop in the graph which is clearer in the upper graph and more pronounced in the lower. The East bearing appears to be skewing by up to 3mm.

These spikes might be thought to be aberrations of the measurement but the detail recorded allows closer inspection. There is a spike in every graph around 00.20. Looking closer at the joint movement around this spike we get Figure 8 which shows many points on the spike tracing up and down movement. Cross referencing this with the toll record shows that a single vehicle crossed the bridge at this time. The very pronounced spikes in the bottom graphs represent the roller flicking back and forth as the joint moves under live loads.

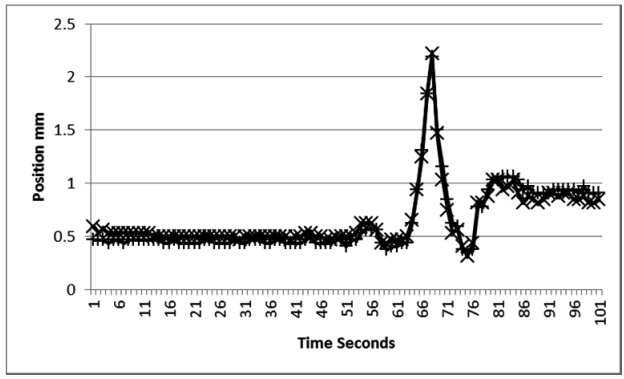


Figure 8 Joint movement East and West showing spike

The rollers are positioned well below the neutral axis of the girder so bending of the girder moves the rollers. There is a step in static position between before and after this spike suggesting that the bridge has been arrested in movement. The trace for the two ends of the east bearing shows the cause of this.

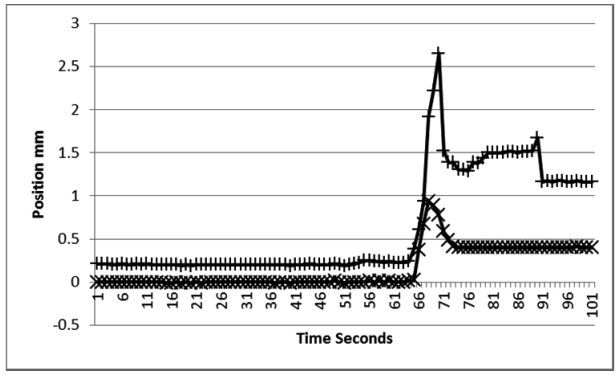


Figure 9 Displacement of the two ends of the East bearing

The West end of the East bearing is showing considerable reluctance to movement, forcing the East end to move more violently. Indeed, since these plots were set to zero at midnight, the West end is seen not to have moved at all in this 20 minute period.

Another interesting feature of Figure 7 is the separation of the traces for the two sides of the joint, visible around 18.00 to 19.00. This separation occurs consistently and is directly related to the rise in temperature of the West web at this time. A similar movement in the opposite direction occurs on sunning mornings. The total skew is about 8mm in 6m on a typical hot day which represents a rotation of 0.3mm in the width of a roller, a movement which is certainly not part of the design capacity.

Much of the detail evident in the record would have to be dismissed as anomalies if it were not for the fact that the record is so complete. Even increasing the time interval to 10 seconds would have resulted in the spikes of movement appearing as single points leading simply to confusion, rather than enlightenment. The lesson here must be: always measure as much and as frequently as possible. The data can always be compacted later by averaging or filtering but unmeasured detail can never be recovered.

Anomalous behaviour of a tuned mass damper

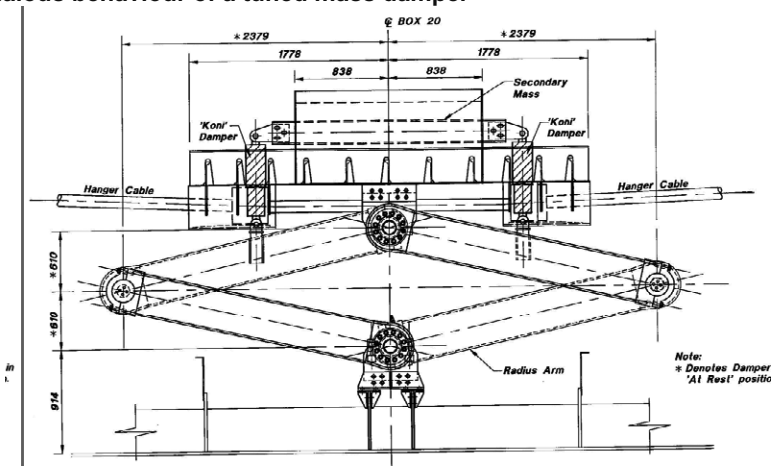


Figure 10 Tuned Mass Damper showing pantograph and cables

This damper is also on Cleddau Bridge. Click counter records suggest that one side of the damper travels further than the other, despite the fact that a pantograph is designed to keep it travelling true. Accelerometer measurements showed the damper to be working in terms of wind response but did not explain the apparent difference in travel. Physical observation recorded as a video on a simple digital camera showed some rocking motion during oscillation.

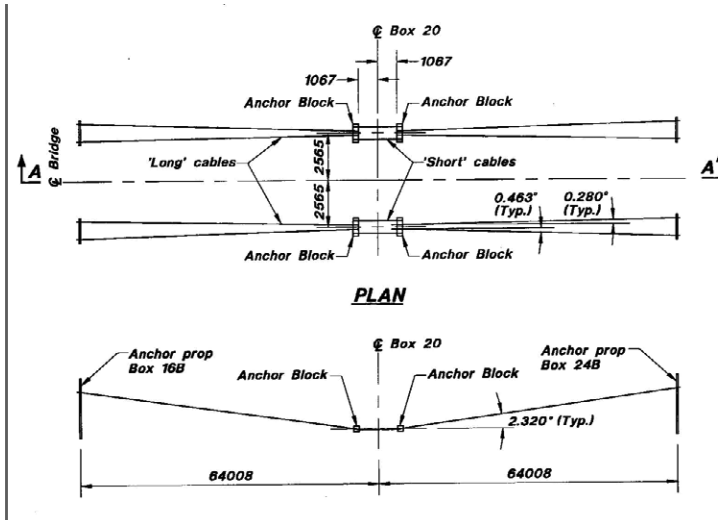


Figure 11 Damper support system

Measurements

Line pull potentiometers were again employed to record position of the mass at each of four corners. The natural frequency of the bridge is about 0.5Hz so we decided to record positions ten times a second. For this operation a logger with multiple A to D channels was used[i], allowing synchronous measurements. Here we were gathering, and transmitting over a mobile phone line, over 5M data

items per day, including the time stamps. The records are stored on the bridge for an hour then uploaded to the web.

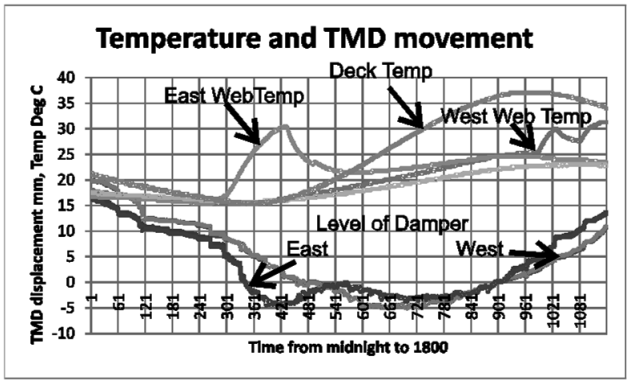


Figure 12 TMD movement record at Cleddau

Results

Despite the monitoring being installed immediately before the spring equinox, no gales occurred for many months. However, the gauges did show a consistent diurnal tilting of the mass correlated with the skewing of the joint and related to thermal plan bending of the bridge (Figure 12).

Further measurements

Having found a plan rotation of the bridge at the joint, it was desirable to measure the corresponding plan deflection of the main span. This was done using a laser light lever (Figure 13). A laser was set up at one end of the span with a slightly expanding beam directed along the length. At mid span an optical grating was placed in the beam and the diffraction pattern produced (principally a central dot) was focussed on a screen at the other end of the span (Figure 14). The dot then moved twice as far as the grating. A digital camera was set up to photograph the system every ten minutes. In this case we knew we were only looking for confirmation of diurnal bending. Two systems were set up, one close to each web. These showed that the vertical displacement of the webs was parallel and governed largely by the temperature of the deck. The sun on the webs bent the girder to the side by about 7mm (Figure 15) which correlates with the joint skew.

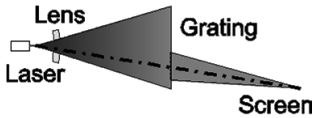


Figure 13 Laser light lever

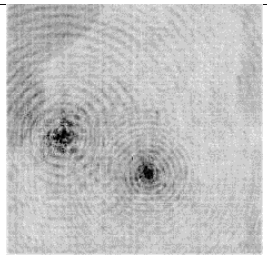


Figure 14 Laser image negative showing two measuring points

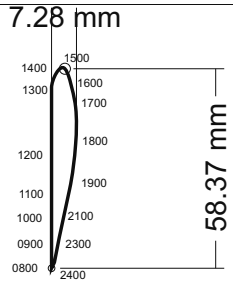


Figure 15 Mid span deflection trace with times

Understanding

To understand this behaviour, it is necessary to know a little more about the Tuned Mass Damper (TMD). In its basic form, a TMD involves a mass, a spring and a damper attached to a structure. In buildings these are often blocks mounted on rails with a horizontal spring and damper. The mass then sits with the spring at neutral until oscillations begin when the mass moves out of phase and damps the motion. In a bridge, where the motion is up and down, holding the spring at neutral tension is more difficult. At Cleddau (Figure 11 Damper support system), the 30tonne mass is suspended from the ends of the girder using 4 cables to each end. These cables have a very shallow sag but each pair also form a V in plan so that there is a stiff truss holding the mass in the straight line between the girder ends. Below the mass is a spring pantograph (Figure 10 Tuned Mass Damper showing pantograph and cables) which is stiff enough to pull the mass some way sideways as the girder bends in plan. The net result is a tilt of the mass.

When gales did arise, the bridge bent sideways with the wind. The mass did indeed rock as it bounced, even though it was a cloudy day with no lateral bending due to the sun.

Believing the data

When monitoring produces results which don't correlate with other aspects of behaviour, the results must be tested thoroughly. In the cases described above, there was enough redundancy of measurement and correlation between different measurements, to give confirmation that the data were correct. There is, though, still some reluctance at the bridge owner's office to believing that the rollers are jamming enough to resist expansion of the bridge. It will probably be necessary to add further measurements to produce convincing arguments.

On the other hand, if the measurements do not agree with established wisdom, it can be hard to persuade a client to believe the data.

A masonry viaduct

A brick masonry viaduct with 85 spans of 9m was showing considerable signs of distress. In some spans there were the common cracks in the arch under the inside edge of the spandrel walls (Figure 16) and in others there was apparent ring separation (Figure 17).



Figure 16 Crack through arch inside spandrel wall

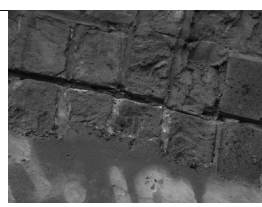


Figure 17 Ring Crack



Figure 18 Closed



Figure 19 Open

A Moire Tell Tale (Figure 18) was attached across the ring separation crack and filmed as a train was passing. The relative movement was small (about 0.5mm Figure 18) but was entirely radial. This shows that the movement is caused by the arch being pushed down by load not by shear displacement in the joint. To prove the point, cores were drilled in from the edge of the ring and up through the ring about 1m in from the edge. These cores showed that the crack stepped up through the rings to the extrados of the arch in about 0.7m. The rings were not separated. Figure 20 shows how the movement took place. Despite the rigour of these measurements, the engineers concerned were so determined on the diagnosis that they argued for stitching of the rings and a contract was awarded at a cost of £25k. The result was merely to add damage to an already damaged bridge.

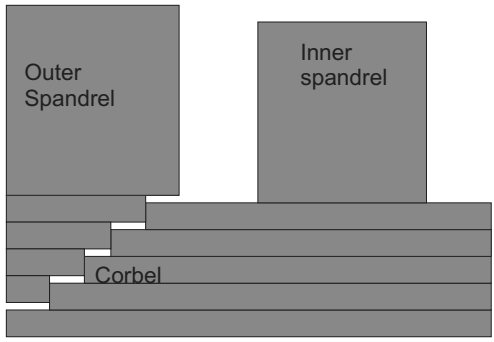


Figure 20 Corbelling edge of arch

Conclusions

Monitoring can be very useful but it is vital that the engineer understands what can be measured and what is being measured.

Redundancy of measurement is vital if there is to be any confidence in the results.

Careful monitoring will often reveal behaviour that is quite unexpected.

If proper complementary measurement confirms the behaviour the results must be believed and acted upon.

[i] Measurement computing, http://www.mccdaq.com/usb-data-acquisition/USB-7202.aspx

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ISBN 978-0-7277-5822-4

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doi: 10.1680/feng.58224.137

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Tsunami Casualty Mitigation through Vertical Evacuation Buildings: A Case Study from the March 11th 2011 Tohoku Earthquake

Katsuichiro Goda, University of Bristol, Bristol, United Kingdom
Ian Dasiewicz, University of Bristol, Bristol, United Kingdom
Shiliang Li, University of Bristol, Bristol, United Kingdom
Michael Quinn, University of Bristol, Bristol, United Kingdom
Jinyuan Zhang, University of Bristol, Bristol, United Kingdom

Introduction

A catastrophic M_w9.0 earthquake and the subsequent giant tsunami struck the Tohoku region of Japan on March 11th 2011. It caused tremendous casualties, massive damage to structures and infrastructure, and huge economic loss. The total economic loss has reached 300-400 billion U.S. dollars (Daniell and Vervaeck 2012). This event has revealed weakness and vulnerability of urban cities and modern society in Japan, which was thought to be one of the most earthquake-prepared nations in the world. In particular, the number of fatalities caused by this event (exceeding 19,000 deaths/missing) is high in light of the resources and efforts invested prior to the event.

To gain useful lessons from this tragic event, an Earthquake Engineering Field Investigation Team (EEFIT) was organised and dispatched to the Tohoku region of Japan under the auspices of the Institution of Structural Engineers. Many useful observations regarding tsunami damage were made for risk mitigation in the future (Fraser et al. 2013; Goda et al. 2013). One of the urgent needs that were revealed by this event is to develop an integrated risk management framework by combining hard and soft measures for reducing earthquake/tsunami risk effectively. The hard measures include coastal defence structures (revetment, dike, breakwater, seawall, etc.), tsunami-resistant buildings, and warning systems, whereas evacuation training and planning, such as evacuation routes and hazard maps, are essential for mitigating the consequences due to ground shaking and tsunami.

This study reviews tsunami damage to buildings during the Tohoku earthquake and then presents a case study to plan/design vertical evacuation buildings in coastal plain areas. It is noteworthy that some cities in the Tohoku region were more equipped with vertical evacuation buildings than others. For instance, in Kesennuma, about 3000 people were saved in 15 designated evacuation buildings, whereas Ishinomaki had 3 evacuation buildings only, although Ishinomaki had greater population than Kesennuma. The lack of such evacuation buildings and unexpectedly high tsunami wave resulted in high fatality rates in many coastal cities and towns (Fraser et al. 2013). The proposed approach aims at mitigating tsunami-related casualties by emphasising the 'systems' aspect of vertical evacuation

buildings (i.e. such buildings must be planned and prepared as a system in a community, rather than individual buildings). As a case study, the Town of Yamamoto, which is located in the southern part of the Sendai plains, is focused on. In Yamamoto, the total number of fatalities due to the tsunami was 676; the fatality rate in the inundated area was about 7.7%. It is noted that during the 2011 tsunami event, only about 90 people survived by evacuating to a shelter building. This is significantly less than other neighbouring municipalities; for instance, in Watari and Natori, about 2100 and 3250 people, respectively, evacuated and survived. Therefore, there is an urgent need to prepare physical infrastructure for tsunami vertical evacuation. The designed structures will have key desirable features for tsunami emergency, including open ground level, pile foundation, and emergency equipment (power generator, light/sign, radio, etc.), and must meet the need from local communities for normal use. For structural design of individual buildings, guidelines proposed by FEMA (2008) are followed. The designed system of vertical evacuation buildings in Yamamoto can provide sufficient capacity to accommodate evacuees under various scenarios (e.g. different tsunami heights and evacuation speeds) similar to the March 11th event. Importantly, construction of vertical evacuation buildings in Yamamoto can be justified from a cost-benefit perspective and thus becomes a viable option for tsunami risk mitigation.

Tsunami damage observations

The 2011 Tohoku earthquake generated massive tsunami waves in the Tohoku region due to coseismic ruptures of multiple fault segments along the Japan Trench. Figure 1 shows a map of the Tohoku region together with an estimated fault rupture area of 500 km long and 200 km wide (Shao et al. 2011); the estimated slip near the Japan Trench was about 30-40 m. In the figure, the run-up height, which is the maximum tsunami height above the tidal level at the time of tsunami arrival, is also included (Mori et al. 2011; The 2011 Tohoku Earthquake Tsunami Joint Survey Group, http://www.coastal.jp/tsunami2011/). The maximum run-up height of 39.75 m was observed at a district in Miyako, Iwate prefecture; tsunami inundation height near shore exceeded 10-15 m high at several locations, causing tremendous damage to buildings and infrastructure.

An important feature in understanding tsunami damage along the coastal line of the Tohoku region is that there are two different forms of coastal line: rias (northern Miyagi and Iwate prefectures) and coastal plains (central/southern Miyagi, Fukushima, and Ibaraki prefectures). The rias are submerged river valleys and have a jagged-shape; when tsunami approaches, the narrowing valleys near shore amplify the tsunami height significantly, thus resulting in greater run-up and inundation height. On the other hand, in the coastal plains, as there are not many obstructions near shore and the land profile is flat, tsunami height is not significantly amplified but tsunami inundates far in-land horizontally. Such differences of the coastal characteristics can be observed in the run-up data shown in Figure 1, and lead to very different tsunami damage patterns (see photos in Figure 1).

To illustrate tsunami damage features due to the March 11th event, several photos are shown in Figure 1.

- The photo in Taro shows a catastrophic failure of the famous seawall “The Great Wall”; it consisted of two layers of concrete walls with about 10-11 m high

above mean sea level (the total length of the two wall layers was about 2430 m). The near-shore (outer) layer was almost completely destroyed by the tsunami; only part of the wall around the gates remained. Tsunami wave overtopped the inner seawall and destroyed the community; the total number of fatalities due to the tsunami in Taro was 200 (including the missing). The interviews from survivors indicated that the existence of massive tsunami defence infrastructure gave a false sense of security and might have resulted in delay of evacuation.

- The photo in Minamisanriku, located in a rias coastal region, shows a devastated 3-storey steel building that was used as the Crisis Management Department headquarter. It was located about 500 m from the shoreline and was inundated by a 10 m tsunami. In this building, 30 staff were trapped when the tsunami arrived and unfortunately 20 of those lost their lives. Tremendous tsunami damage occurred elsewhere in Minamisanriku, and several reinforced concrete (RC) buildings were destroyed (in addition to numerous wooden houses). One of the vertical evacuation buildings survived; however, it suffered from significant foundation damage due to scouring (Fraser et al. 2013). The death toll (including the missing) in Minamisanriku amounted to 875.

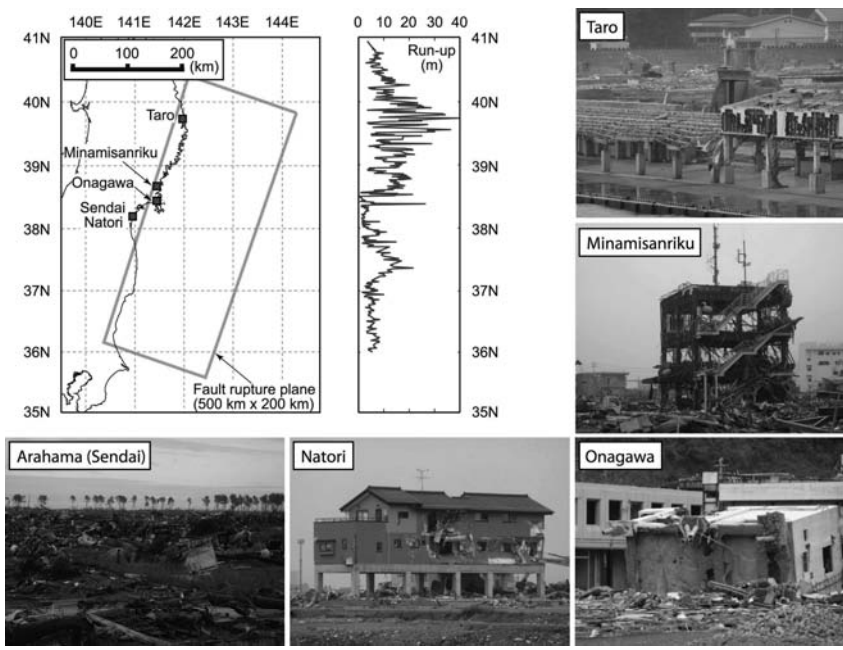


Figure 1: Map of Japan, run-up data, and tsunami damage photos at several cities.

- The photo in Onagawa shows an overturned 2-storey RC shear-wall structure with pile foundation. One of the most significant features of the tsunami wave in Onagawa was the very high tsunami inundation (up to about 20 m) with fast flow (estimated to be about 6-8 m/s; Fraser et al. 2013). This building was

subjected to various external loads including: strong shaking, liquefaction (which might cause damage to piles), and tsunami (hydrostatic and hydrodynamic forces, buoyant forces, etc.). The effects of scouring were evident at the building location. The shore front of Onagawa was destroyed completely, and the death toll amounted to 915.

- The photo in Arahama demonstrates devastation of local residential communities in the coastal plains. The tsunami height ranged from 5-10+ m, depending on locations, which was quite high for coastal plains but not as high as in rias regions. It is noted that the inundation height exceeding about 2 m is critical for typical residential wooden houses and the chance of being washed away increases significantly (Shuto, 1993). The photo in Natori shows an elevated residential house, noting that there are not many houses surrounding this house. The construction of a house with an open ground floor mitigated tsunami damage significantly. Another noticeable feature from the same photo is the impact of debris, such as car, ship, and lumber. Other typical damage features in the coastal plains include deep scouring of foundations (note: sand is a typical material near the ground surface).

The above damage cases are specific and are not a comprehensive list of actual tsunami damage. Nevertheless, they provide useful information on effective mitigation measures against tsunami damage. For more information, interested readers should consult Fraser et al. (2013) and references therein.

Tsunami evacuation in Yamamoto

This section presents a case study for planning and designing a system of vertical evacuation buildings in the Town of Yamamoto, where many lives were lost during the 2011 tsunami event. The main objective is to enhance tsunami life safety for residents in Yamamoto. Firstly, demographic characteristics of Yamamoto are summarised. Subsequently, tsunami inundation and observed damage feature during the 2011 event are discussed. Then, a system of vertical evacuation buildings, which are sufficient to provide shelter to all evacuees, is developed by taking into account local needs for normal occupancy. In addition, a simple cost-benefit analysis is conducted to evaluate the effectiveness of the tsunami evacuation buildings as life-saving intervention.

Demographic feature of Yamamoto

The Town of Yamamoto is located in the southernmost part of Miyagi prefecture. As of 2005, total population and population density were 17,713 and 275 people/km^2, respectively (the density is less than the national average of 336 people/km^2). The population had a high percentage of the elderly people (>65 years old) of 27.8% (see Figure 2), which was more than the national average (21%). This indicates that the mobility of evacuees is one of the critical factors in tsunami evacuation.

The town mainly consists of residential and farming areas, and thus the majority of buildings in Yamamoto were low-rise wooden structures. In Yamamoto, primary industry, typically farming and fishing, accounted for 20.7% (see Figure 2), which was significantly higher than the national average (4.8%). Many residents also work outside the town and commute to Sendai (about 35 km).

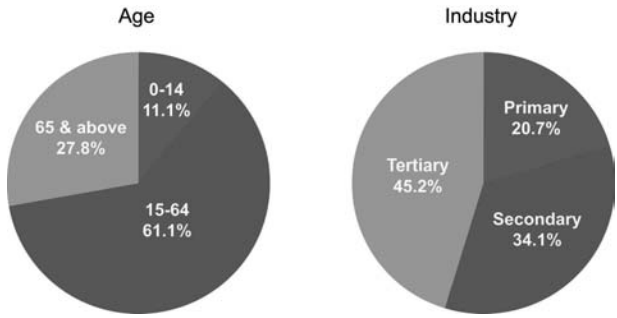


Figure 2: Age distribution and industry characteristics in Yamamoto.

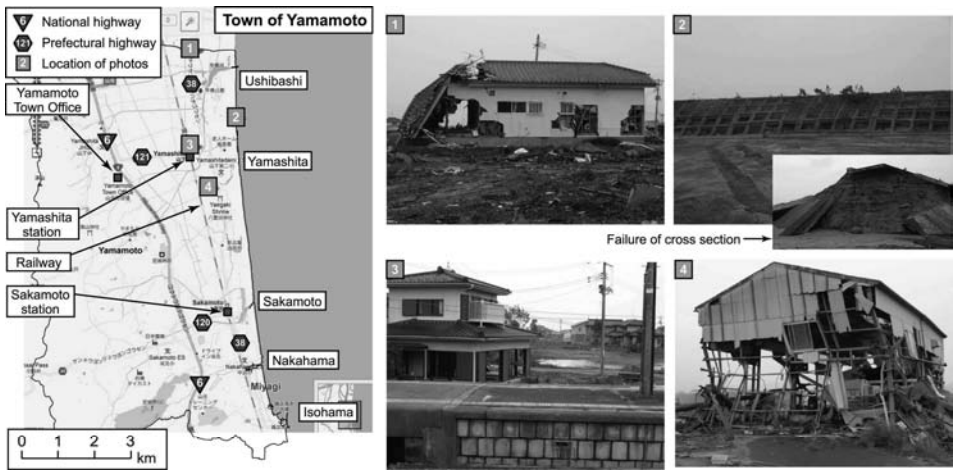


Figure 3: Map of Yamamoto, and tsunami damage at several locations.

Tsunami damage in Yamamoto

Yamamoto was severely damaged by the 2011 tsunami, which arrived about 65 minutes after the mainshock, and the number of fatalities was high (676 people). Because of the flat and low-lying topography, the tsunami inundated relatively far inland from the shore (about 2-3 km).

Figure 3 shows a map of Yamamoto together with several photos of damaged buildings and infrastructure in Yamamoto. Local communities, such as Nakahama and Yamashita, were relatively close to the coast. Consequently, many houses were washed away and destroyed by high tsunami waves (see photo 1); inundation heights of about 12 m and 2-4 m were observed at the shore and at 1-2 km inland from the shoreline (e.g. near Yamashita and Sakamoto stations), respectively (see Figure 4). Along the coastal line, there was a concrete block revetment (designed for storm surge); some part of the cross section was catastrophically failed and the lee side of the revetment was significantly damaged due to scouring (see photo 2). Near

the Yamashita station (photo 3), tsunami caused minor-to-moderate damage to buildings. Several agricultural warehouses (steel structures) were damaged by debris impact and damming effect (see photo 4). Otherwise, steel structures remained intact with external panel walls removed by the tsunami.

Tsunami evacuation planning

A high fatality in Yamamoto was caused by a combination of insufficient hard and soft mitigation measures against a large tsunami event. Historically (typically, 1894 and 1933 Sanriku tsunamis for design purposes), the tsunami risk was not recognised as imminent in the southern part of the Sendai plains; consequently, physical protection for large tsunami was not implemented. Public awareness for tsunami hazard in this region was low. For example, post-tsunami surveys conducted by Murakami et al. (2012) indicated that 87% of respondents in Natori, where serious tsunami damage was observed (similar to Yamamoto), thought "tsunami would cause only little or no damage", even after the occurrence of the M_w9.0 mainshock. The questionnaire results also indicated that only one third of the respondents took evacuation actions immediately after the earthquake, while the majority responded relatively slowly. In evacuation, about one third of the respondents travelled on foot or bicycle, while two thirds used cars; those who started evacuation late experienced traffic jam during the evacuation, resulting in further delay of evacuation to high ground. One of the critical aspects in evacuation using a car in this region was that the major highways run in the North-South direction (parallel to the coastal line), e.g. the prefectural highway 38 in Figure 3; less accessibility to roads in the East-West direction increased the chance of being caught by tsunami during the evacuation. These findings regarding the evacuees' behaviour in Natori are applicable to Yamamoto.

Moreover, it is important to secure evacuation routes both horizontally and vertically. Traffic signs to evacuation places at high elevation, local hazard maps, and education/training are key factors for successful evacuation. Residents who live relatively inland (e.g. about 2 km or more from the shoreline) can get out from tsunami hazard zones safely, while those who live near the shoreline need to evacuate to nearby vertical evacuation buildings. After the 2011 tsunami, the Government of Japan proposed a target policy "evacuation on foot within 5 minutes". Therefore, preparation/construction of vertical evacuation buildings is of critical importance. By assuming a mobility of 4 miles per hour for a healthy person (FEMA 2008), the 5-minute evacuation time corresponds to about 500 m, whereas for a mobility-impaired person (2 miles per hour), the distance needs to be reduced to about 250 m. Realistically, available evacuation time will be about 10-20 minutes (in addition, time to start evacuation and time to climb to an elevated level, e.g. 5 minutes for each action, need to be considered).

In this study, five vertical evacuation buildings are planned such that these buildings as a system can cover the majority of the residential communities near the shoreline. For this purpose, the digital geographical information database developed by the Geospatial Information Agency of Japan (http://www.gsi.go.jp/kiban/) is used. The locations of the five buildings are shown in Figure 4. The building height and floor area are determined based on the guidelines suggested by FEMA (2008). Specifically, the building height is determined to exceed the design tsunami height: maximum anticipated tsunami inundation height (which is taken as the inundation

height for the 2011 Tohoku tsunami; see Figure 4) plus freeboard (3 m and additional 30% of the inundation height). See Table 1 for the inundation height, design tsunami height, and building height. The floor area depends on the population within the primary coverage area (i.e. 500 m radius from a building), which is estimated by counting the number of houses and by considering 3 people per house. FEMA (2008) recommends that an area of 1 m^2 per person is required for short-term evacuation (up to about 24 hours). Furthermore, possible variation of the population, depending on the time of the day and considering a wider catchment area, is taken into account (note: evacuees from the extended catchment area require additional evacuation time, generally 10-20 minutes longer). The covered population for the primary area and the floor area of the building are summarised in Table 1.

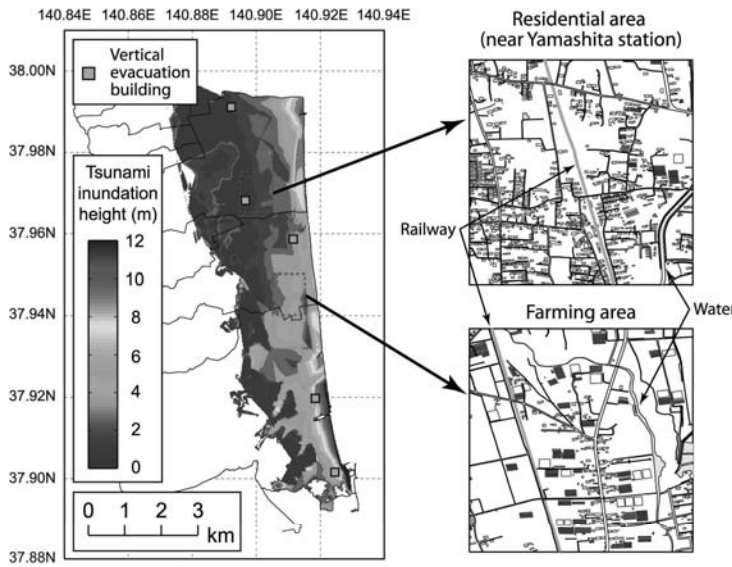


Figure 4: Locations of five vertical evacuation buildings.

The proposed vertical evacuation buildings are intended for multi-purpose occupancy. Therefore, local needs for services are taken into account. For the site 1, a care home for elderly people is considered; the extended catchment area includes beaches and parks near the coast. For the site 2, an elementary school is selected (note: there is no elementary school in this local area); the catchment area mostly covers residential communities near the Yamashita station. For the site 3, a sports and recreation centre is proposed, anticipating a local need for such a facility. For the site 4, a new post office in Nakahama, where the original one was completely destroyed due to the 2011 tsunami, will be constructed. Because of the high tsunami height at this location, a 5-storey RC building is considered; the upper floors may be occupied by local shops and restaurants. For the site 5, a newly-constructed fish processing plant is designated as a local vertical evacuation building to promote one of the main industries in Yamamoto; the ground floor can be used for processing and warehouse space, while the upper floors can be used for office space and for

evacuation purposes. The catchment area includes residential houses, shops, fishing harbour, and beaches.

Finally, the construction costs for these five vertical buildings are estimated using the 2011 Japan building cost information (CRI 2011). The total cost for the five vertical evacuation buildings is estimated as 47.49 million GBP (Table 1). By assuming that the majority of the fatalities (676 people) can be saved by utilising these buildings, a simple cost-benefit analysis is conducted, which is valuable in deciding whether investments for vertical evacuation buildings in Yamamoto are justified from an economic viewpoint. Suppose that the lives of 600 people can be saved, the incurred cost per saved life is about 80,000 GBP/life. In developed countries, such as Japan and U.K., a typical value assigned to a year of quality human life is around 20,000-30,000 GBP/year. The cost for the vertical evacuation buildings is significantly lower than the benefit (e.g. assuming 10-20 years of quality human life can be gained by saving a life), and thus the utilisation of vertical evacuation buildings should be considered as a viable option for tsunami casualty mitigation.

Table 1: Summary of five vertical evacuation buildings.

Evacuation building site & occupancy type	Latitude & longitude	Inundation depth[1] (m)	Design tsunami height[2] (m)	Building height (m) [number of storeys]	Covered population[3] [Floor area (m^2)]	Cost[4] (million GBP)
Site 1: Care home	[37.9911 & 140.8921]	1.95	5.54	14 [4]	1320 [2400]	18.10
Site 2: Elementary school	[37.9682 & 140.8966]	1.85	5.41	14 [4]	1030 [1500]	10.82
Site 3: Sports centre	[37.9587 & 140.9115]	4.09	8.32	14 [3]	780 [1000]	6.15
Site 4: Post office	[37.9197 & 140.9184]	10.49	16.64	17.5 [5]	740 [800]	6.89
Site 5: Fish processing plant	[37.9016 & 140.9245]	7.76	13.09	17.5 [5]	970 [1000]	5.53

[1]The inundation height is based on the observed inundation height in the 2011 tsunami event (Mori et al. 2011; see Figure 4); [2]The design tsunami height is calculated based on the FEMA's guidelines (2008); [3]The covered population is based on the number of households (3 people per household) within 500 m radius from the evacuation building; and [4]The construction cost includes all relevant costs (foundation, structure, finish, electric work, ventilation, etc.), and is estimated based on the 2011 Japan building cost information (CRI 2011).

Design of Yamashita Elementary School

To illustrate the structural design of a vertical evacuation building, the elementary school (site 2) is focused upon. The neighbourhood of the elementary school is shown in Figure 5, together with the boundaries representing the primary coverage area and the maximum catchment area. It covers residential communities near the Yamashita station. The evacuation (travel) time for the easternmost houses is about 20 minutes.

The elementary school is a RC structure. A sketch of the building is shown in Figure 5. Key features of the designed structure in terms of tsunami risk mitigation include:

(i) open ground space; (ii) use of breakaway walls, and (iii) access ramp to the top floor at the rear side of the building. The open ground floor reduces tsunami-induced forces acting on the structure (see the photo for Natori in Figure 1). The walls at the first and second floor levels are designed as breakaway walls; they ease the build-up of tsunami forces on the structure (particularly, hydrostatic and hydrodynamic forces). The walls are designed such that the wall will break away when the water pressure exceeds 0.0479 kN/m^2 (FEMA 2008).

In tsunami, the building is subjected to various loadings (FEMA 2008); for the design purpose, the design tsunami height of 7 m (i.e. ground and first floor levels are submerged) is considered (see Table 1). For the considered structure, one of the most critical forces is caused by debris impact (assuming that the breakaway wall performs as designed). The debris impact by a lumber or a small container is estimated to be about 800-1000 kN. Another significant tsunami-induced load is the damming force; it is estimated to be about 1250 kN. If the breakaway walls are not taken into account, both hydrostatic and hydrodynamic forces, acting at the first floor level, can be significant; they are about 1800-2000 kN, respectively (across the entire front wall).

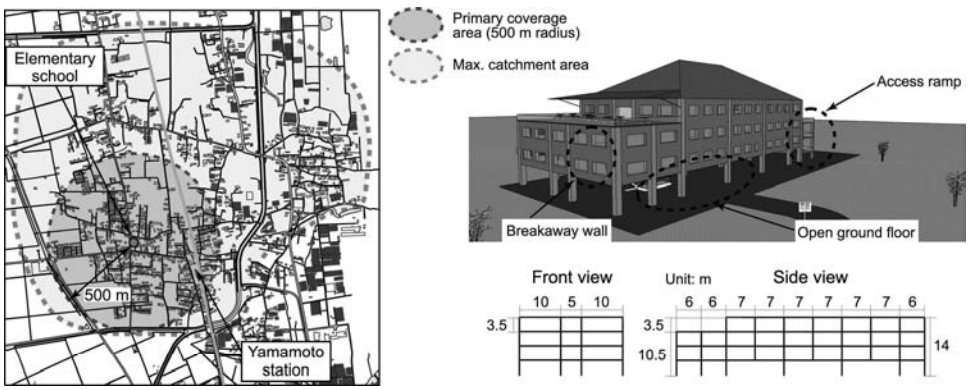


Figure 5: Location and sketch of the elementary school.

The structural components are designed according to the requirements based on Eurocode. The worst case scenario is used to determine sizing of columns and beams by assuming additional live load from 1000 evacuees and by accounting for various tsunami load combinations (to account for the worst case, effects of breakaway walls are ignored). A typical size for columns is a 0.8 m by 0.8 m square cross section with 12 reinforcement bars of 12 mm diameter around the column. The structural analysis indicates that one of the most critical loading situations is caused when the debris strikes the corner column of the building; to reserve higher structural capacity, the column size for ground and first floor levels (facing the tsunami flow) is increased to a 1 m by 1 m cross section with 20 reinforcing bars with 32 mm diameter. For beams, a cross section of 0.5 m by 0.5 m is selected, while for beams near the building front, the cross section size is increased to 0.7 m by 0.7 m (note: the arrangement of rebar depends on the location within the building). It is noted that the beam-column joints are detailed to achieve ductile behaviour and meet the

seismic provisions of the strong-column weak-beam principle to avoid vulnerable soft-storey collapse mechanism.

Finally, the foundation is designed by considering piles. For this site, it is considered that shallow foundations are not suitable based on damage observations during the 2011 tsunami event; at various locations, shallow foundations suffered from significant scouring (Fraser et al. 2013). Moreover, soils in the Sendai plains are susceptible to liquefaction; soils are sandy materials and their in-situ *N*-values from the standard penetration tests near the ground surface are relatively small (e.g. $N < 20$). For these reasons, pile foundations, reaching rock layers with sufficient bearing capacity, are necessary (typically, depth to the rock layers is about 10 m). To provide sufficient vertical support, 6 RC piles, each with 0.4 m diameter and spaced at 1.0 m interval, per column location are embedded into the ground (in total 180 piles).

Summary and conclusions

The March 11th 2011 catastrophic tsunami caused tremendous tangible and intangible loss to many communities in the Tohoku region of Japan. In particular, fatality risks at coastal cities and towns were exceptionally high and must be reduced significantly. To achieve safer and resilient society against natural disasters, observed tsunami damage features are reviewed; key lessons to be learned from this event include (focusing on low-rise structures in the coastal plains): open ground floor to ease tsunami-induced forces, significant impact force due to water-borne debris, and extensive effects due to scouring, which impairs foundation stability.

A case study was conducted by planning and designing a system of five vertical evacuation buildings in Yamamoto. Locations of the buildings were selected based on the demographic characteristics of existing local communities; the height and size for each building were determined by the anticipated tsunami inundation height at the site (plus additional safety margin) and the primary coverage area for tsunami evacuation. The occupancy for normal situations was based on the local needs for community services. The structural design of these buildings against tsunami loading was carried out by referring to guidelines given by FEMA (2008). Moreover, a simple cost-benefit analysis was conducted to investigate the cost-effectiveness of such life-saving intervention for tsunami risk mitigation; it was found that the cost for vertical evacuation buildings in Yamamoto can be justified economically. Most importantly, the success of such physical infrastructure depends on proper management of the facilities (e.g. equipments for emergency) and continuous efforts for disaster education and training within local communities. In fact, public awareness about significant tsunami risk and prompt evacuation action during the disaster are the most critical factors.

Acknowledgements

The first author thanks the EEFIT members (Fraser et al. 2013; Goda et al. 2013) for sharing their valuable observations and experiences during and after the mission. The participation of the EEFIT-Tohoku mission by the first author was funded by EPSRC (EP/I01778X/1).

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Forensic Engineering: Informing the Future with Lessons from the Past
ISBN 978-0-7277-5822-4

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doi: 10.1680/feng.58224.148

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Bond Street Station Upgrade: the replacement of escalator 1 and 2

Paul Perry, BSc(Hons) CEng MICE MIStructE MHKIE, Halcrow Atkins, London, UK

Introduction

As part of the Bond Street Station Upgrade, two new escalators have been replaced within an existing tunnel below the existing station building and commercial development above, known to those using Oxford Street in London as West One Shopping Centre.

This 21st century keyhole surgery needed extensive investigation of the existing tunnel and station structure as well as a thorough understanding of two sewers that have been encapsulated within the tunnel lining.

Following the award of a design and build contract to upgrade Bond Street Station by London Underground to Costain Laing O'Rourke Joint Venture on August 2010, Halcrow Atkins were appointed as a design alliance to provide multi-disciplinary consulting engineering services for the entire upgrade scheme, of which this was an early element of the Works. The intention was that the initial design was developed in detail and once approved, formed the basis of construction.

From a comprehensive archive search of drawings and photographs of the station and tunnel over the centuries, a conceptual design was formulated, which given time, with good engineering judgement and assessment, enabled a scheme to emerge.

Detailed design and construction were completed to allow the new escalators to be commissioned and brought into service in May 2012 ahead of London 2012, the Olympic Games.

The existing station

Bond Street Station is a London Underground station situated in Westminster, first opened in 1900, was one of the first stations to receive escalators in 1925/26 (Martin, 2012). This was associated with a policy during the twenties in modernizing stations and, in particular, installing escalators to replace lifts where passenger usage justified it (Barker, 1974: 249). Since then, it was subjected to various alterations including the most significant in 1979 when it was integrated into the project to construct the Jubilee Line and in 1981 when the West One Shopping Centre was completed.

The station is on the Central and Jubilee lines with the Central Line platforms some 21m below the street level and the Jubilee constructed lower, at some 30m below

street level. The station ticket hall is located at a shallow depth, with part below Oxford Street and part in the basement below the West One Shopping Centre.

From this station ticket hall, two upper escalator inclined tunnels were constructed, the original at the west side contains escalator 1 and 2 and the east side containing escalators 3, 4 and 5. Both of these tunnels connect to an intermediate concourse level connected to the passageways to the Central Line as well as at the head of a set of three lower escalators which connect to the lower Jubilee Line platforms.

A number of public sewers were altered and re-routed and pass through the station although the Middle Level Sewer has remained in its original position over the years, passing through the invert of the inclined barrel of the escalator 1and 2 tunnel. When the inclined escalator tunnel was constructed, the typical cast iron segments were omitted from the invert at the location of the sewer and a bespoke bolted cast iron segmental saddle was constructed over the sewer. The saddle was embedded in the concrete maintenance steps passing over it and the sewer. A second sewer, the King's Scholar Pond Sewer, was diverted during the station construction and is located below the floor of the upper machine chamber. This was not affected by the replacement of the escalators under the Bond Street Station Upgrade.

The proposed upgrade

Bond Street Station is soon to be an important part of the Crossrail Project railway running through the heart of London, with a new station concourse being constructed to the south of the original station.

At the same time as the Works for Crossrail are advanced, London Underground are progressing a separate project to upgrade the station. The upgrade has been planned to alleviate congestion at the station and through a number of major enhancements such as new tunnels, cross passages, a new northern satellite ticket hall and replaced escalators, is set to ensure the station survives for some considerable time hence and maintain the ambience of quality that station operators seek (Edwards, 1997: 121).

The proposed works to replace escalator 1 and 2 with Kone escalators undertaken in advance of the main upgrade works to allow completion for the Olympic Games held in London in the Summer of 2012. These advanced works included:

- Enabling works (cable protection & replacement)
- Strip out of the existing escalator mechanism
- Dismantling of the existing escalator trusses
- Removal and reconstruction of both the public level / ticket hall floors above the upper and lower machine chambers
- Part removal of the invert concrete in the inclined escalator barrel, over the sewer saddle
- Provision of steps for maintenance along the invert of the escalator tunnel and over the saddle of the Middle Level Sewer
- Provision of new support plinths and columns within the upper machine chamber
- Breaking out of existing concrete plinths in the upper and lower machine chambers

- Provision of protection measures, if required, for the Middle Level
- Replacement of the existing steep ladder access to the upper machine chamber
- Provision of a new floor access hatch and ladder into the lower machine chamber.

Existing Records

Drawings from London Underground's records are normally extensive and thorough and with the support from London Transport Museum, are important when commencing such a project. The finding of a existing specific record has been most helpful in confirming the design and has shaped the course of events (Perry, 2009). They established the detail and when read in conjunction with 1920's progress photographs taken from the London Transport Museum's extensive records. This confirmed the details of the new escalators being constructed within the existing basement of the then Grosvenor Hotel in 1925, with the inclined tunnel being excavated below Oxford Street and the existing buildings opposite into the Central London Railway tunnels and Bond Street Station below. These records showed that the escalators came into service on 8 June 1926. It was recorded at the time, as an annotation of one of the 1925 photographs that "the steel column and girders have been put in to support the front of the Grosvenor Hotel. The old brick piers that used to carry the hotel can be seen resting on the top of the girders. The hotel is 100ft high and the weight of the front facade which is now carried by the girders is about 300 tons" (Figure 1).

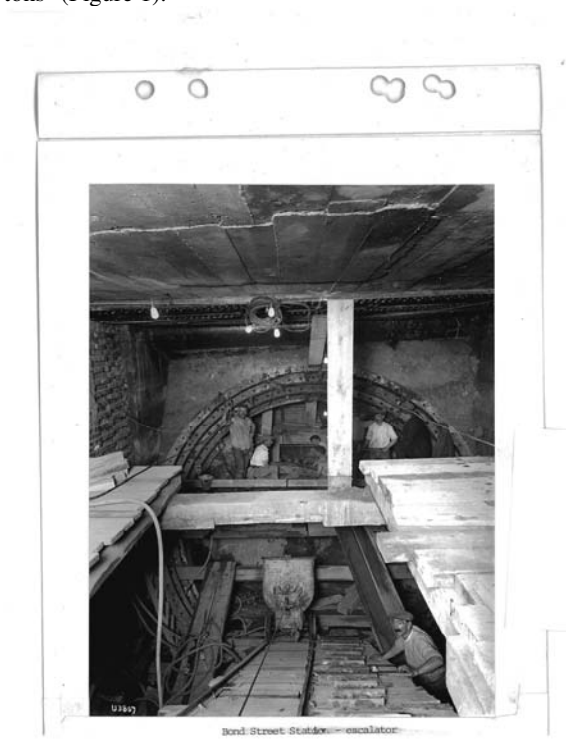


Figure 1 October 1925 Bond Street Station Escalator Construction.

Further record drawings from London Underground archives and from Ove Arup and Partners Library and Archives, ensured that a clear understanding was had of the works completed for the Fleet Line (now Jubilee Line) at Bond Street Station and the West One Shopping Centre undertaken in 1972 to 1981.

Part of the original drawing shows a cross section of the cast iron tunnel lining, manufactured to a bespoke section to accommodate an existing sewer passing within the existing tunnel soffit, was instrumental in determining the scheme to be developed. Figure 2a. The detail of this, drawn up, is shown in Figure 2b.

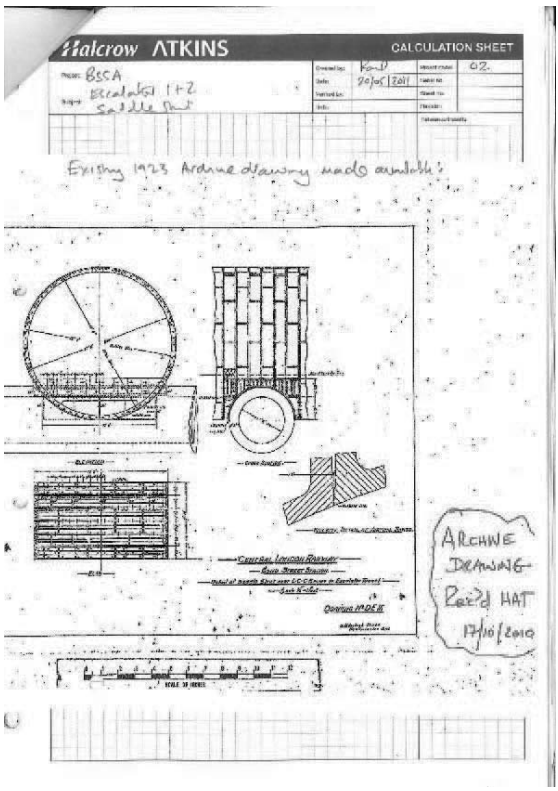


Figure 2a Archive drawing showing the “saddle strut”

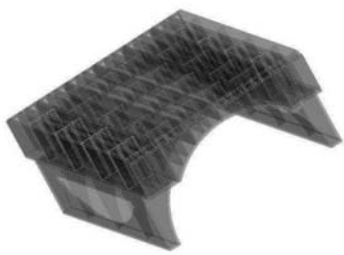


Figure 2b Redrawn "saddle strut" for incorporation in the design

The upper machine chamber roof / public level floor consists of a variety of suspended floor construction supported from a grillage of steel beams supported on steel columns and a concrete perimeter wall. The upgrade scheme required an area of this to be rebuilt. Records of previous investigations and structural assessments were made available, which proved most helpful when considering modification to this suspended construction. The existing construction consisted of a mix of suspended construction of 1925, 9 inch (and some 7 inch) deep filler joist with a 9 inch deep (assumed in previous assessments as un-reinforced) concrete infill slab between supported from a variety of 1925 Rolled Steel Joists, retained alongside the 1979, 310mm deep reinforced concrete slab on Grade 43 steel universal beams.

The Middle Level Sewer with the bespoke bolted cast iron segmental saddle located at the point where it passed through the invert of the escalator barrel required particular consideration during detailed design and construction. Included in the records made available by London Underground was the 1925 Drawing of the tunnel rings and bespoke saddle element. Also included was an Outline Design Report of the replacement of the escalators with a description of a visual inspection undertaken by Future Amtec for Thames Water in October 2008 of the sewer and an assessment of this noting the risks associated with unloading the sewer when the invert concrete was removed in the barrel coinciding with an event of high water pressure within the sewer. Clearly any effect on the structural integrity of the saddle would be catastrophic with regards failure of such leading to flooding of the station tunnels below. The report described the analysis that supported the assessment and the conclusion that if the saddle exists with the bolts in place and in good condition, there would be sufficient resistance to withstand the water pressure and prevent any failure. This conclusion was based on follow up survey, monitoring and detailed design being undertaken during the construction.

Engineering Assessment

A key constraint to the works was the Middle Level Sewer which as it predated the construction of Bond Street Station, when the escalator tunnel was constructed to cross its path, it was decided to omit some typical grey cast iron segments at the invert of the escalator tunnel and provide a bespoke bolted cast iron segmental "saddle strut" to allow this to bridge the existing sewer below. The integrity of this arrangement had to be maintained, especially when the escalator trusses, the longitudinal support walls and the concrete steps were removed and the arrangement unloaded in the temporary condition.

The importance of confirming the existing tunnel structure at the "saddle strut" was crucial in terms if ensuring that it would resist a full head of water pressure under a surcharged sewer and that when the alterations works were undertaken, this function was maintained and ensured that flooding of the station arising from failure of this arrangement was avoided. For the analytical assessment, the Middle Level Sewer was shown as 5'6" (1.7m) internal diameter subject to occasional water surcharge pressure of approximately 11.4m head of water (assuming sewers surcharge to ground level of Oxford Street, 122.00 LUmD in an extreme storm event).

From a review of the archive drawings and photographs, whilst the bespoke bolted cast iron segmental "saddle strut" had been shown in detail, the connection detail to

the segment lining had not. In the engineering assessment, it was assumed that this would be a "pin", ie not transfer moment and hence give a greater bending moment requirement at the centre of the tunnel. The capacity for the "saddle strut" was checked against this worst bending moment and the temporary propping and monitoring instrumentation determined.

Initially the area of existing hearting concrete below existing escalator trusses, was all that could be seen of this particular section. It was therefore decided to undertaken investigation to open up and expose the connection of the "saddle strut" with the tunnel lining ahead of demolition and removal of the existing escalators. The preliminary drawings showed four locations on top of the "saddle strut" (IP1 to IP4 inclusive) where this would be undertaken in a deliberate sequence of sectional removal of the trusses and temporary propping of the "saddle strut" until the actual condition was known.

Material Properties

Material properties of the structural section were obtained to verify assumptions made in the engineering assessment. Structural investigation of the existing concrete, the cast iron tunnel linings, the structural steelwork beams and the cast iron bespoke saddle over the sewer, meant a detailed understanding of the existing was able to limit the risk of the unknown and assist with construction proceeding.

Materials consulting engineers Sandberg were engaged to examine and test the existing structural steelwork in the upper and lower machine chambers and ferroscan existing hearting concrete to determine the location of reinforcement. Costain's in-house testing laboratory were engaged to undertake non destructive testing of existing concrete within corbels built-in to the tunnel lining.

London Underground's Standard 1-053 Revision A3 (London Underground, 2010) gives a warning re pre 1920's steel and as Bond Street Station first opened on 24 September 1900 with the steelwork to support the hotel for the new escalators following in 1925, it was deemed prudent to test the steelwork for material properties for use in the design concluding with the use of a characteristic strength of 230N/mm2 in accordance with clause 3.4.11.3 of the LU standard.

In the assessment of the "saddle strut" over the sewer, material properties associated with G1O cast iron taken from BCSA Historical Steelwork Guidance Note (Bates, 1984) were used. This gave a permissible compressive stress of 150N/mm2 and a permissible tensile stress of 38N/mm2.

100mm diameter cores were taken in the invert of the tunnel, within the tunnel ring segments, to obtain the in-situ concrete strength and reduce the amount of breakout of existing concrete. The results gave an estimated in-situ cube strength of 28.5N/mm2 to 48.5N/mm2, using a modification factor of 0.95. This confirmed that a value of 25N/mm2 taken in the engineering assessment was prudent and allowed this invert concrete to be retained giving benefits of reduced scope, shortened programme and reduction in risk of damaging the existing tunnel lining during demolition.

Design

The scheme to replace the existing No 1 and 2 escalators by Kone escalators meant that the new escalators had different equipment layouts below and therefore floor plates and floor tray locations above to that of the existing. This meant that demolition and reconstruction of the suspended floor construction at the public circulation levels above the machine chambers at the top and bottom of the escalators was necessary. At the bottom, the suspended reinforced concrete floor was detailed with a construction joint and couplers to allow it to be constructed in two distinct sections, associated with the phasing of the hoarding and pedestrian flow through the passageways at this level.

It also meant that the equipment support for the new escalators, in the machine chambers and along the inclined tunnel, would require demolition of the existing plinths and support walls and columns and reconstruction of a new array. The works within the escalator tunnel involved the part retention of the concrete within the invert of the existing escalator tunnel soffit, with new reinforced concrete plinths supporting the escalator trusses and attached via drill and grout reinforcement between new and retained concrete.

In addition to this, the scheme was required to improve the existing steep ladder access from public circulation level into the upper machine chamber.

Within the existing escalator barrel it was intended to improve the maintenance access arrangement on the incline, between the escalator trusses. This involved a re-profiled steps and hand holds / rails.

An important requirement stipulated by London Underground was that the design life for all new civil works was 120 years. The characteristic loading used for the design of the permanent structure were prescribed by the appropriate London Underground standard.

The constraints imposed by the existing tunnel and structures are such that no other structural forms were considered as appropriate. Lifting capacity within the existing inclined escalator tunnel has limited the use of sizeable structural steelwork and pre-cast reinforced concrete elements. The design was drawn up in a three dimensional model to ensure integration into the existing tunnel and across the architectural, mechanical, electrical, public health and civil disciplines. Extracts of the 3D Model showing the invert concrete and existing saddle strut are shown below.



Figure 3 Three Dimensional Model showing the invert concrete and “saddle strut”

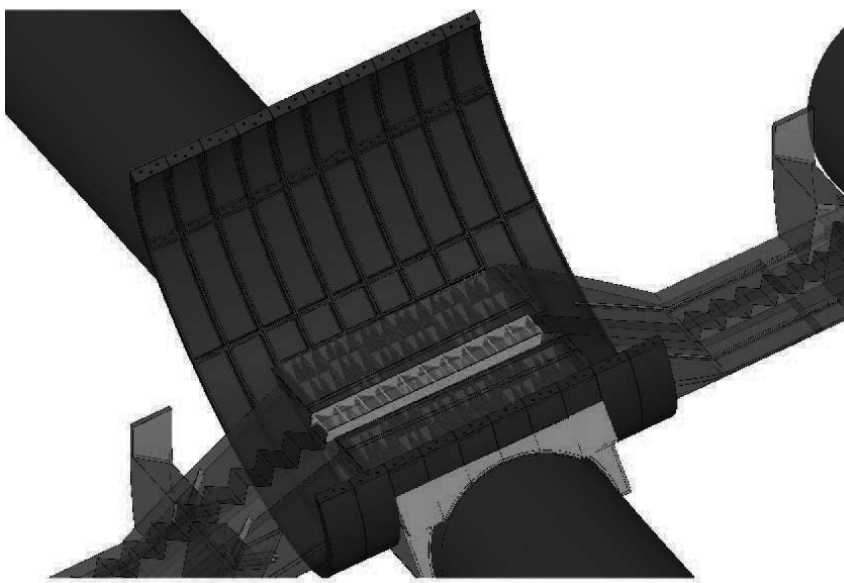


Figure 4 Detail taken from Three Dimensional Model showing "saddle strut", sewer and existing tunnel lining incorporated within proposed scheme to replace escalators

Construction

Following installation of temporary propping, monitoring of water flow within the sewer and review of existing in-tunnel instrumentation, a steady state was achieved. Propping and sequential demolition and removal of the old escalator trusses, gradually exposed the tunnel construction.

Initially the existing escalator trusses and support walls were removed from the west side, followed by the existing escalator trusses and support walls on the east side. The engineer's envisaged method assumed that, in the first instance, the escalator truss and existing longitudinal support wall removal would be taken to within 5m of the centre line of the existing "saddle strut" over the sewer. Temporary propping (Figure 5) was installed at the saddle strut to ensure that additional support was given to the tunnel segments reliant on the saddle strut to complete the circular action.

As the "saddle strut" was approached, an intrusive investigation was undertaken to open up and expose the top surface of the web to the units that made up the saddle.



Figure 5 View looking down the inclined tunnel (ie North) with temporary propping in place with removal of the old escalator truss in progress (on the East).



Figure 6 View looking down the inclined tunnel (ie North) with temporary propping in place with removal of old escalator longitudinal support walls in progress (on the West).

The intrusive investigation undertaken by the contractor was to open up the location for investigation at IP3 and IP4 first, ie the west side of the tunnel to "saddle strut" connection and this was inspected (Figure 7).

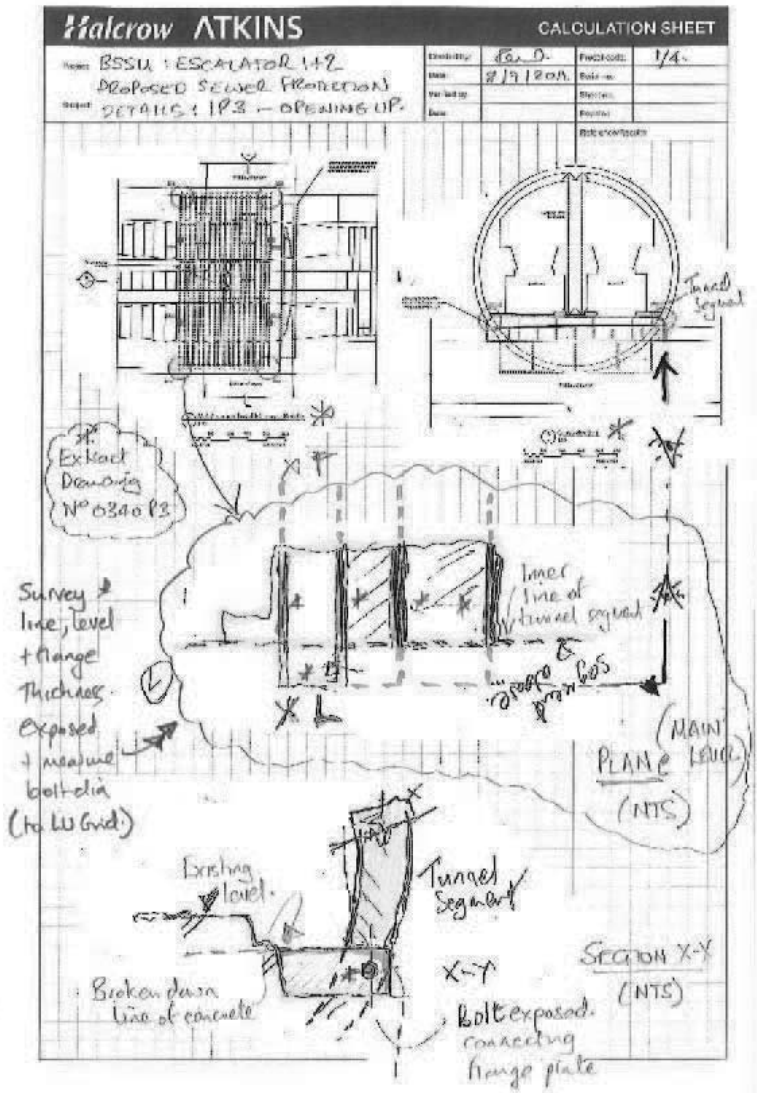


Figure 7 Sketch notes from the inspection of opening up the connection between the bolted connection of the “saddle strut” to the tunnel ring

The exposed flanges of the saddle strut appeared to be in good condition and the saddle strut section shown to be below the tunnel segment. There was no sign of movement between the top of the saddle and the soffit of the tunnel segment, which would be evident from an open joint. Existing concrete had been removed to a sufficient depth that a bolt was exposed showing that the "saddle strut" was located within, and bolted to, the flange plate of the tunnel lining. The exposure of the connection gave confidence to the assumption in the engineering assessment and

confirmed the structural competency of the arrangement that had been modelled and drawn in January 2011. (Figure 8).

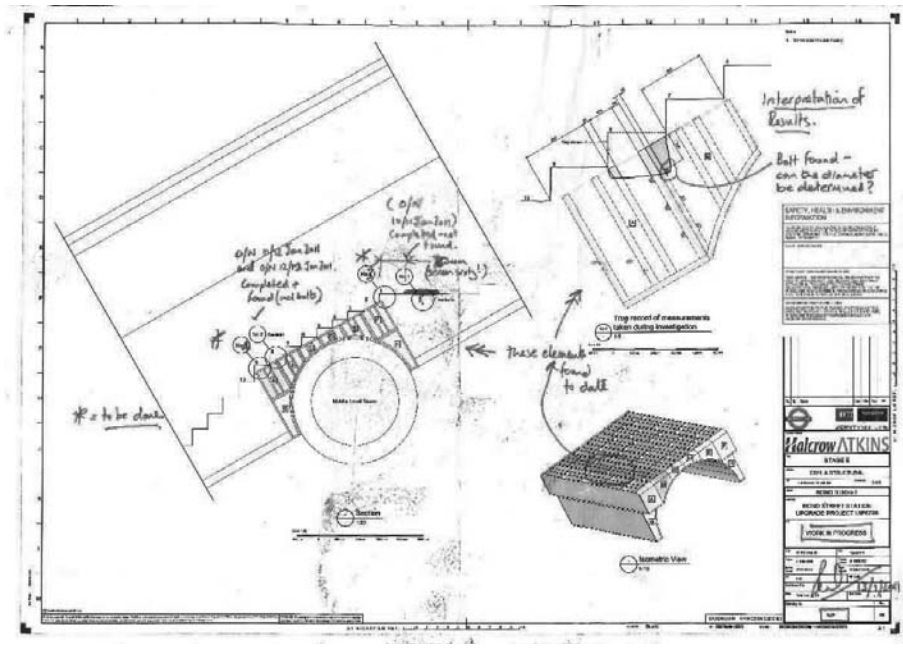


Figure 8 “work-in-progress” drawing recording the results

The remaining locations on the east side of the tunnel at IP1 and IP2 were opened up second, inspected, and similarly noted that the exposed flanges of the "saddle strut" were in good condition, that there was no sign of movement between the top of the "saddle strut" and the tunnel lining and the bolts between "saddle strut" and tunnel lining flange were shown to be present.

The results of the investigation had proved the assumptions made in the engineering assessment and allowed the work of removal of the remaining sections of the old escalator trusses to continue.



Figure 9 View looking up the inclined tunnel (ie South) with removal of old escalator truss and longitudinal support walls complete and "saddle strut" evident.

Civil works progressed to allow renewal of the lower machine chamber and reconstruction of the public suspended floor. From the lower machine chamber the incline concrete was poured until the upper machine chamber below Oxford Street and the station and the shopping centre was reached. Here reconstruction of the upper machine chamber and the suspended public floor construction, was achieved.

Subsequent installation of the new escalators and their equipment, the follow on builders-work and fit out and finishes has been programmed to achieve a power-on and handover to London Underground ahead of a predicted in-flow of visitors to the London 2012 Olympic Games.

Instrumentation and Monitoring

An array of instrumentation was determined from objectives developed from the design (ICE, 2011:9) and was installed within the existing tunnel to monitor the tunnel during the course of the works. These included automatically read targets within the tunnel crown along the inclined alignment including at the location of the "saddle strut", electro-levels, strain gauges and manual level monitoring of specific locations on the "saddle strut". Water levels within both sewers were read by Thames Water and passed to London Underground. Prior to commencing the Works, "trigger" levels were agreed for states of Green (safe), Amber (warning) and Red (action required) and a "steady" state demonstrated discounting minor variations as being associated with temperature effects (Figure 10).

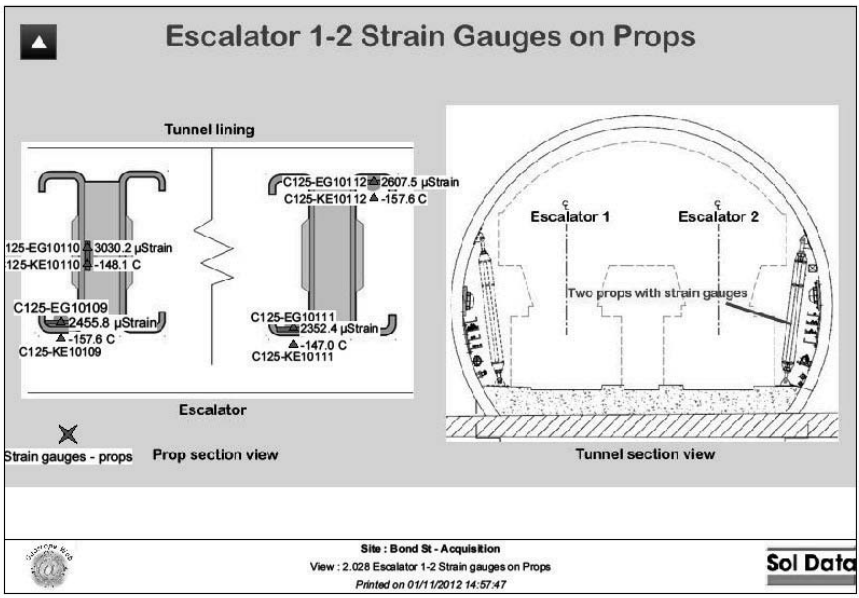


Figure 10 Strain Gauges on temporary props over "saddle strut"

The monitoring, readings (Figure 11) and assessment of such were the basis that temporary props located at the sewer saddle could be removed in September 2011.

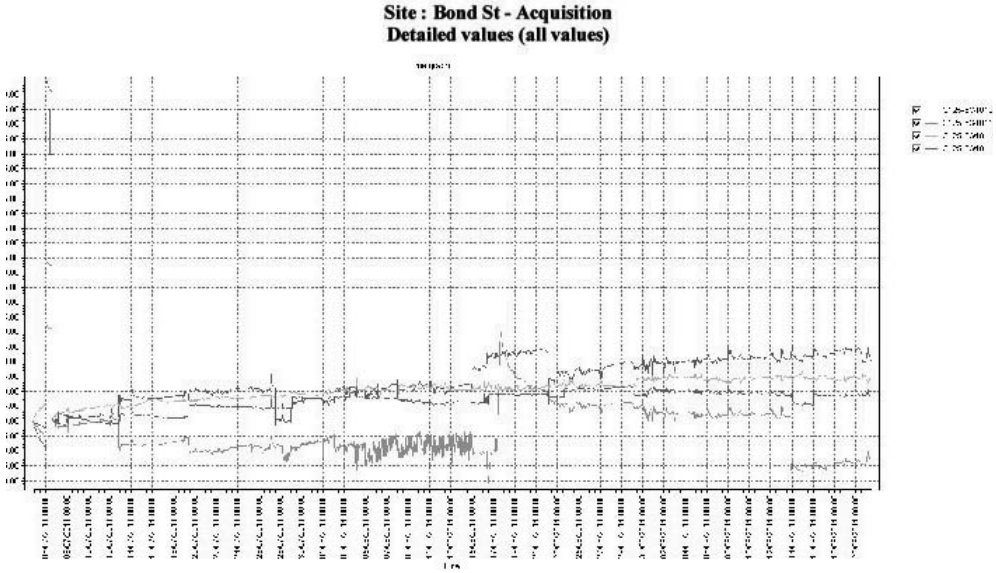


Figure 11 Readings from strain gauges on temporary props over "saddle strut"

Best practice

The client's team, the contractor's team and the design team were located in one office, immediately above the work site during both the design (RIBA Stage E & F) and the construction phase for this work. The New Engineering Contract (NEC) contract was operated in the "Design and Build" format, all allowing best practice to be achieved and creating value (ICE, 1996) with swift communication and problem solving in this unique environment.

Good archive records combined with diligent investigation to open up and record and thorough engineering assessment were important to the success of this work.

Engineering details were drawn up in a three dimensional model, with "slices" cut through this model, to create the general arrangement plans and sections. The three dimensional model was created using input from all the design disciplines so that these "slices" were cut to form coordinated drawings. This also assisted in understanding the sequence of construction and the detailed temporary works needed within the station and the inclined escalator barrel.

This case history is cited as best and not just good practice, due to the standard of available archive material, uniqueness of the whole project team being located above the works during both design and construction and success in completing ahead of the London Olympic games.

Below shows the completed civil works within the existing inclined escalator tunnel, with the installation for the truss for the new escalator in progress.



Figure 12 Civil works in the incline complete with first truss element being installed for the new escalators

Checklist for future projects

Using the above case history, the approach from the design point of view has been re-evaluated and a checklist for improvement on future projects is now suggested.

Archive Material: to obtain confirmation from more than one source of critical dimensions, locations & details

Confirm existing details from a number of sources, eg archive drawings & photos
Investigate records at specialist organisations, eg Museums & Libraries
Obtain archive drawings from the local authority
Concentrate efforts on building control records, not planning records
Confirm original consulting engineers employed and review their records
Read the legacy drawings in the contract, print out and hold for reference, amending as further information is un-covered.

An important benefit here is to ensure the accuracy of the proposed scheme.

Strengths: to ensure that the most appropriate parameter is used

Read appropriate standards to investigate the relevant design strength
Ensure material properties are obtained from testing to confirm assumptions
Arrange in-situ testing to provide a comparable measured strength
Use both NDT and testing of core samples when considering existing concrete
Consider use of modification factors when converting to a comparable estimate

The economy of design and size of structural elements will be greatly assisted here

Engineering Assessment: to provide realistic assessment

Quote key design parameters related to a standard and justified by test
Ensure topographical survey is undertaken and compare with archive records
Always open up and expose to confirm important elements to be altered
Calculations should always refer to a survey, a drawing or an assumption
Condition should be assessed, especially when considering design life of 120 years
Use a three dimensional model to illustrate both existing and proposed

A systematic framework to an assessment with key data quoted forms an accurate record for efficient and timely checking and approval.

Engineering Drawing: to depict known facts of the existing tunnel or structure within the contract documents

If drawn work is a "slice" from a three dimensional model, include image on drawing
Show key plan with north up the page, vital in a complex underground station
In a tunnel, show the location relative to a ring or segment
With an escalator, always quote the engineers work point, line and level
Add notes to cross reference to an archive drawing or survey
Remember temporary works are rarely recorded, which does not help if they have been left in the ground

This will ensure a definitive as-built record

Engineers Envisaged Sequence: to illustrate the assumptions made in the design

Prepare a separate drawing
Include the assumed construction sequence of demolition, propping, construction.
Show assumed temporary works, location, detail and requirement
Cross reference to the standard specification (eg BTS) or particular specification

This ensures the design assumptions with regards construction methodology are communicated in an unambiguous manner.

Monitoring: to provide review of the works against set "trigger" levels

Prepare a separate drawing show all the instruments relative to the tunnel rings
Include the assumed locations, instrument type, frequency and presentation of results
Set "trigger levels", realistic and ensure they illicit an action plan
Include removal of all instrumentation and proposed reinstatement.

This will be the template to ensure accurate and effective monitoring for the Works.

Conclusion

This paper represents a good practice case study, show-casing techniques, procedures and requirements in the determination of the unknown and the confirmation of the known to ensure the new escalators are completed for use by the travelling public visiting London for the 2012 Olympics.

The author would like to thank my colleagues in Halcrow Atkins involved with this aspect of the Bond Street Station Upgrade and London Underground and Costain Laing O'Rourke Joint Venture who have kindly given permission for the publication of this technical paper.

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Forensic Engineering: Informing the Future with Lessons from the Past
ISBN 978-0-7277-5822-4

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doi: 10.1680/feng.58224.165

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Latent Building Facade Failures

Kimball J. Beasley, P.E., F.ASCE, Wiss, Janney, Elstner Associates, Inc., Princeton Junction, New Jersey, USA

Abstract

Cracked, bulged, and displaced building facades are obvious indications that something is wrong and that repair, or at a minimum, an investigation is needed. However, facades that show no signs of failure may still be degraded or inadequately supported to the point of instability. Facade failures that are concealed from view are more concerning than failures that are readily detectable from visual observations.

Underlying facade deficiencies that rise to the level of life safety hazards can sometimes be difficult or impossible to detect from visual observations. However, there are often certain characteristics or conditions of the facade that can help us to foresee hidden or potential facade failures. For example, a facade system with inadequate support redundancy, complicated connection details, or weak internal stone rifts may be more vulnerable to failure. These failures may become evident in dramatic ways, such as collapse of the facade, or in more subtle ways, such as minor cracking or water leakage.

This paper provides a list of *latent facade failure risk factors* that can be used to recognize and classify risks and consequences of potential facade failure during original building design and during assessment of an existing building facade.

Introduction

There is an expectation that building roofs, windows, and mechanical systems will eventually wear out and require replacement or major repairs. However, facades are generally expected to last the economic life of the building. Roofs and windows will leak and heating or air conditioning systems will simply stop working, indicating that replacement should be considered. Facades, however, do not always provide clear symptoms that signify the need for major repair or replacement.

While building facades in existence today vary widely, certain types of facade materials, configurations, or support systems tend to be more reliable and others more prone to problems. Selection of facades by building owners and designers usually focus on function, initial cost, and appearance. During design attention is not always paid to potential vulnerabilities of the facade system to future underlying failures that would be difficult to detect.

The manifestation of latent building facade failures is often preceded by unexpected forces and/or by gradual reduction in the capacity of the facade or its supports to resist ordinary forces. These failures may have initially occurred some time ago with few or no outward signs. Such hidden failures may become evident in dramatic or subtle ways. Facades may bulge and displace, or their support systems may fail, resulting in collapse. Water leaks to the interior, or deterioration or corrosion of supports, can

occur if surface seals or internal wall weather barriers fail. Facade materials may also experience undesirable changes in appearance, such as staining or exfoliation due to non-durable materials or moisture trapped in the wall. In many cases facade failures are not readily identified, are difficult to remediate, and have high economic and operational consequences.

The level of risk associated with the facade failure depends on the potential consequences or outcomes of the failure. Failures that result in portions of the facade dislodging from a tall building in an urban environment have greater consequences than serviceability facade failures, such as water leaks. Failures such as water leakage may be less dramatic but can have significant implications for building performance.

Facade Types

Exterior building walls fall into four basic categories: solid barrier, surface barrier, cavity, and rain screen. Traditional solid barrier walls are usually composed of brick or stone facades, interlocked or anchored to brick, stone, or clay tile backup. Surface barrier walls have the weather seal at the wall surface. These more contemporary walls are often panelized with discrete gravity and lateral anchors. Cavity walls generally have metal anchor-tied masonry cladding with an internal void space in the wall that channels water to through-wall flashings and weep systems at horizontal interruptions (Brock 2005). Rain screen walls often have panelized veneers. In many cases the facade panels are anchored and supported on a system of tracks and girts. Rain screen systems are intended to prevent water penetration to the interior by deflecting rainwater at the wall surface and minimizing water exposure to the weather barrier within the wall. Curtain walls clad with glass, metal, or tile can be supported and laterally anchored in a variety of ways and may be either surface sealed or internally drained.

With traditional solid barrier walls, the entire wall system acts as a weather barrier by absorbing water and gradually dissipating it to the exterior as moisture vapor. Facade support is generally achieved by abundant mechanical engagement with the wall. With most contemporary facade systems, the supports and lateral anchors need to be adjustable to accommodate variations during construction. The connections also need to be flexible to accommodate differential movements from building and wall deflections and from planar expansion and contraction of the facade. In addition, both traditional and contemporary wall systems require the wall, facade, and support materials to be durable and protected from deterioration by controlling water penetration and retention in the wall.

While a detailed understanding of all possible combinations of building facade materials, support systems, and water management methods is not always possible, a conceptual understanding of the exposure and imposed forces and the expected performance of the types of facades can help to assess the potential facade failure.

Failure Categories

Facade failures usually fall into two broad categories: (1) unexpected behavior leading to undesirable outcomes or (2) inability to resist design loads or unforeseen forces. The first failure category, unexpected and undesirable facade behavior, is usually visually evident. However, the second category, the inability of the facade to resist loads, is often not easily detected until distress is advanced or severe. The direction and magnitude of forces that act on building facades may not always be clearly understood. Design-level wind or seismic loads may not occur during the service life of the building, which means that the facade supports are not tested in service for these loads. Also, the capacity of facades and their supports to resist forces may gradually diminish over time due to deterioration or fatigue. To avoid conditions that can lead to failure, building facade attachments must support the weight of the facade and resist all expected and unexpected forces (Beasley 2001, P. 42-

45). The facade components also need to endure erosion from wind and water, the effects of temperature extremes, and ultraviolet light (if exposed) without significant loss of strength. Design of facades and their supports also needs to be sufficiently constructible to accommodate construction errors under harsh field conditions. The facade that (with reasonable maintenance) does not perform satisfactorily for the economic life of the building is considered to have failed. Clearly, progressive collapse or high-consequence failures are the most disturbing category of failures.

Recognition of facade failures beneath the surface is more likely with certain wall systems. Thin facades tend to show bulges and displacements. Brittle facade materials crack and spall. Facades that are not designed with adequate support redundancy or facades that have tensile forces imposed on weak points (e.g., along mortar-bedded masonry joints or stone rifts) are more vulnerable to failure. Other problem-prone facades include adhered ceramic or stone tile veneers without supplemental mechanical attachments, highly anisotropic (differing strength properties depending on the orientation) or variable-strength stone panels, and composite facades that combine unproven and incompatible materials.

Hidden and premature failures are commonly associated with facade systems that mimic but are inferior to traditional stone or brick which are popular for economic reasons. The shorter economic life and the mode of failure of such facade systems need to be understood to fully appreciate the cost/benefit compromise. Synthetic stucco, cementitious panels, and cultured stone facade systems, which are in common use today, offer a cost-effective option for developers and owners. However, non-durable components, incomplete or deficient design details, and inadequate flashings and internal weather resistant barrier systems, frequently compromise the ability of these systems to resist deterioration and air and water penetration.

Failure Avoidance

Design Considerations

Facade failures that develop over time often begin during design. Façade designs that adequately account for imposed forces, facade and structure movements, exposure and durability, water management, constructability, and future maintenance will most successfully avoid failures (Johnson 2003). Since all forces that will act on the facade cannot be known, the design must apply an appropriate factor of safety to anticipated forces. Certain physical properties of the facade or structure may result in direction and magnitude of forces not anticipated in design. For example, a facade material with a high coefficient of thermal expansion and rigid connections may experience forces greater than anticipated in design. Since most building facades are situated above accessible sidewalks and property, there is an inherently greater risk commonly associated with facade failures.

Construction Considerations

The best facade design can be undermined by inadequate communications and poorly supervised construction practices. The contractor needs to understand the facade design concept and should have experience with the particular type of facade system being constructed. Good communication between the contractor and designer, and adequate involvement by the designer during construction, provide important safeguards against construction errors.

Owner Considerations

A primary responsibility of the designer of a new building facade is to convey to the building owner all pertinent information about the facade system. The owner must have a full appreciation of the facade's performance limitations and future care and maintenance needs. Similarly, owners responsible for existing buildings need to realize that no facade system is maintenance free. Certain elements of the

facade need scheduled periodic maintenance and the entire facade requires vigilant attention. In many major cities in the United States a periodic safety inspection of the facade by a qualified professional is mandated by law.

Facade Failure Potential

Facade characteristics can be categorized to help predict the potential of the facade to develop hidden failures. This can help designers and owners make informed decisions during design, including an assessment of future risks associated with the building facade selection. Also, an understanding of the failure potential for facades on existing buildings can help to assess the need for remediation or action to reduce the risk to an acceptable level. Certain characteristics and conditions may be used to classify the level of risk associated with latent facade failures. The *Latent Façade Failure Risk Factors* described below are intended to identify characteristics and conditions associated with building facade failures.

Latent Facade Failure Risk Factors

1. *High consequence* - e.g., facades situated directly above congested urban sidewalks or similar exposures.
2. *Insufficient support redundancy* - e.g., inability to redistribute and continue carrying loads if a portion of the facade or its support system fails, especially with eccentric or overhead (hung) facade elements.
3. *Improper tensile force* - e.g., tensile or flexural stress imposed along mortar bond lines or along a weak rift of anisotropic stone panels.
4. *Poor reliability* - e.g., facade materials or anchorage systems with variable strength properties, unproven systems or materials, or indeterminate performance characteristics.
5. *Inadequate durability*- e.g., corrodible, mild steel anchors or embedments exposed to moisture, degradable facade materials subject to strength loss or deterioration.
6. *Poor constructability* - e.g., complex anchorage assemblies, reliance on precision facade or support installation, difficult construction environment, insufficient accommodation for construction tolerances.
7. *Inadequate water management system* - e.g., absent or poorly configured internal flashings, obstructed water drainage paths, potential to retain water behind the facade, reliance solely on surface seals.
8. *Potential warning signs* - e.g., location and orientation of cracks may indicate restrained movement or material degradation; stains may indicate corrosion of underlying mild steel supports; movement at ends and corners of long or tall walls may indicate ineffective facade expansion joints and accumulated differential movement that may have damaged underlying facade supports or anchors. The presence of these symptoms indicates potential latent facade failures.

Case Studies

As with the wide possible range of building facade types and exposure conditions, facade failures can occur from a great number of causes with countless circumstances and consequences. The descriptions and captioned photographs below show examples of some representative types of failures to provide a better understanding of various conditions that may lead to facade failures.

Case 1 - Thin stone façade panels on this building relied on a combination of wire ties and spot application of mortar for support. As the mortar-to-stone bond deteriorated over time the wire ties alone were inadequate to support the panel. This condition would score poorly for Risk Factors 2, 3, 4, and 7. The consequence of failure, Item 1, is not included because most of the facades are above relatively inaccessible landscaping.



Figure 1. Stone facade panel collapse resulting from inadequate support redundancy.

Case 2 - The sedimentary stone panels on this building are highly anisotropic due to weak bedding planes oriented parallel to the panel surface. Portions of the stone are vulnerable to detachment as internal delaminations develop over time. This condition would score poorly for Risk Factors 1, 2, 3, 4, 5, and 7.



Figure 2. Unsupported stone fragment created by concealed crack (arrow) along rift of face-bedded stone.

Case 3 - Corrosion of a mild steel plate embedded in the wall behind this stone panel resulted in dislodging of the stone from expanding rust scale. Spalling or displacement of masonry or concrete facades from "rust jacking" of embedded steel reinforcing or connections is one of the most common forms of latent façade failures. This condition would score poorly for Risk Factors 1, 5, and 7. Item 7 is included in this and the other cases where no drainage cavity and water weep system exist.



Figure 3. Corroding steel plate dislodged stone panel.

Case 4 - Thin marble panels exposed to temperature fluctuations on this building bowed and warped from marble hysteresis caused by non-uniform expansion of the marble near the panel surfaces. Hysteresis occurs when tightly bonded marble grains dislocate over time from repeated temperature variations (Beasley 2006, P. 111-112). This condition would score poorly for Risk Factors 1, 4, 5, and 8. Item 8 is included because bowing panels were apparent throughout the building prior to a panel dislodging and falling.



Figure 4. Anchor detachment occurred as this thin marble panel bowed and displaced.

Case 5 - Failure of the marble facade panel connection occurred where dowels used to align and laterally secure these panels lacked edge clearance. Forces inadvertently transmitted across joints of facade panels that are intended to be independently supported can cause load concentrations at hard points resulting in localized spalling and cracking. This condition would score poorly for Risk Factors 1, 6, and 8. Item 8 is included because anchor point cracks and incipient spalls were visible at several locations on the building.



Figure 5. Marble facade panel fractured and spalled due to inadequate end clearance for stainless steel dowel.

Case 6 - Ceramic tiles are secured to the wall substrate solely with an adhesive mortar. The tile had dislodged and collapsed as shown and had debonded but not yet collapsed at various other locations throughout the building. This condition would score poorly for Risk Factors 1, 2, 3, 4, and 7.



Figure 6. Ceramic tile veneer collapse resulting from a lack of supplemental mechanical anchorage and poor adhesion to wall substrate.

Case 7 - Ceramic tiles were adhered to wood-fiber impregnated portland cement based backer boards. Bowing and dislodging of the composite wall panels resulted from differential planar contraction from reduction of moisture in the cement backer board relative to the tile. Facade components and mateerials that have a successful performance history individually may not function as intended when used together in novel and unporoven combinations. This condition would score poorly for Risk Factors 1, 4, 7, and 8. Item 8 is included because bulging wall panels were apparent throughout the building.



Figure 7. Thin composite panels bow and crack from differential expansion and contraction of the tile relative to the cementitious backer board.

Conclusions

Undetected facade failures can result from a variety of causes. An awareness of certain characteristics of the facade system that can create vulnerabilities to facade failures may help to avoid or minimize risk. During design of the building, classification of a facade based on *Latent Facade Failure Risk Factors* can help to assess the potential for future facade failures. Recognition of subtle signs of distress during the service life of the building will also help to determine the existence of latent facade failures, and to ensure that distress conditions are addressed in a timely manner.

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Forensic Engineering: Informing the Future with Lessons from the Past
ISBN 978-0-7277-5822-4

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doi: 10.1680/feng.58224.175

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Remote Investigation of Blast Effects on Buildings

Stephen Hudson Arup London United Kingdom

Introduction

The bombing of central Oslo on the 22 July 2011 was shocking for the world to see. But it was seen and seen by countless people across Europe and the World, due to the accessibility and permeability of the media, via the internet and other means of communication. What was striking was the quantity of information available and the timeliness with which it was presented. Graphic detail became available within minutes of the event and the quantity quickly expanded. Access to media providers from across Europe added to the diversity of imagery available. This included evidence of the deformation caused to building elements by the explosive force: glass, reinforced concrete and metal that had been shattered, detached, bent and buckled. By using internet search engines and open-source mapping it was possible to compare and contrast the building elements before and after the event. This information also greatly assisted in identifying the location of damaged elements and their position and orientation relative to the point of detonation. It became apparent that, from the evidence available, it is possible to quantify the level of damage caused and to back calculate the size of explosive device that had been detonated. The quantity and variety of imagery available provided the potential for multiple independent assessments and thereby check the validity of each by the closeness of their correlation.

Just over a month after the Oslo bombing, the United Nations Building in Abuja, Nigeria was also attacked. In contrast to the earlier event the imagery available was relatively limited in quantity and the variety of damage shown. Only the front façade of one building was shown. The sources of information were also not as numerous, often with the same image being displayed by several news providers in the United Kingdom. It appeared that, from the imagery seen, only one building had been affected, which may or may not have been the case. Shortly following this terrorist attack by Boko Haram a structural assessment was conducted by Arup Nigeria and in order to complete their analysis they shared the large number of photographs recorded with the specialist team in London. It was evident that the vehicle-borne device had detonated inside the building. The most graphic examples of the damage caused were inside also and had not been seen by the media photographers outside the building at the time of the incident. In this instance the Arup survey information, rather than the open-source information used previously, had the potential to be exploited in order to identify the size of the explosive device.

These were just two headline-grabbing incidents among many large explosive events that have occurred before and since whereas Iraq, Pakistan, Nigeria and Syria are notable for the frequency of events that occur there. From a professional and dispassionate perspective, there is much that can be learned from these events such as the cause and the effect. Our specialist knowledge is deepened by gaining an appreciation of the methods used by violent groups around the world and this knowledge also assists in identifying patterns in a series of events and in forming profiles on malicious groups or individuals. Also by studying the behaviour of buildings under such events we, as engineers, can better design for similar occurrences in the future. In a third case study the events that have unfolded in Damascus has been studied. During the 16 months period between December 2011 and March 2013 11 large explosions

have been reported in the city by Western media. There has not been the same level of detailed media coverage of each event as there was in Oslo and there has not been access to first hand recorded information as there was in Abuja. In contrast there are eleven similar but different incidents available to compare and contrast. The level of confidence in the conclusions is lower but the range of information from the same geographic area, at regular intervals within a short time period and potentially by the same or related groups is also valuable. There is also merit in gaining a small amount of information quickly rather gaining none at all.

This paper will consider each scenario in turn, detailing the information utilised, the analyses conducted and the conclusions, followed by a discussion of the wider significance of this methodology and possible applications for it. Potential developments for this methodology will also be discussed in order to highlight future possibilities. It is not the intention of the paper that first-hand investigations can be replaced by the methodology described here. This remote investigation technique is intended to provide an option when a first-hand investigation is not possible or to proceed or supplement another investigation. Every incident is site specific and the evidence created depends on site features. The advantage of personal attendance by an experienced observer is that the tell-tale damage details can be looked for.

Before considering the first case study the methodology that is applicable to all case studies will be described.

Methodology

The process employed was to identify damage and then correlate the loading required to create that damage with the load resulting from a range of explosive charge sizes. The process was repeated for a number of different examples of damage in each scenario so as to validate the other conclusions.

As a starting point it was necessary to know the spatial geometry of the environment:

- the distance from the point of detonation;
- the orientation to the blast, appreciating the effect angle of incidence has on severity of blast loading;
- the influence surrounding topography might have, be it man-made or natural. This might confine and or reflect the blast changing the loading experienced by the damaged element.

In the most basic scenario this may be just a distance and angle or in more complex scenarios this may require the creation of a building or cityscape model in three dimensions.

The blast loading was then calculated, for these geometric constraints, for a range of probable explosive charge weights. The parameters calculated were pressure, duration and subsequently the impulse, the accumulated pressure with time. While various methods could have been used, for the case studies documented Conwep[1] was used for simple geometry, utilising the Kingery and Bulmarsh[2] equations, and where the blast interacted with more complex forms and topography Air3D[3] was used, a computational fluid dynamics (CFD) software package developed specifically for blast modelling.

In order to identify the performance criterion, single degree of freedom (SDOF) calculations were performed on each element. The resistance function was found for the element and then Ergo[4] software was used to plot iso-damage curves of impulse verses pressure for the observed deformation. Depicted in Figure 1a, an iso-damage curve shows that if the impulse is sufficiently low a certain performance criteria (rotation, displacement, ductility, failure, etc.) will not occur no matter how high the pressure and if the pressure is sufficiently low it

will not occur no matter how high the impulse. There is a transitional zone between the two creating a boundary line and if both the pressure and impulse exceed this then the performance criteria will be exceeded. By conducting a large number of analyses for multiple pressures and impulses it is possible for the line to be plotted.

By plotting the recorded pressure and impulse for the various explosive charge sizes considered on the same graph as the iso-damage curve it was then evident which charge size was most likely to have caused the observed damage (Figure 1b).

Examples where there was variation in damage and deformation were of particular value because it indicated that the material response was quite finely balanced; a small change in the loading would change the effect seen and this presented significant potential for precision. Where there was variation with distance it may be possible to see a variation in material response or where distance remains the same but damage is variable it would suggest that the performance criterion was marginal.

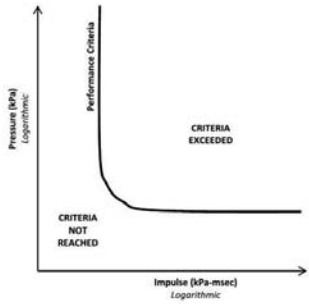


Figure 1a. Iso-damage curve

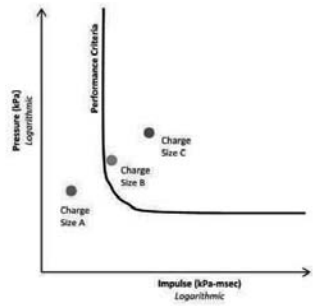


Figure 1b. Iso-damage curve with charges

Oslo Bombing – 22 July 2011

In the time that has passed since the Oslo bombing much of the detail of what happened has become clear. What is presented here is based on what was available and how it was interpreted at the time. It was not known for certain what type of device had been used or how it had been delivered. A crater was visible just outside the government building known as H Block indicated the most likely point of detonation. Also damage was decreasingly severe with distance away from this point. All images were found via Google Images[5]

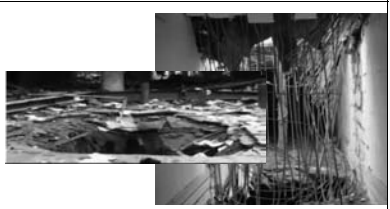
Qualitative Blast Effects

Of the effects observed, several items, shown in Figure 2, provide vivid evidence of the effect of the explosion and add to the viewer's appreciation of the severity of the event. They were not used however to quantify the charge weight for reasons of subjectivity or uncertainty.

	Figure 2a. Loss of foliage.
	Foliage had been stripped from the right hand trees which were in line-of-site of the blast but where the building provided more shelter the foliage remained.
	Figure 2b. Charing.
	Blackening was visible on the low wall and cantilever roof. If caused by the fireball, this would only have occurred within meters of the blast. If caused by secondary fires, this could have occurred at greater ranges.
	Figure 2c. Masonry cladding.
	Masonry facade cladding detached from the face of building R4. It was not possible to judge from the picture what mechanism had caused failure and what strength the wall would have had.

Quantitative Blast Effects

Figure 3 shows examples of deformation or damage that, while potentially quantifiable, were not selected due complexity or the precision required to perform the calculation, i.e. there were other examples that required less effort, but also the evaluation of blast loading lacked the level of complexity or precision to give such calculations validity.

	Figure 3a. Failure of wall and breaching of floor slabs
	The short stand-off of the explosive charge, the precise architectural layout, structural form and material composition raised too many variables; in these examples the assumptions that have to be made with open source information lead to too great a variation in possible results. The ground floor slab outside H Block has breached as has the basement slab below and a wall. It has been calculated that the extent of the breaching is caused by a charge size of several hundreds of kilograms of TNT but it is not possible to be more specific without closer inspection.
	Figure 3b. Throw of laminated pane
	The laminated pane has been thrown and on further investigation it was assessed to have moved 40m from the location of the curtain wall in the background. It has been calculated that an impulse of several thousand kPa-msec is required to achieve this but without knowing how it behaved aerodynamically it is not possible to calculate this with certainty. i.e. what was its trajectory, its orientation and did it tumble?
	Figure 3c.
	The light sheet metal elements at roof have deformed and detached. With closer inspection it may be possible to make deduction from the size and extent of damage.

Five example of damage were chosen for quantitative analysis because of the clarity of the identifiable performance criteria and because they showed different forms of deformation at different distances and orientations from the point of detonation. The examples chosen include cracking and loss of rigidity in a cantilever roof and low wall due to bending, deformation and detachment of window mullions due to bending and glazing failure. Each analysis is descripted sequentially below.

Blast Load Derivation

A 3D computer model was created of the buildings within approximately 250m of the seat of the explosion including all buildings where damage had been observed. The horizontal dimensions were scaled from Google Maps[6] and the heights estimated from Google Street View[5]. The dimensions from the model were then input into Air3D[3]. Although this model is approximate and simplified it is sufficient to replicate the effects of confinement and reflections on the blast load. A series of probe arrays were placed in the Air3D model in order to record the pressure-time histories. Figure 4 shows the stages of Air3D model creation from Google imagery to output of the analysis.



Figure 4a.



Figure 4b.

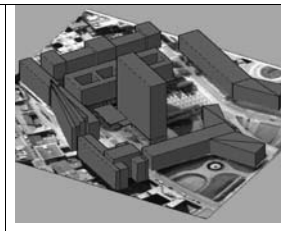


Figure 4c.

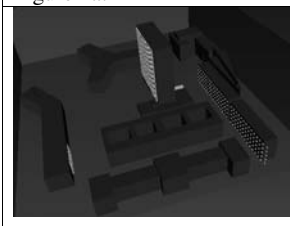


Figure 4d.

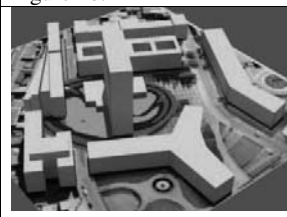


Figure 4e.



i



ii



iii



iv

Figure 4f.

Analysis of Damaged Elements

H Block Cantilever Roof



Figure 5a

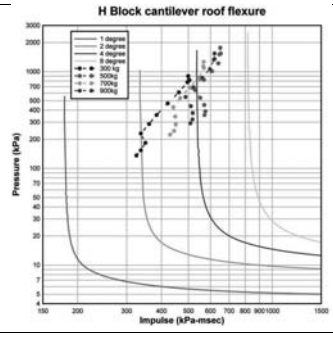


Figure 5b

The cantilevered overhang in Figure 5 has lost its flexural stiffness and is sagging. The degree of sagging varies with position along the length of the cantilever perpendicular to H Block

The iso-damage curve shows that a 300kg net TNT equivalent charge is unlikely to have caused flexural failure in the overhang whereas a 900kg charge would have caused loss of flexural rigidity to the entire north side overhang. The most likely charge size is therefore 500-700kg and because sagging occurred in less than 50% of the overhang the charge size appears to be nearer the 500kg line.

The variability of damage along the width of the cantilever allows a close match to charge size to be made. However due to the small standoff, a small error in the assumed charge location will have a significant effect on the results. The reliability of this conclusion relative to the other examples of damage identified is considered to be medium.

H Block Low Wall



Figure 6a

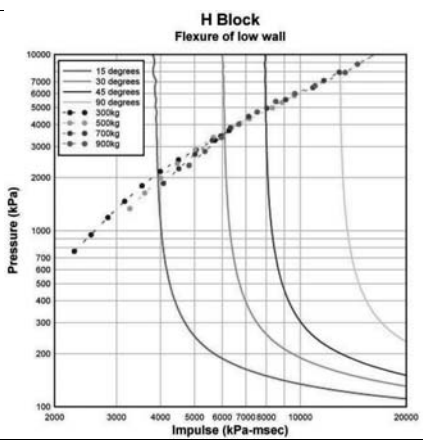


Figure 6b

The cantilevered low wall in Figure 6 has lost its flexural rigidity and has bent over by as much as 90 degrees nearest to the blast, and significantly less further away but a visible rotation can be seen along the length.

From the pressure-impulse graph one would expect to see greater rotation along the wall width if 900kg had been used and less than 30 degrees if 300kg had been used. 500kg and 700kg correlate most closely with the observed rotation with 700kg being most credible because it encompasses a range from 15-90 degrees.

The variability of damage along the width of the wall allows a close match to charge size to be made. However due to the small standoff, a minor adjustment to the assumed charge location will be quite significant. The reliability of this conclusion relative to the other examples of damage identified is considered to be medium.

R4 Block – Window Mullions



Figure 7a

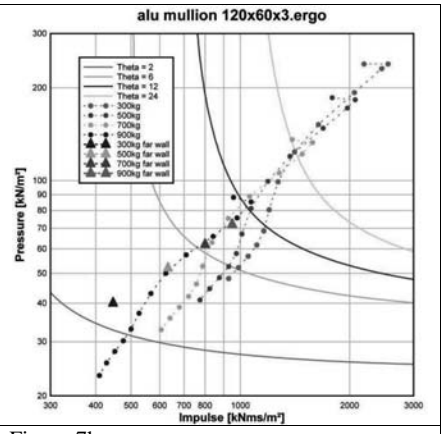


Figure 7b

The blast loading caused varying degrees of flexure in the window mullions at different locations shown in Figure 7. On the nearest corner to the blast the mullions have been removed, slightly further away they are visibly bent and further still the deflection is not noticeable from the photographs.

If the charge size was 300kg no mullions would have detached. If it were 900kg all would be plastically deformed and many would be detached. Of the 500kg and 700kg charges the proportion of not deformed to detached lies midway between the two lines.

Due to the large variation of damage with distance at different locations and the ability to discard two charge sizes this conclusion has high reliability relative to the other examples. The greater standoff distance also reduces the effect of any adjustment in the location of the charge.

H Block – Rear Face Glazing

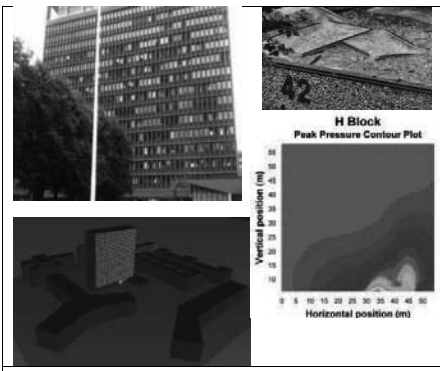
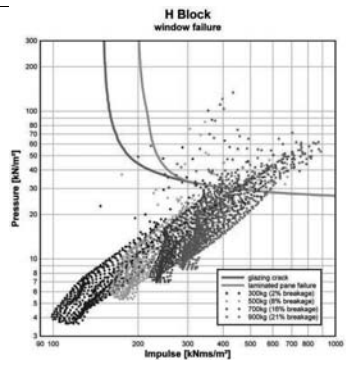


Figure 8a. The photographs show the damaged glass, the 3D model shows where the pressure-time histories were recorded and the contour plot shows the peak pressure that was recorded at each point on the façade.



8b

Some windows have failed in Figure 8 while others have not, suggesting that the loading was at the threshold of window failure. By counting the visible window it was observed that 25% had failed. It can be seen from photographs of debris that the detached panes appeared to be laminated and the large quantities of finely broken glass suggest a toughened outer pane.

The analysis predicted that a 900kg charge would cause 21% glass failure due to pressure, which corresponds most closely to the observed 25% breakage. However the contour plot shows that the observed area of glazing was more onerously loaded than that which is obscured by the trees in the picture. The actual breakage could therefore be less than 25%.

Due to the greater uncertainty of glass make up and obscured observation the reliability of the conclusion is considered low.

VG News Building – Glazing



Figure 9a

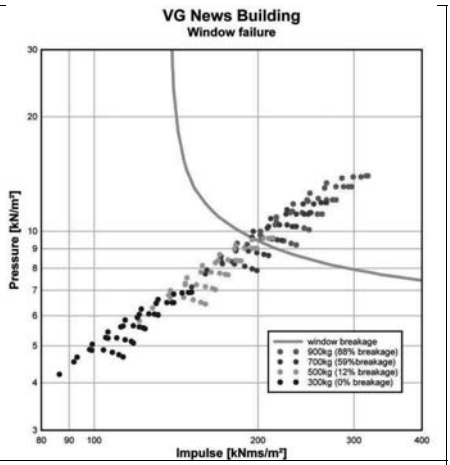


Figure 9b

Some windows have failed in Figure 9 and some have not suggesting that the loading was at the threshold of window failure. It can be seen from photographs that the glass has broken into large irregular shards suggesting annealed glass. One window shows a broken outer pane and intact inner pane showing that it is double glazed.

The analysis predicted that a 300kg charge would cause no glass failure whereas 900 kg would have broken the majority of the windows. The proportion of broken panes lies between that expected from 500 kg and 700kg and closer to 700kg.

Being able to disregard two charge sizes and due to the close correlation of the analysis with the observations suggests the results are moderately reliable.

Case Study Summary

Summarised in the table (Figure 10) are the conclusions from each damaged item quantified above. For each item the range of possible charge size and the best fit is shown. The relative confidence in the conclusion is shown by a thicker or thinner row. There is a clear trend apparent between 500kg and 700kg with 600kg seeming most likely.



Figure 10

These results are dependent on the accuracy of the assumptions made. The assumed dimensions and material properties of the building elements, based on inspection of the photographs and professional experience, are considered likely to resemble the actual building elements. If however the building elements were more or less substantial than assumed then the assessed charge size would have been larger or smaller also. By looking at several examples of damage erroneous assumptions are likely to be evident by results that don't fit the trend.

The CFD model has assumed that the building surfaces reflect the blast pressures. In reality some pressure would have passed into the building and some energy would have been expended by deforming the building elements. The pressures recorded are likely to be slightly more onerous than those experienced in reality and therefore the damage observed may have been caused by a slightly larger bomb than estimated.

Anders Breivik provided a detailed description of his intended vehicle borne explosive device. He stated that the van was to carry a main charge of 950kg of homemade explosives consisting of 900kg of ammonium nitrate aluminium fuel oil (ANALFO) and 50kg of ammonium nitrate aluminium nitro methane (ANALNM). Assuming that he carried out this plan, it is clear that the explosive effect was not equal to that weight of TNT. There are two possibilities here:

- The homemade explosives may not have been as powerful as TNT, fertiliser based explosives are generally found to have an equivalency to TNT of 0.4 – 1.1 depending on the quality of the material used, the method of processing and the duration and method of storage.
- The large quantity of explosives only partially detonated.

As shown in Figure 11, if 950kg of explosives detonated and the effects of 600kg of TNT has been observed, the charge used in Oslo had an effective equivalency to TNT of approximately 0.6 but if the equivalency reached 1.1 then only 550kg would have detonated. It is known that Anders Breivik had experimented and refined his methods and so the equivalency is likely to be quite high. However he had recorded that in his trials he struggled to fully detonate the explosives. He also claims his trial was a 50kg main charge, which would not have given him experience of detonating much larger quantities.

		Weight of ANALFO / ANALNM exploded										
		950	900	850	800	750	700	650	600	550	500	450
		Percentage of 950kg charge detonated										
		100%	95%	89%	84%	79%	74%	68%	63%	58%	53%	47%
Equivalency to TNT	1.1	1045	990	935	880	825	770	715	660	605	550	495
	1	950	900	850	800	750	700	650	600	550	500	450
	0.9	855	810	765	720	675	630	585	540	495	450	405
	0.8	760	720	680	640	600	560	520	480	440	400	360
	0.7	665	630	595	560	525	490	455	420	385	350	315
	0.6	570	540	510	480	450	420	390	360	330	300	270
	0.5	475	450	425	400	375	350	325	300	275	250	225
	0.4	380	360	340	320	300	280	260	240	220	200	180

Figure 1. Yellow indicates the range of what was possible, orange likely and red assessed to be most likely detonated weight.

From the range of possible equivalencies shown in the table (Figure 11) it is likely that the actual bomb was nearer to optimum equivalency but did not total detonate Therefore it is assessed that less than 80% of the 950kg charge detonated with an equivalency greater than 0.8 resulting in an effective equivalent TNT charge weight of 600kg.

UN Nigeria – 26 August 2011

Following Arup Nigeria's structural survey approximately 400 photographs[7] were shared with the London Resilience, Security & Risk team as well as a key indicating the location and orientation of each photograph. A set of technical drawings for building was also provided. The photographs showed evidence of a crater inside the building where the likely point of detonation was. This was within a full building height atrium and other signs of damage within this atrium also indicated that the point of detonation had occurred there: reinforced concrete columns had been stripped of cover and in some circumstances the concrete had cracked, been reduced to rubble within the confinement of the reinforcement and in some cases almost detached in shear; reinforced concrete balustrades surrounding the atrium had cracked; lift shafts metalwork had buckled. At greater distances throughout the building masonry walls had breached, windows had shattered and general destruction was evident. In order to gain a sense of perspective, the damage shown in the photographs was categorised as per US Army Corps of Engineers classifications[8] and colour plot was produced showing the extent of severity of damage throughout the building, shown in Figure 12.

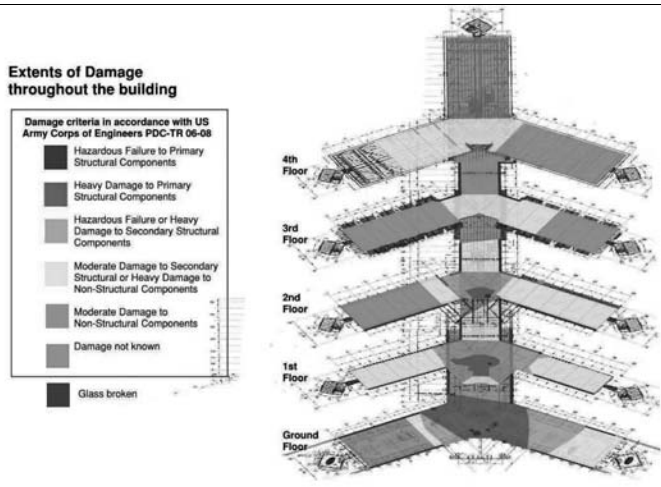


Figure 12

Qualitative Blast Effects

Once again, not all that contributes to the full appreciation of the effect of the blast is quantifiable. The extent of violent disruption to fixtures and fittings demonstrates the penetrating and enveloping power of the blast, the shattering of hollow blocks is evident at shorter standoffs and remnants of the car is very graphic. See Figure 13.



Figure 13. Qualitative damage

Quantitative Blast Effects

Two examples of damaged cause that have not been used to quantify the blast load were the effect of shear in the tops of columns and deformation in the lift shaft metal work. The former is interesting as it demonstrates the blast reflecting off the underside of floor slabs and increasing in magnitude in this upper zone where the column meets the slab. See Figure 14.



Figure 14 Quantitative damage not analysed

The elements chosen for analysis included the glazing on the South, North East and East facades, Masonry infill panels situated in the suspended lobby area, the North East façade on the first floor and internally and the reinforced concrete balustrades.

Blast Load Derivation

As in the first case study an Air3D analysis was completed to derive the loads (Figure 15) but in this example the dimensions were taken from the technical drawings and it was the internal blast-structure interaction that was of interest. Rather than representing the building as a solid block, the voids for each floor and the atrium were included, the columns, balustrades and lift shafts were included and cavities were included for windows. Target point arrays were included on many of the internal surfaces where visible damage had occurred.



Figure 15a.

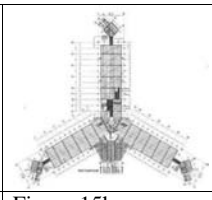


Figure 15b.

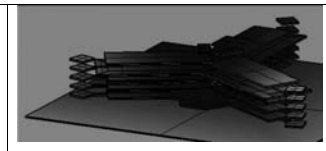


Figure 15c.

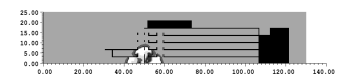


Figure 15c.

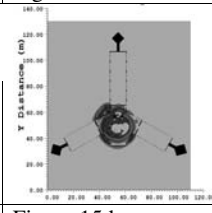


Figure 15d.

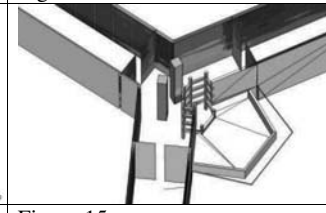


Figure 15e.

Analysis of Damaged Elements

South East Elevation



Figure 16a

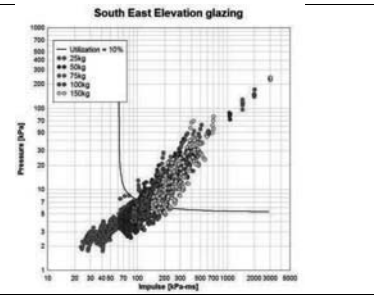


Figure 16b

Almost all the windows in the front face of the building had failed (Figure 16). The majority of broken glass had been cleared away, due to the delay between the event and the photographs, making identification of glass type difficult. From one photograph of a retained cracked pane it appears likely that the glass is laminated annealed.

Only the 100kg and 150kg charge weights create loading on the South Eastern façade sufficient to break almost all windows and of these two the 150kg charge appears to correlate most closely with the observed damage.

South East Elevation



Figure 17a. The photographs show the damaged glass and the contour plot shows the peak pressure that was recorded at each point on the façade.

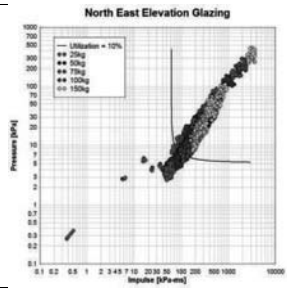


Figure 17b

Approximately 2/3rd of the windows appear to have failed on this façade (Figure 17) but this proportion is distributed across the façade with increasing distance from the point of detonation; at the furthest distance no windows are broken and at the shortest distance they are all broken.

The pattern created by the 25kg and 50kg analyses on the iso-damage curve most closely represents the damage seen in that a significant proportion of target points are below the damage criteria. Of these 50kg seems closer as more failed than did not.

East Elevation



Figure 18a

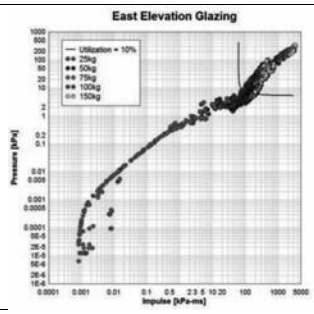


Figure 18b

Compared to the South East elevation the furthest reaching windows on this façade (Figure 18) are further from the point of detonation. The orientation of the windows to the blast is also more oblique and consequently fewer windows are broken; just less than 50%. The internal layout and possible obstructions to the blast may also have had an effect.

The plot from the target points suggests that anything lager than 50kg would have likely caused more widespread damage. Of the two remaining charge weights the observed damage appears equidistant between them.

Elevated Lobby Masonry



Figure 19a

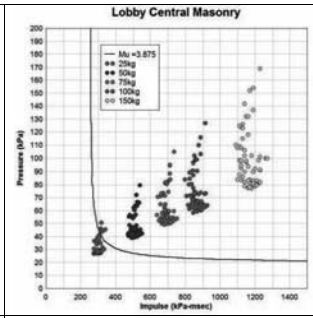


Figure 19b

In the area that overhangs the main entrance the masonry wall had partially failed (Figure 19). It appears most likely, from the spatial geometry and the observed damage that the wall failed due to internal loading; the scab of masonry that has come away has left a hole larger on the outside than the inside. That the wall has not more extensively failed is beneficial to assessing the charge weight as the damage can be closely aligned to the performance criteria.

Of the five charge weights considered only the 25kg charge creates a plot that straddles the iso-damage line.

South East Elevation 1st Floor Masonry

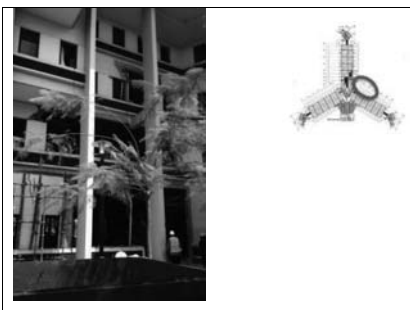


Figure 20a

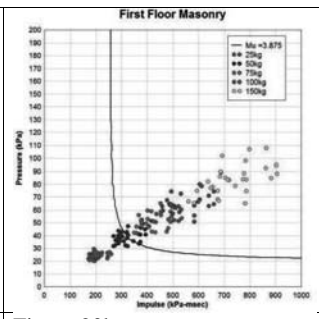
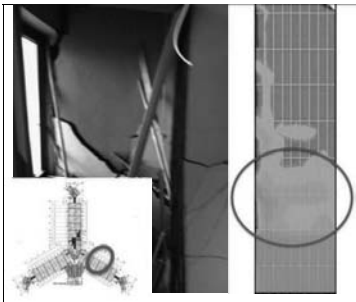


Figure 20b

It can be seen from the picture (Figure 20) that the masonry has breached on the ground, 1st and 2nd floor on the rear face of the East Wing. Neighbouring walls on the corresponding position of the North Wing have not failed and this is likely due to the oblique angle that they were orientated to the blast. The internal partition walls in the East Wing appear to have confined the blast, exacerbating the effects in this area but also limited the effects from spreading elsewhere. It can be seen in the photo that the effects diminish with height.

It is difficult to judge from the photographs whether there was more than enough energy to fail the wall or only just enough. From the plot of the target points 25kg seems unlikely as failure would not have been expected to occur. Of the 50kg and 75kg charge target points, 75kg provides the closest correlation as all target points lie within the zone of failure, however larger charge sizes cannot be ruled out based on this example.

East Wing 2nd Floor Internal Masonry Wall



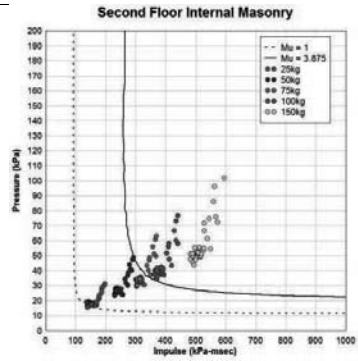


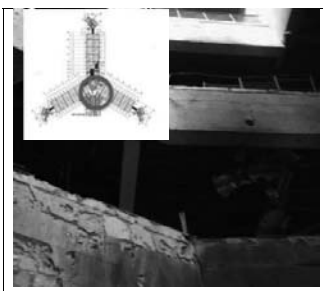
Figure 21a. The photographs show the damaged wall and the contour plot shows the peak pressure that was recorded at each point up the building height.

Figure 21b

Large cracks and deformation are visible in the internal masonry wall and deflections can be seen (Figure 21). As intensity of load decreases from ground floor up the damage reduces also. It is evident from the photographs that this wall is beginning to fail and so a close correlation can be made between observed damage and possible load.

The target points have been plotted over two iso-damage lines in this example; ductility equal to 1 and 3.8, which are considered to be the lower and upper bound respectively of visible damage prior to failure

East Wing Internal Wall



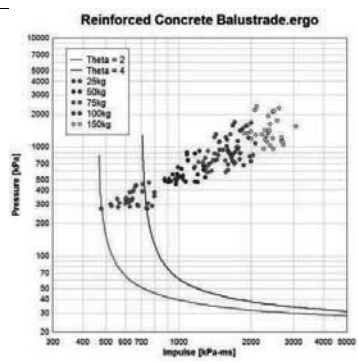


Figure 22a

Figure 22b

Within the internal atrium the reinforced concrete balustrade has cracked in bending (Figure 22). This is very apparent on the first floor, with visible rotation and apparent but less so on the floor above.

Diversity of element type, location and material is valuable to this assessment as it aligned results give credence to each other. The short stand-off however increases the error of any assumptions made.

Case Study Summary

Shown together in the summary table (Figure 23) it can be seen that there is a trend towards 25-50kg with 40kg being most likely. The actual quantity of homemade explosives is likely to be significantly more.

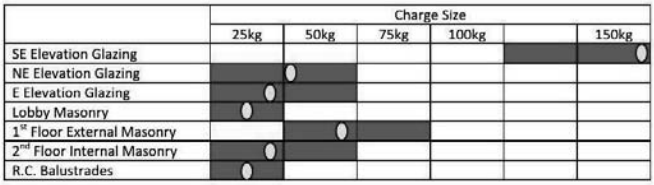


Figure 23

Damascus, December 2011 – March 2013

Since the unrest in Syria has intensified into conflict there have been many incidents of blast across the country reported in the Western media and undoubtedly not all that has occurred has been reported. For this case study a selection has been restricted to reported car bombs within the district boundary of Damascus. This has been done to rationalise the selection to that which is manageable and comparable, not because they are considered any more significant than acts of violence elsewhere in the country or carried out in other means such as conventional munitions.

While the same process has been followed as in earlier examples, it has been carried out to a level of detail proportional to the level of information available. It has also been conducted in a more expedient fashion to show that a low level of valuable analysis can be completed in a timely fashion for a larger quantity of events. The events, observations and analysis results are summarised in Figure 24 and the subsequent table. Pressures and Impulses were calculated based on distances and angle of incidence compared with the graph in Figure 25. Images sourced from Google Images[5] and the BBC News website[9]. Videos of the incidents proved particularly valuable as screen shots could be taken when the camera should damage of interest that was not available elsewhere as still images.

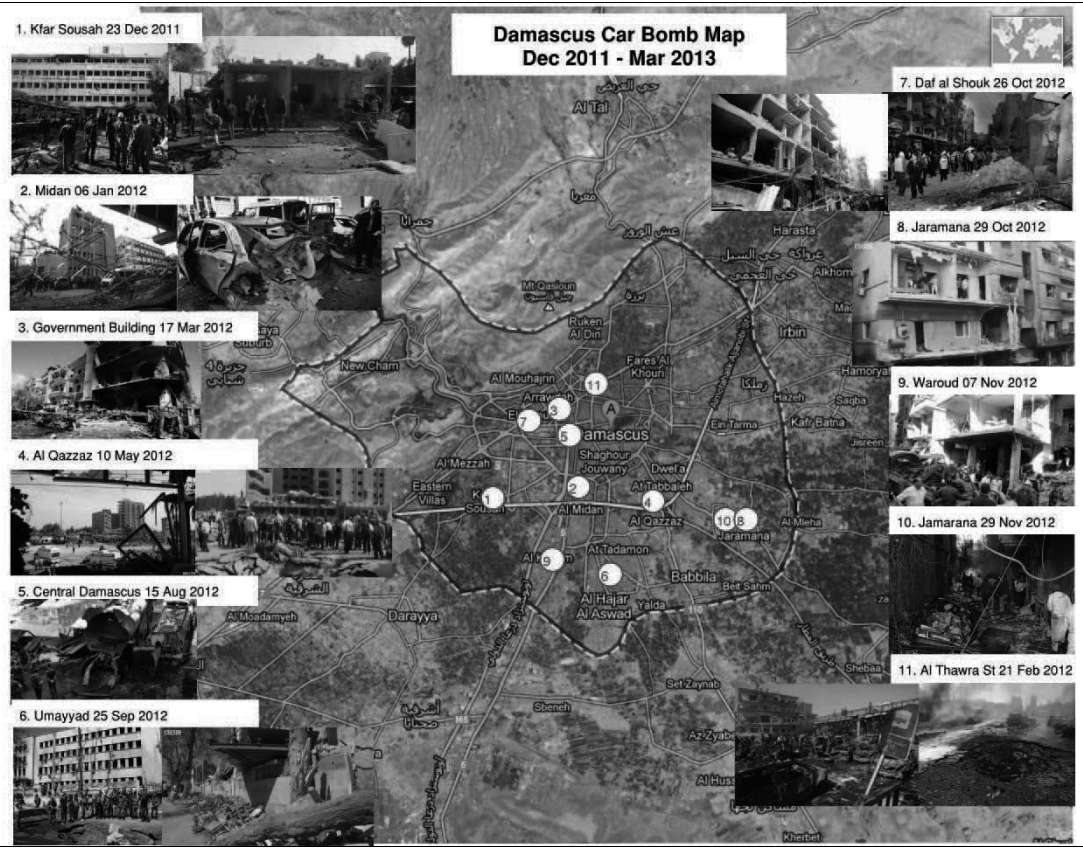

Damascus Car Bomb Map
Dec 2011 - Mar 2013
1. Kfar Sousah 23 Dec 2011
2. Midan 06 Jan 2012
3. Government Building 17 Mar 2012
4. Al Qazzaz 10 May 2012
5. Central Damascus 15 Aug 2012
6. Umayyad 25 Sep 2012
7. Daf al Shouk 26 Oct 2012
8. Jaramana 29 Oct 2012
9. Waroud 07 Nov 2012
10. Jamarana 29 Nov 2012
11. Al Thawra St 21 Feb 2012
Damascus
Al Midan
Al Qazzaz
Al Mezzah
Eastern Villas
Shaghour
Jouwany
Dweila
At Tabbaleh
At Tadamon
Babbila
Bet Sahm
Al Hajar Al Aswad
Yalda
Darayya
Al Moadamyeh
New Cham
Al Mouhajrin
Fares Al Khouri
Ruken Al Din
Mt Qasioun
Harasta
Irbin
Hamorya
Hazeh
Saqba
Kafr Batna
Ein Tarma
Jereen
Al Mieha
Jaramana
Shebaa
Set Zaynab
Sbench
Az Zyabe
Khorbet
Al Tal

Figure 24

	When	Where	Target	Device Description	Observations	Estimated Device Size
1	2011 Dec 23	Kfar Sousah. Second blast also reported	Security services	Suicide car bomb	3m diameter crater: 10-50kg. Masonry infill panels totally breached at 11m 34 degrees: >10kg. No glass remaining across visible façade: >10kg. Initial signs of RC frame damage at 6m. None at greater distances although possible bracing from shear wall: 30kg	20-40kg
2	2012 Jan 06	Midan Sector	Bus carrying security personnel	Small device in boot of car or suicide bomber on bus	Damage not extensive. Local distortion of vehicle body panels. Annealed glass windows broken on an opposite building up to 3rd floor. 18m at 34 degrees broken. 21m at 45 degrees not broken: 2.5kg	2.5kg
3	2012 Mar 17	Central Damascus	Air Intelligence	Car bomb	Failure of masonry on the ground floor window surrounds which is less apparent at first floor level and not apparent at second and floor and the flanking face of the building.	5-10kg
4	2012 May 10	Al Qazzaz	Military Intelligence – Palestine Branch		8m diameter crater 50m from the façade. Failure of masonry on seven floors of the front face: >400kg. No masonry failure on the flanking sides: <600kg. Annealed glass broken in a building opposite across 200m of open ground: >400kg	400-600kg
5	2012 Aug 15	Central Damascus	Military Compound	Attached to oil tanker	Little evidence seen of damage other than to the fuel tanker where the tank had ripped open. Localised damage suggests a small charge size.	*<5kg*
6	2012 Sep 25	Umayyad Square. Second attack near	Military HQ	Suicide mini-van.	5m diameter crater: 50kg. Block wall and railing pushed over at 9m: >10kg. All glazing visible has broken up to 39m away:>10kg	10-50kg
7	2012 Oct 26	Daf al-Shouk			Blast in moderately confined street. Masonry infill panels totally breached up to 30m away and 4 stories up.	>75kg
8	2012 Oct 29	Jaramana (2nd in Al Hajar al Aswad)	Bakers / Residential area.		Blast in moderately confined street. Masonry panels breached in close proximity. Decreasing extent of breaching with height distance along street.	10-20kg
9	2012 Nov 07	Waroud (2nd in Qadaam)			Limited information. Collapse of masonry but the extents are unclear. Possibly some damage to RC column.	*<50kg*
10	2012 Nov 28	Jaramana main square		Device left in car followed by suicide car bomb. 2 other smaller blasts elsewhere	Confined street increases the complexity of estimating the blast load. Limited information from which to make assessment. Masonry has collapsed but extent is unclear	*<<50kg*
11	2013 Feb 21	Al Thawra St			Crater of 4-5m: 30kg. car bomb on multilane highway near to bridge. A building 40m from the blast had lost all windows: >10kg. On the same building there was some but limited masonry failure: 30-50kg. Metal sheeting running along the side of the bridge away from the point of detonation for 30m had detached nearest to the point of detonation and showed little deformation at 30m: 30kg correlates best.	30kg

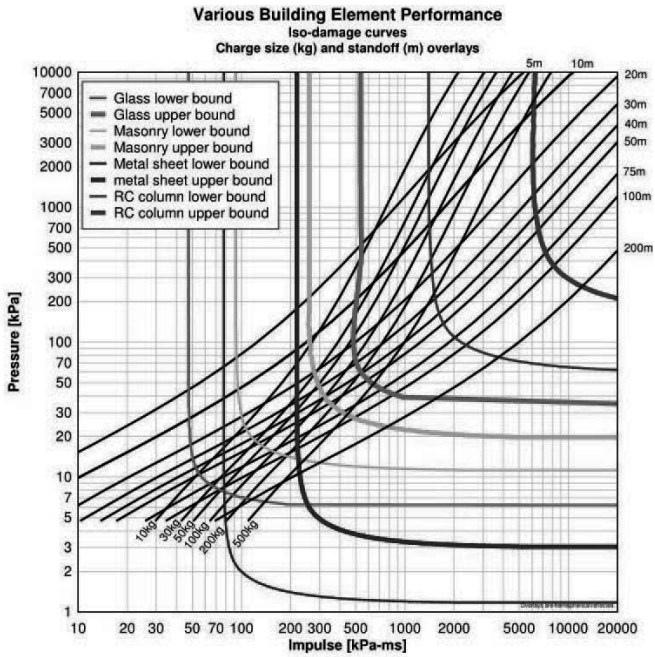


Figure 25

Case Study Summary

While there is variation between each incident, charge weights under 50kg appear to be most common. They have occurred mainly in the Central and Southern areas of Damascus and they are frequently reported as being carried out at the same time as another attack. Figure 26 shows the variation of charge weight used over time.

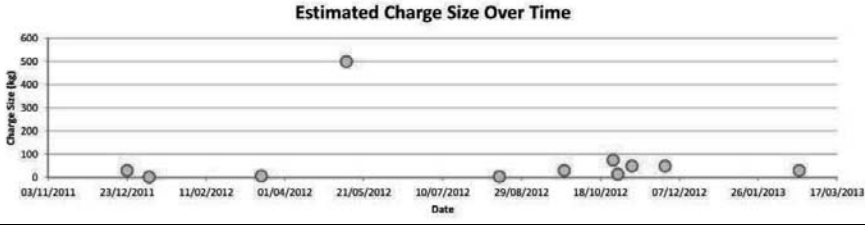


Figure 26

Conclusions

While there is no replacement for first hand structural site surveys when rigorously detailed information is required, this paper has sought to demonstrate that there are options available that allow valuable information to be collected that might otherwise be missed. Figure 27 shows a pyramid of trade-off between quantity and detail and it is intended that the three case studies discussed demonstrate that the different levels of the pyramid offer a valuable contribution.

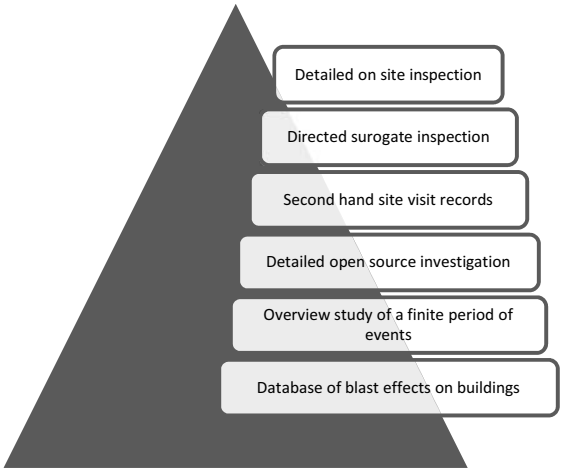


Figure 27

The advances in communication via the internet, access to information and permeability of the media has made remote investigations, especially open source investigations, far more viable. Further advances may also increase the potential of these techniques: LIDAR scanning and picture cloud meshing will improve remote visualisation; smart phones and social media are providing up to the minute first-hand account, as long as they can be relied upon.

Various factors have been discussed that help improve the quality of the analysis. These include urban density offering quantity and variety of damage, public accessibility and post incident control, media interest in the event and availability of open source imagery, first hand-knowledge of the area or descriptive journalism.

It is envisioned that such techniques will allow customers and practitioners to be better informed where time, cost or practicality prevent first-hand detailed inspection. It could also form the initial phase of more detailed follow on work.

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Forensic Engineering: Informing the Future with Lessons from the Past
ISBN 978-0-7277-5822-4

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doi: 10.1680/feng.58224.195

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Food market northern from Montreal – Investigation after a failure of the roof under the snow load

Stefan Zmigrodzki P.Eng.M.Sc., CIMA+ Consulting Engineers, Montreal, Canada

Introduction

The building discussed in this paper houses an important food market northern from Montreal. The structure is a steel frame construction supported by a conventional system of foundation walls and footings. Originally built in 1982, the structure was expanded three times in 1987, 1995 and 2002. While the building had been a single-storey structure since its opening in 1982, the 2002 expansion project included a second floor in the newly constructed section of the building, as well as architectural turrets on the building's facade. Figure 1 presents an aerial view of the structure with the outlines of the original construction, as well as those of the three expansions.

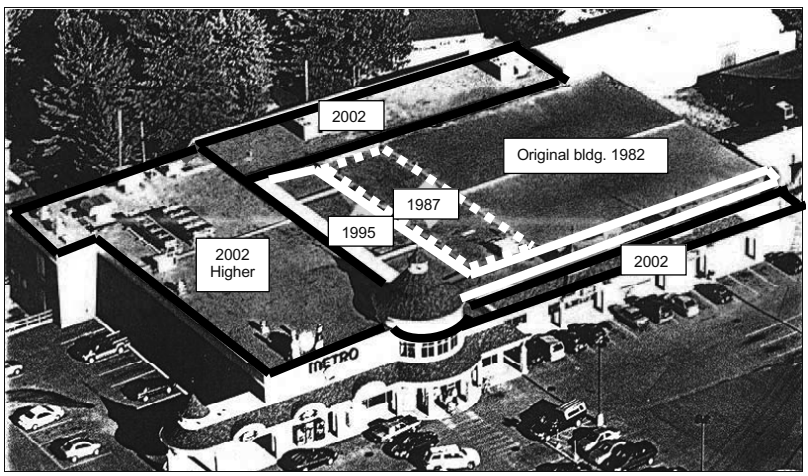


Figure 1. Food market – Construction phases

On 15 March 2005, three years after the most recent expansion, a portion of the roof originally built in 1982 settled under the snow loads. The owner of the building had the roof strengthened, and then claimed the costs of the work with their insurance company. The claim was rejected on the grounds that the building did not meet the requirements of the building code, and that structural problems integral to the original structure had led to the failure. Following the insurance company's decision, the owner hired Consulting Engineers CIMA+ to evaluate the state of the structure and to provide an independent opinion with respect to the causes of the roof failure, and the quality and integrity of the structure as a whole.

Assessment of causes of the roof failure of the roof

Despite their poor quality, the photographs in the report obtained from the insurance company indicate the method of failure of the roof structure. The photographs, reproduced in Figure 2, show that the bottom flanges of two beams buckled at their support columns. As the beams were continuous over the columns, the bottom flanges of the beams were in compression and lateral support should have been provided at these locations to prevent lateral buckling. Such lateral supports were missing and the flanges failed as indicated by the arrows in Figure 2. The beams in question were both parallel to the building's façade. The lateral supports had been absent since 1982 but no attention was paid to these structural elements by the engineers in charge of the expansion projects.

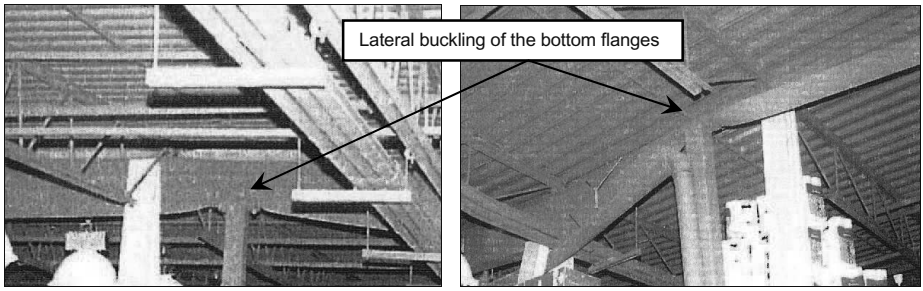


Figure 2. Roof structure after the failure

Changes to the roof's geometry in 2002

The 2002 expansion project included a second floor in the newly constructed section of the building, as well as architectural turrets on the building's facade. These changes in the building's configuration, most notably a roof with locally elevated sections, produced an increase in snow loads on the roof and higher stress in the beams which led to the failure. It is interesting to note that the position of the failure in both beams is aligned with one of the architectural turrets. Figure 3 shows the view of the building facade and an aerial view of the roof. The white marks on the roof indicate the locations of the beam failures.



Figure 3. Market – Façade (left) and roof (right) in 2002

The new roof configuration had a significant impact on the snow load. Previously, the relatively smaller and single-storey roof of the original building had been unobstructed and was exposed to winds which swept away part of the snow and reduced accumulation. Following the additions

of the 2002 expansion project, the original section of the roof became partially obstructed by the architectural turrets and the additional storey on the side of the building. The elevated section and the turrets acted as shield, reducing wind action, and increasing snow accumulation. Figure 4 summarizes increase in surface area of the supermarket during its life time. The change is expressed in 3 ways: as a net increase in roof area in square meters, as a percentage of the area of the building at the time of expansion, and as a percentage of the area of the original construction. The table also indicates which edition of the National Building Code of Canada (NBCC) was in effect during each year of construction.

Market expansion				
Item	Expansion projects			
	1982 Original	1987	1995	2002
Expansion project (m^2)	930	390	437	1 300
After expansion (m^2)	930	1 320	1 757	3 057
% of existing		142%	133%	174%
% of 1982	100%	142%	189%	329%
National Building Code of Canada in force - Edition				
	1980	1985	1990	1995

Figure 4. Market expansion

Figure 4 shows that each of the expansion projects represented a substantial increase in the roof area, but the 2002 project was by far the most important. Following the last expansion, the building's footprint nearly doubled in size when compared to its area in 1995, and more than tripled that of the original construction. The combination of increase in surface area and the changes to the roof's configuration significantly reduced the exposure to wind and resulted in a different snow load pattern on the roof.

Changes to the calculation of snow loads in the National Building Code of Canada (NBCC)

The formula for calculating snow loads on a roof in 2002 was different than that in effect in 1982. The inclusion of rain loads in combination with snow loads, as well as new parameters to reflect the effects of slope and snow accumulation in the vicinity of elevated roof elements, had been added to the provisions of the National Building Code of Canada (NBCC). Figure 5 shows the formulas in use in 1982 and in 2002 as well as the values of the snow load obtained from those formulas for the section of the roof of the original building.

Snow load (kPa) Load based on the National Building Code of Canada (NBCC)	
1982 - NBC 1980 exposed roof	2002 - NBC 1995 no exposed roof
So (ground load in kPa) = 2.7 kPa Cs (wind exposure factor) = 0.6	Ss (ground load in kPa) = 2.4 kPa Cb (basic roof load factor) = 0.8 Cw (wind exposure factor) = 1.0 Cs (slope factor) = 1.0 Ca (accumulation factor) = 1.0 Sr (associated rain load in kPa) = 0.4 kPa
So • Cs = 1.62 kPa	**Ss • (Cb • Cw • Cs • Ca) + Sr = 2.32 kPa** **Compared to 1982 : 143%**

Figure 5. Snow loads on the roof obtained from the 1980 and 1995 versions of NBCC

Figure 5 shows that the snow load on the original portion of the roof at the time of the design of the 2002 expansion was 43% higher than that used in the original 1982 design. This increased load should have been used to verify the capacity of the existing roof structure. It is obvious that the verification was not done and the higher snow load resulted in a new load pattern on the roof following the 2002 expansion. This new load pattern ultimately led to the failure of beams.

Structural verifications in 2002

The structural plans of the roof for the 2002 expansion, reproduced in Figure 6, show both the new and the existing structures. The expansion included both a two-storey section (solid line), and two single-storey sections at the same elevation as that of the existing structure (dotted lines). Two dark rectangles in the middle of the existing structure indicate the location of the failure.

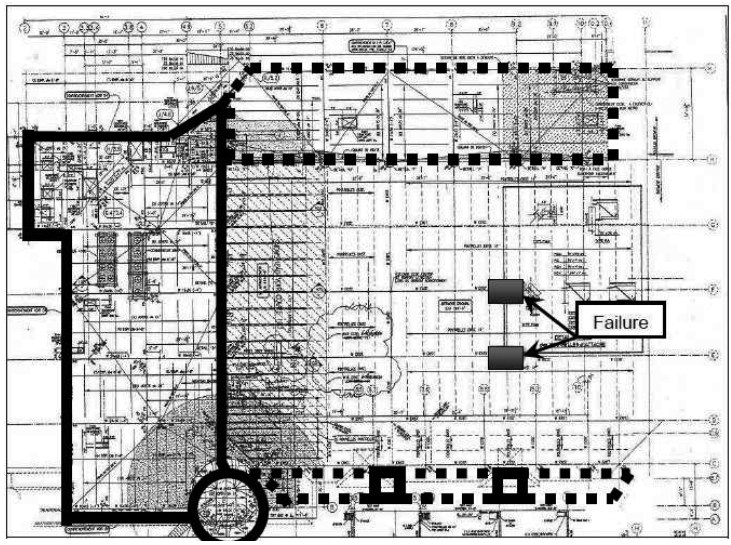


Figure 6. Roof structures as shown on the plan of the 2002 expansion

A detailed review of the structural drawings revealed that no expansion joints had been installed between the new and existing structures, nor were there any expansion joints within the existing structure prior to 2002. Following construction, the building therefore acted as one single structural system, requiring the capacity to support all of the loads as determined by the NBCC. The entire structure therefore became the responsibility of the structural engineer in charge of the 2002 expansion project.

Summary

The actual depth of snow on the roof the day of the failure is not known so the real load cannot be compared to the theoretical one. However, the real load must have been higher than that calculated in 1982 as it caused a failure by the lateral buckling of two improperly supported beams. The lack of lateral supports would have been noted if an appropriate verification of the roof's capacity under heavier snow loads had been performed by the engineer in charge of the

2002 expansion. The footprint and the geometry of the building changed dramatically during that project, and affected what became a new single cohesive structural system. This new structural system's capacity should have been verified not only against the new snow loads but for all other loads in accordance with the building code in effect. An appropriate strengthening of certain members should have been done to make the new structure conform to the provisions of the code. Appropriate lateral supports for the beams which ultimately failed should have been part of such a strengthening.

General investigation – Lateral stability

While the initial investigation focused on the causes of the failure during the winter of 2005, the findings of that investigation led to the assumption that there were other structural deficiencies that had not yet appeared. It should be noted that for this type of structural system, earthquake loads are those that govern the design of structures in the Province of Quebec. As mentioned earlier, the series of expansions that the building underwent during its life time did not include the installation of expansion joints between the new and existing sections of the structure. This means that, from a seismic point of view, after each expansion project, a different structural system was created as a result of the changes made to the building. It was decided therefore to review the stability of the building as it evolved over its life time, particularly during the 2002 expansion.

Changes to the structure's lateral bracing

Determining the complete history of the structure's lateral stability proved to be a difficult task considering the fact that the building was designed and constructed in four phases by different engineers who, theoretically, followed four different editions of the building code. As each new part of the building was built without an expansion joint, the existing structure was wholly integrated in the new system. And since the 2002 expansion project included no modifications to the existing part of the building, the new and final structural system inherited all of the previous potential problems. The analysis began with a review of the plans from each project.

Original construction (1982) and first expansion (1987)

The first expansion in 1987 was a simple addition to one side of the original building, as shown in Figure 7. There were no structural or architectural plans on file from 1982, but the structural drawings of the first expansion in 1987 provided enough information about the structure's lateral bracing at the time.

The notes on the drawing details of the wall that was demolished during the addition and those of the new exterior wall are reproduced in Figure 8 with the original text in French translated into English.

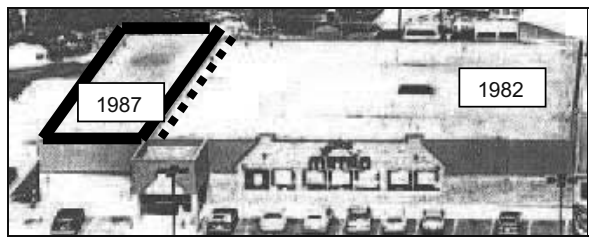


Figure 7. Expansion in 1987

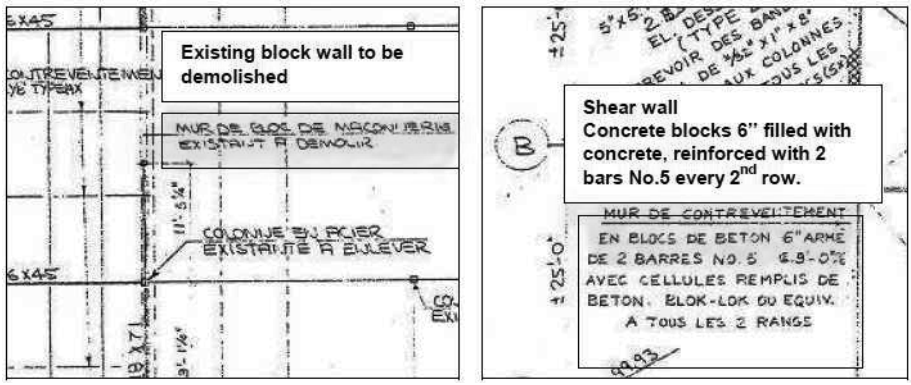


Figure 8. Notes regarding the side wall built in 1982 to be demolished (left) and the new side wall built in 1987 (right)

It was determined that the original building had been braced by concrete block shear walls; the location of the walls inside the building however was unknown. In 1987, as one side wall was demolished (dotted line on Figure 7) to make room for the growing supermarket, a new concrete block wall was raised as part of the new side wall. While the building was braced laterally by concrete blocks, the question of how the new roof was connected to the existing one to form a larger diaphragm was unclear.

Second expansion (1995)

The second expansion in 1995 included a further addition to the same side of the building that had expanded in 1987, as well as a narrow addition in front of the original facade, as shown in Figure 9.

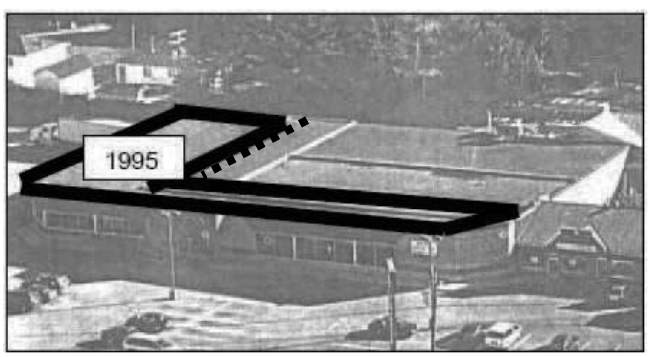


Figure 9. Second expansion (1995)

As with the original construction, no structural plans were available from 1995 expansion. The only information obtained about bracing was found on an architectural drawing of the exterior walls. The section through the exterior wall is shown in Figure 10.

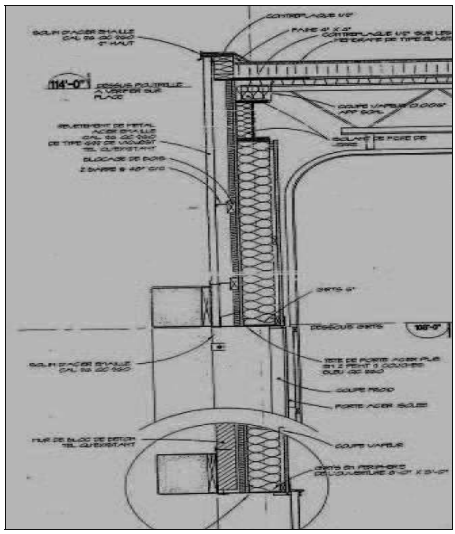


Figure 10. Second expansion (1995) – Section through the new exterior wall

The new exterior side wall was made of light-gauge steel framing; the concrete blocks in use in 1982 and in 1987 for exterior walls were no longer part of the new facades. It is not known if any steel cross bracing was added to the new wall because this wall was demolished in 2002. If this was the case, then the building was laterally stabilized by two different types of bracing. This fact should have, at the time of the 1995 expansion, given rise to serious questions about the seismic response of the building. The rigidity and seismic response of steel bracing is very different from that of concrete shear walls. The resulting earthquake loads and their distribution between the bracings are also different and this aspect should have been addressed in 1995.

The third expansion (2002)

The third and final expansion in 2002 was the most important. The new section added to the side of the building that had twice already been expanded, included a second storey. Two additional sections were added at the back and at the front of the building, the latter with architectural turrets. The third expansion is shown in Figure 11.

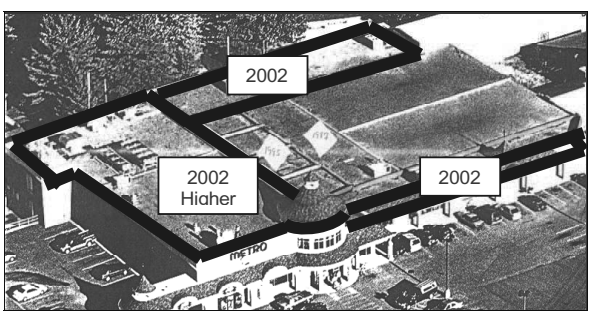


Figure 11. Third expansion (2002)

The 2002 expansion was well documented. The engineering plans presented enough clear information about the structure and its bracing. Figure 12 shows the location of cross bracings on the roof and on the 2nd floor in the 2 storey sector of the building; the cross bracings are indicated by black rectangles.

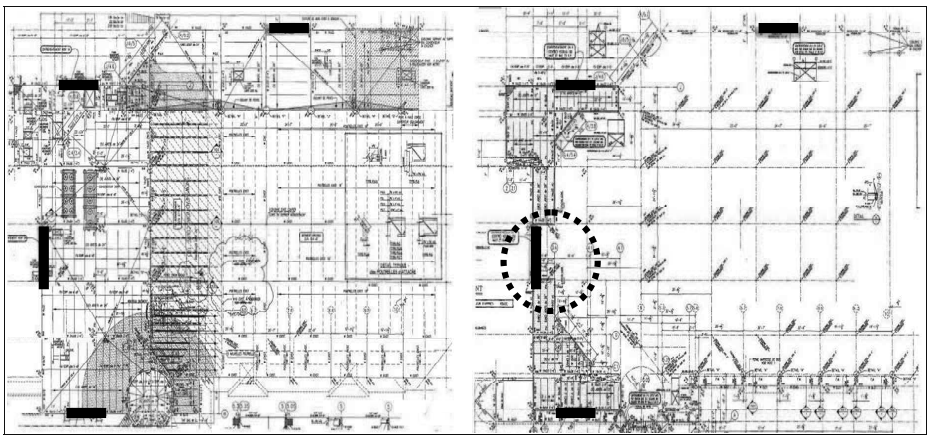


Figure 12. Two-storey section of the third expansion (2002) – Location of cross bracings on the roof (left) and 2nd floor (right)

As shown on the plan of the 2nd floor, there were two independent parts of the floor and the bracing was installed between them (dotted line around). Because of the basic consideration of static equilibrium, both parts of the floor should have had three bracings; it was not the case and the bracing as shown was ineffective. Figure 13 shows that the bracings are made of HSS (Hollow Structural Section) members.

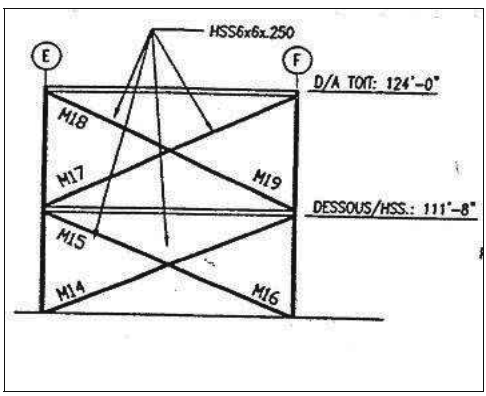


Figure 13. Third expansion (2002) – Typical cross bracings

Other problems related to the structure's lateral bracing were found on the foundation plan. A cross section of the foundation wall with the details of a column and its anchor bolts is presented in Figure 14.

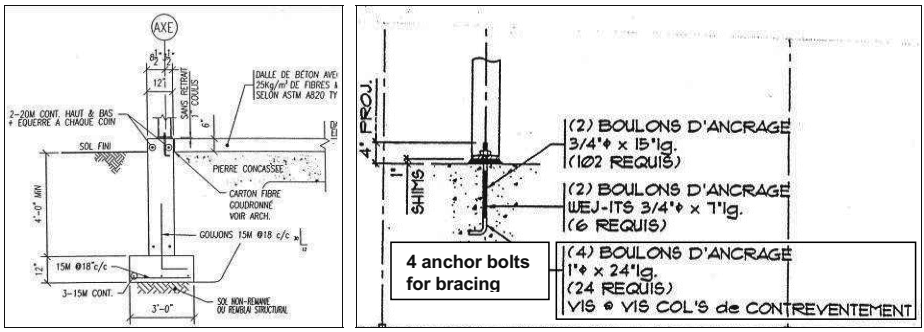


Figure 14. Third expansion (2002) – Detail of the base plates and anchor bolts

As shown in the section of the foundation wall, no rebar was added to the concrete in the wall. The wall's ability to resist uplift forces from the columns was therefore governed by the concrete's tensile strength. This principle not only drastically limited the wall's resistance against such loads, but it was not in accordance with the building code. The use of unreinforced concrete is prohibited in such cases. The detail of the anchor bolts shows that there were four bolts aligned in a row. Because of the flexibility of the base plate, only the two bolts nearest the columns were effective; the expected capacity was therefore reduced by half.

An additional problem discovered in the 2002 expansion plans was the connection of the new parts of the structure to the existing building; Figure 15 shows a section through grid line between the new and the existing roofs.

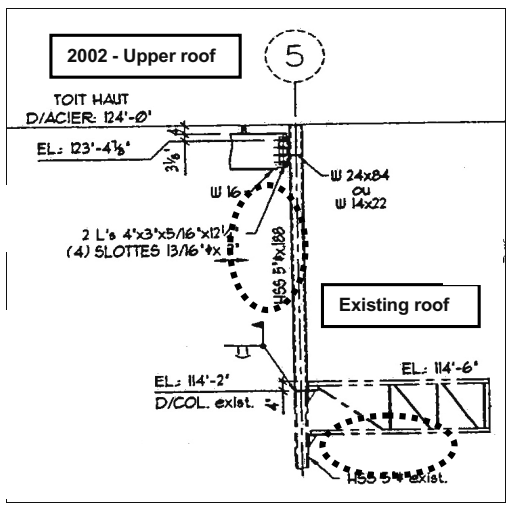


Figure 15. Third expansion (2002) – Extension of the existing columns

The detail on Figure 15 clearly shows that the existing HSS (Hollow Structural Section) columns were elongated by welding extension members to the existing elements. The new columns became components of two different structural systems with different seismic

responses and oscillation periods. This is a very unusual concept which should have required a dynamic analysis and a careful evaluation of all members and connections to verify if such a system could perform as anticipated. This evaluation was not performed. The columns presented a very high risk of a progressive failure during an earthquake.

Summary

The design concept of the 2002 expansion led to the conclusion that the stability of the structure in its final form had not been investigated by the engineer in charge. Effectively, the new sections built in 2002 were not properly designed to carry the governing lateral loads in accordance with the building code. The stability of the building does not appear to have ever been addressed throughout any of the expansions, even during that in which a substantial increase in volume and mass was probably the most important as far as the earthquake loads are concerned. An investigation of the stability of a building that had already been altered by previous expansions and showing symptoms of weakness was not performed when it should have in 2002, and the new structure formed from this phase induced even more problems. The safety of the building following the expansion project in 2002 became an important issue that had to be reviewed in detail.

Conclusions

The findings from the investigation not only gave sufficient support to the insurance company's decision to reject the owner's claim, but they also shed a new light on the quality of the entire structure.

The findings and the needs for further investigation which would probably lead to additional strengthening requirements were presented in a final report that was reviewed with the market's owner. Following the acceptance of the report, a lawsuit was launched by the owner against the engineer in charge of the 2002 expansion project to recover the costs of needed repairs.

The case presented in this paper shows the impact that limited design considerations within expansion projects can have on the life cycle performance of a structure. It shows the influence that changes to a roof configuration can have on snow loads. It also reveals the impact of not having expansion joints between new and existing sections of a building on its seismic response before and after the expansion projects.

Engineering skills during expansion projects are a very important issue. This paper demonstrates how negligence or limited knowledge can lead to critical situations. In the present case, such a situation could have been avoided if the project had been a reviewed by another engineer. This would have probably prevented the failure and eliminated the risks borne by the structure.

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SECTION 4:

CASE STUDIES – GROUND AND BELOW GROUND WORKS

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ISBN 978-0-7277-5822-4

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doi: 10.1680/feng.58224.207

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Investigation of Collapse of Slurry Trench in Underground Metro Construction

Makarand G. Khare Principal Engineer, AECOM India Pvt Ltd, Chennai, India
M Ramanathan Associate Director, AECOM Asia Co Ltd, Chennai, India

Introduction

The diaphragm wall installation is a first step in construction of a typical cut-and-cover metro station. The diaphragm walls for underground metro station are required to be constructed with a minimum intrusion beyond the station footprint because of the limited land availability in congested urban landscape. The slurry trench technique is the most widely used construction method for diaphragm walls. In this technique, a trench is excavated and simultaneously filled with bentonite/polymer slurry. A carefully excavated slurry trench results in little or no soil disturbance (Parkinson and Gilbert, 1991). However there are reports of ground subsidence or even severe cave-in of slurry filled trench excavations (Hajnal et al. 1984 and Filz et al. 1997). The trenching process disturbs the in-situ state of equilibrium in soil mass resulting in inward movement of the trench face. The slurry filled trench is subjected to lateral earth pressure which includes the effects of surcharge load (due to construction and/or vehicular traffic and adjacent buildings) and groundwater pressure. The excessive lateral pressures can lead to collapse of the trench faces. The instability of slurry trench can also threaten the safety of nearby structures. Therefore, the stability of slurry trenches during construction of diaphragm wall is of paramount importance in sites where water table is close to the ground surface, soil is weak, and structures generating high surcharge loads are in close proximity.

Two cases (referred to as Case 1 and Case 2) of slurry trench collapse during the construction of underground metro stations are discussed in this paper. A polymer based artificial slurry was used in Case 1 and bentonite slurry was used in Case 2. The stations referred in Case 1 and Case 2 were constructed by different Contractors.

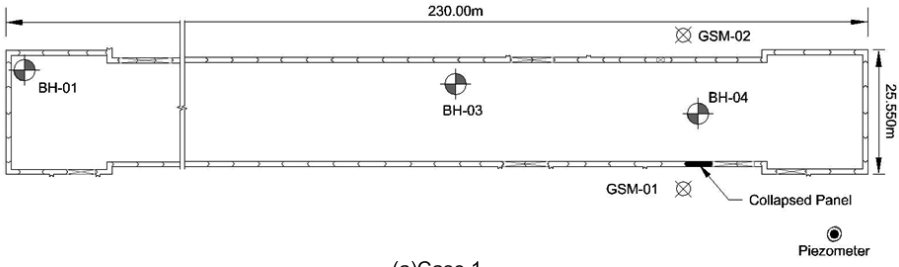
In case of a trench supported by bentonite slurry, the slurry permeates into soil fabric and forms an impermeable thin film (also called as filter cake) along the wall. The impermeable film helps to transfer the hydrostatic pressure on the trench face and gives the stability to the trench. The shearing resistance of the slurry saturated zone and the plastering effect of the filter cake formed on the trench face also contributes towards the trench stability.

The polymer based artificial slurry behaves as a pseudo-plastic fluid and unlike bentonite slurry, achieves the trench support without forming a filter cake (Kaul, 2010). The polymer based slurries form a membrane on the trench face which enhances the stabilization by transferring hydrostatic pressure to the walls of excavation. The high concentration of negative charges on soil particles helps the binding of the soil with the positive charge present on long carbon chain of polymer fluids. The bond between soil and polymer gives additional stability to the trench.

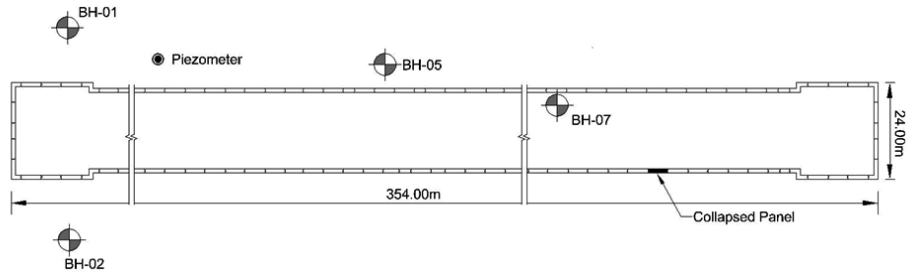
Site Geology

The regional geology is characterised by an easterly sloping coastal plain. The eastern and southern parts of the area have shallow bedrock, while the central and northern parts have zones of deep bed rock. The greater part of the district is covered by Pleistocene to Holocene alluvium of rivers in the north and in the south. The thickness of alluvium varies from a few meters in southern parts to more than 50m in central and northern parts. The average depth to bedrock in the district varies from 20m to 25m. The alluvium consists of silty sand, sandy clay, silt and occasional gravels and is underlain by crystalline rocks. The natural ground level is about +6.0 m MSL (mean sea level). The ground water table is shallow and rises close to ground level during rains.

Figure 1 shows diaphragm wall panel layout, and locations of boreholes, collapsed panel, ground settlement markers and piezometers. The station box consists of two end shafts (used for launching and retrieval of tunnel boring machines) which are about 3m wider than the middle portion of the station. Table 1 shows the geological profiles for Case 1 and Case 2. The geological profiles are deduced from nearby borehole and the panel borelog. Figure 2 shows the distribution of standard penetration test blow count (SPT 'N_{60}') and the percentage of fines (i.e. silt and clay) content obtained from the sieve analysis of disturbed samples collected from nearby boreholes.



(a)Case 1



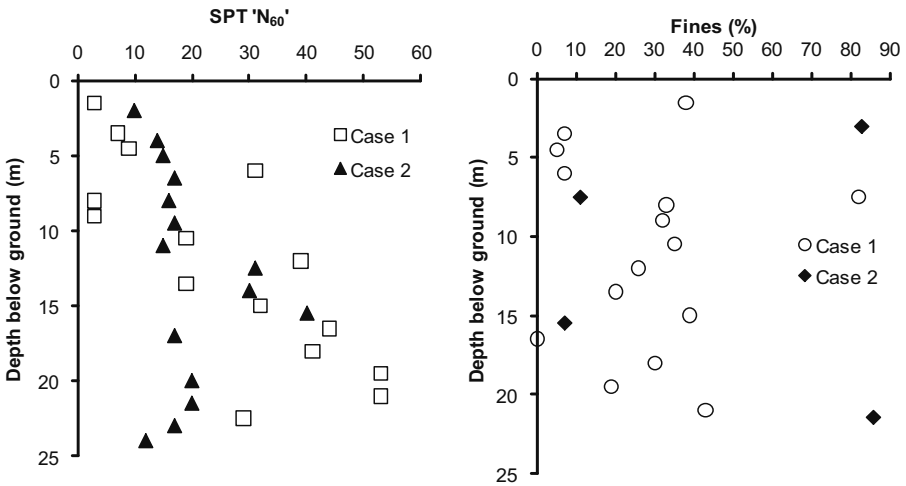
(b)Case 2

BH- borehole, GSM- ground settlement marker

Figure 1 Panel layout, borehole locations, piezometer and ground settlement marker locations

Table 1. Geological Profile

Case No.	Depth below ground level (m)	Description of Stratum
Case 1	0 to 2.5	Made ground, pavement
	2.5 to 8.5	Silty Sand
	8.5 to 10	Silty Sandy Clay
	10 to 22	Silty Sand
	22 to 24	Silty Sandy Clay
	24 to 27	Completely Weathered Rock
	> 27	Fresh Rock
Case 2	0 to 4	Low to high plastic Clay
	4 to 17	Silty Sand
	17 to 28	Low plastic Clay



a) SPT 'N_{60}' with depth

b) Fines (silt and clay) % with depth

Figure 2 Engineering properties of soil

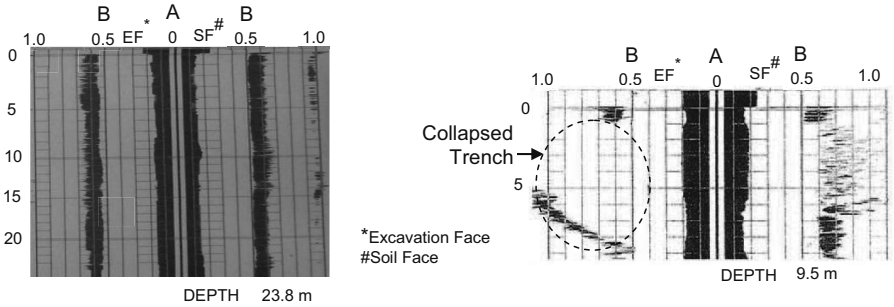
Case 1

The collapse of a 5m long and 1m thick diaphragm wall panel with toe depth of 25.2 m below ground level is described here. Figure 1 (a) shows diaphragm wall panel layout, and locations of boreholes, collapsed panels, ground settlement markers and piezometers. The trench excavation was carried out using hydraulic grab. The panel was being excavated with two 2.5 m long cuts. The first cut was excavated to the design depth of 25.2m. In the second cut, the weathered rock was encountered at a depth of 24.8m below ground. The weathered rock could not be excavated with grab and therefore a sharp edged drop weight was used to chisel the rock. The chiselled rock was then removed by the grab. The completion of panel was delayed because the rock chiselling was hampered during heavy rains.

This station was being constructed on a busy road. Therefore, the diaphragm wall reinforcement cage fabrication was carried out in an open barricaded area on the road. Due to heavy rains the welding of reinforcement cage was deferred. A public holiday further delayed the completion of the trench. On the sixth day (after the start of trench excavation) when the work resumed after holiday, the trench was found to have collapsed at a depth of about 7 m below ground. The contractor carried out KODEN test up to depth of 6.3m in the first cut and 9.5m in the second cut. The results of

KODEN test carried out on the second day of excavation and that after the collapse (on the sixth day) are shown in Figure 3.

The KODEN test is based on the principle of ultrasonic wave travel from sensor to the wall of trench and reflection of the wave from trench wall back to the sensor through bentonite/polymer slurry media. The time taken by the wave to travel in the slurry and the wave velocity is then interpreted to assess the geometric profile of the trench. The vertical line (A) shown in Figure 2 represents the vertical descent path of the probe. The trench wall profile is shown by the inner borders of dark strips (line B). Figure 3 shows the collapse at about 1m below ground and extending to a depth about 9.5m. The collapse occurred on the excavation face (EF) of the diaphragm wall (i.e. the inner face of underground station).



a) Before collapse

b) After collapse

Figure 3 KODEN test results – Case 1

The grain size analysis of samples obtained from the nearest borehole show loose silty sand with SPT 'N_{60}' generally less than 10 and with fines content less than 10% (as shown in Figure 2) upto a depth of about 10m below ground level. The low fines content could have possibly reduced the membrane like effect of slurry which controls the fluid loss and helps in building significant positive hydrostatic pressure from slurry to the excavated face of the trench. The heavy rains caused the rise in water table close to a depth of 2.5m below ground level resulting in reduction of net positive slurry head and the factor of safety for upper 10m of loose sandy soil. The polymer manufacturer recommended slurry unit weight of 10 to 10.4 kN/m^3. The polymer slurry viscosity was maintained at 54 seconds and the slurry unit weight was maintained at about 10 kN/m^3 at the time of collapse, i.e. at the lower end of the recommended range. The factor of safety for the upper 10m thick loose sand layer at the time of collapse is estimated using the approach suggested by Schneebeli (1964). Schneebeli (1964) investigated the effect of arching on short panels on the basis of the theory of lateral earth pressure on silos (Caquot and Kerisel, 1956). In this method, the factor of safety is obtained by comparing the lateral earth pressure (with allowance for arching effect) diagram with the net fluid pressure (slurry pressure minus groundwater pressure) diaphragm. In case of cohesionless soils (c'=0, ϕ' soils) the factor of safety at the water table level and at the base of the trench is obtained using following expressions:

At water table level, factor of safety (F) = $\frac{\gamma_f . D_w}{a.(1-b)}$ ----------(1)

At base of trench, factor of safety (F) = $\frac{\bar{\gamma}_{f.}H_f + \gamma_{w.}D_w}{a.c}$ ----------(2)

Where,

$$a = \frac{\gamma_{d}.L}{N_{\emptyset}.\sin 2\emptyset'}$$

$$b = e^{-n.\sin 2\emptyset'}$$

$$c = \left[1 - \left[1 - \frac{\gamma_{sub}}{\gamma_{d}}\right].b\right]$$

$\bar{\gamma}_f$ = effective unit weight of slurry = $\gamma_f - \gamma_w$

D_w = slurry − water differential head = $H_f - H_w$

n = z/L

γ_f = unit weight of slurry

γ_w = unit weight of water

H_f = depth of slurry

H_w = depth of water

z = depth of soil layer analysed

L = length of trench

γ_d = dry unit weight of soil above water table at a depth z

γ_{sub} = submerged unit weight of soil below water table at a depth z

ϕ' = effective friction angle of soil

N_ϕ = flow factor = $(1+\sin\phi') / (1-\sin\phi')$

The slurry properties and shear strength of soil used for the analysis are given in Table 2. The Figure 4 shows the factor of safety calculated at various depths using equations 1 and 2. The depth of water table is considered at 2.5m below ground level as recorded by the nearby piezometer. The slurry unit weight at the time of collapse is taken as 10.0 kN/m^3. It is evident that the factor of safety was less than 1.0 when water table was just 2.5m below ground. The Schneebeli (1964) approach accounts for the arching effect. In case 1, the trench was open for 6 days, therefore the beneficial effect of arching could have reduced with time. The creep effect could have reduced the shear strength of upper loose sand than that assumed in the back analysis. Therefore, the actual factor of safety at the time of collapse could have been less than those shown in Figure 4.

Table 2. Parameters used for back analysis

Property	Case 1	Case 2
Bulk weight of soil (kN/m^3)	17	18
Unit weight of slurry (kN/m^3)	10, 10.2	10.4, 10.9
Effective friction angle of soil ϕ' (degrees)	30	32

The contractor implemented following measures to increase the factor of safety:

a. The unit weight of slurry was increased from 10 kN/m^3 to 10.2 kN/m^3 to increase the slurry pressure.
b. The slurry head was maintained at about 3m above water table. For this purpose a 0.5m high bund made up of sand filled bags was created around the trench as shown in Figure 5. The bund helped to keep a higher positive slurry head when water table was close to ground level.
c. The reinforcement cage was made ready before the start of trenching to reduce the cycle time of diaphragm wall panel construction.

The factor of safety is again estimated to reflect the corrective measures. The factor of safety estimated using higher slurry unit weight (of 10.2 kN/m^3) and that considering a higher slurry head as well as unit weight are shown in Figure 4. From Figure 4, it is observed that the increase in the slurry unit weight alone resulted in a marginal increase (of less than 8%) in the factor of safety. The increase in the slurry head combined with increased slurry unit weight resulted in about 25% increase in the factor of safety. In this case, the increase in slurry head played major role in increasing the factor of safety of trench. It is evident that these simple and inexpensive measures resulted in substantial increase in the stability of the top 10m of loose sandy soil. A large number diaphragm wall panels were completed successfully by implementing above measures.

Figure 1 show a settlement marker (GSM 01) installed on the pavement and very close to the collapsed trench. The settlement marker did not show any movement as a result of trench collapse. A membrane like effect provided by the upper crust of pavement appears to have spanned over the collapsed soil. The relaying of road over the older paved surface (during maintenance by the municipal corporation) may have resulted in strong yet flexible upper crust with a limited capacity to span over the cavity.

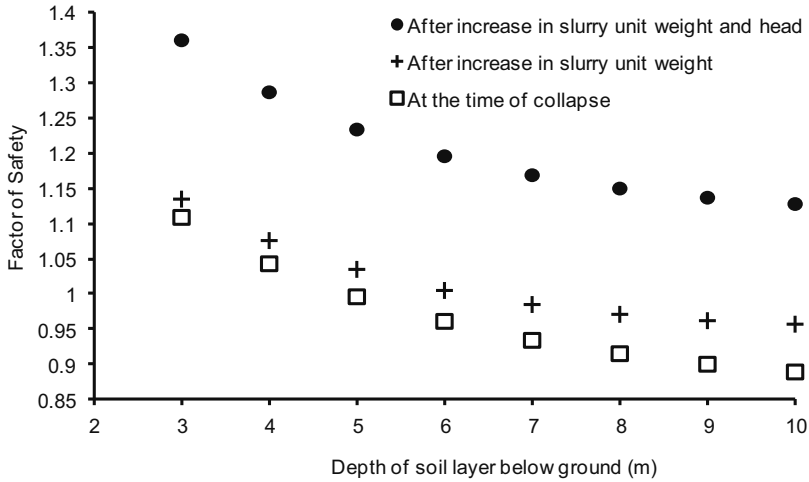


Figure 4 Estimated factor of safety – Case 1

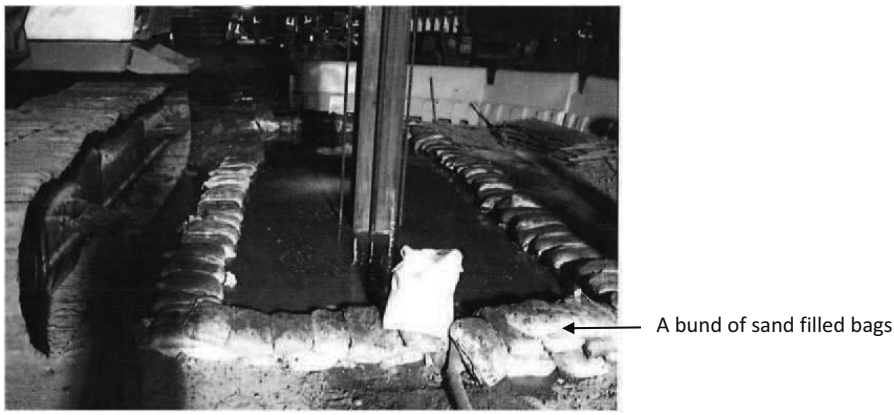


Figure 5 Bund to increase the positive slurry head

Case2

The collapse of 4.5m long and 1m thick diaphragm wall panel with toe depth of 22.6m below ground level is described in this section. The locations of collapsed panel, boreholes and piezometer are shown in Figure 1(b). A week long heavy rainfall resulting in the rise of water table was the primary reason for the collapse of trench in Case 2. The rise in water table reduced the factor of safety in silty

sand observed from a depth of 5 m to 15m. From Figure 2, it is observed that the fines content in this stratum is about 10% and a typical SPT 'N_{60}' value is about 15. The records of ground water levels during the trenching and immediately after the collapse are not available. The available groundwater level record shown in Figure 6 indicates that at the time of trench collapse, the groundwater was likely to have been at a depth of about 2m below ground level. The geological profile given in Table 1 and the soil parameters given in Table 2 are used to investigate the effect of rise in water table on the factor of safety. The factor of safety is calculated using Schneebeli's method described in the preceding section. The factor of safety for the soil at a depth of 10m below ground (which corresponds to the middle of upper sandy stratum) is analysed and the results are shown in Figure 7.

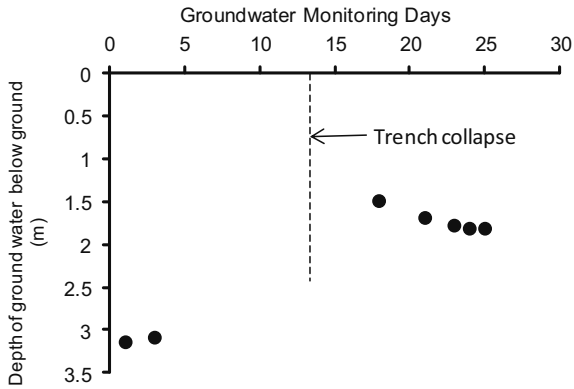


Figure 6 Ground water monitoring data – Case 2

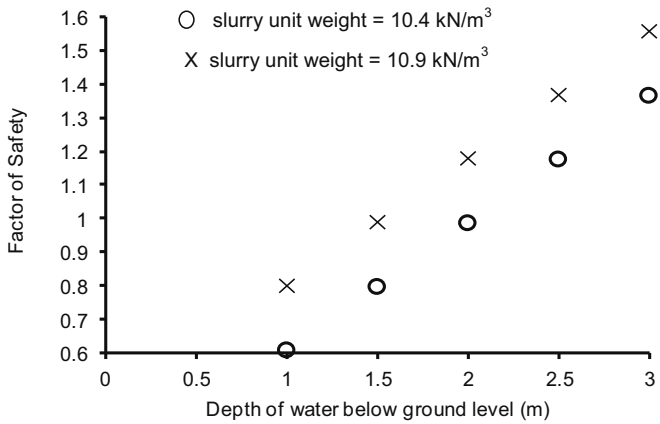


Figure 7 Effect of rise in ground water on Factor of Safety – Case 2

From Figure 7, it is evident that when the water table rose from 3m to 2m below ground level the factor of safety dropped to less than 1.0. The maximum water table recorded was just 1.5 m below ground. At this site, the contractor decided to increase the slurry unit weight from about 10.4 kN/m^3 to about 10.9 kN/m^3 (which corresponds to the higher end of the range of 10.4 to 11.0 kN/m^3 recommended by Contractor's method statement). The increased slurry unit weight (of 10.9 kN/m^3) could counter the high hydrostatic pressure even when the water table rose to about 1.5m below

ground level as is evident from Figure 7. From Figure 7, it is observed that the increase in slurry unit weight improved the factor of safety by about 31% and 14% when water table level was at 1m and 3m respectively. The deeper water table results in increase in the lateral earth pressure and reduces the beneficial effect of increase in slurry unit weight on the factor of safety.

Increasing the unit weight of slurry beyond a certain value could hamper the displacement of slurry by concrete. The bentonite concentration is usually kept in the range of 4-8 percent (which represents the slurry density in the range of about 10.2 to 10.4 kN/m^3) so that the desired rate of flow of concrete can be maintained without compromising on the formation of filter cake on the excavated face. The rising concrete during tremie concreting may not be able to dislodge the filter cake from the trench face (if the slurry weight is high) and reduce the skin friction. Therefore it is prudent to adopt a combination of measures which may include increasing the unit weight of slurry and increasing the slurry head in order to improve the factor of safety without compromising the performance of diaphragm wall.

Conclusions

The safety of diaphragm wall trench excavation is of prime importance in underground metro projects. The stability of slurry trench can be compromised due to unfavourable geological profile or rise in water table. Two cases are described where the collapse of a slurry trench was observed. The factor of safety for the collapsed trench was back-analysed to investigate the cause of failure. The primary factors responsible for the instability of slurry trench were the rise in water table which lead to reduction in positive slurry head and prolonged trenching operations due to poor construction planning. The risk of trench collapse appears to be independent of the type of slurry (i.e. bentonite or polymer). The corrective actions were aimed to increase the margin of slurry pressure over the earth pressure acting on the trench wall. A bund using sand filled bags was created up to 0.5m above guide wall top so that slurry head can be maintained at least 2.5m above water table. The construction activities were planned to minimise the overall time cycle for the diaphragm wall panel construction. A combination of measures such as increasing the slurry unit weight and slurry head were used to improve the factor of safety by about 25%. The measures to prevent the collapse of slurry trench require very little resources and cost in contrast to the consequences in the event of collapse.

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ISBN 978-0-7277-5822-4

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doi: 10.1680/feng.58224.215

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Failure cost analysis of 50 deep excavations in The Netherlands

Korff, Mandy, Deltares/Cambridge University, Delft, The Netherlands/Cambridge,UK
Tol, A. Frits van, Deltares/Delft University of Technology, Delft, The Netherlands

Introduction

This article describes the analysis of about 50 deep excavations in The Netherlands, for which a claim situation has arisen in the past two decades. The cases are taken from the authors' own experience or from immediate colleagues. The authors were involved in each of these excavations, in some cases in the role of designer, but in most cases as an external expert, adviser or expert to the court. The overview thus is not complete. The aim of this paper is to analyse the cause of the failure costs to prevent recurrence, not so much the costs themselves. The project features, the characterization of the damage, the construction aspect that played a role, and a brief indication of the cause of the failure costs are given.

Description of failure cases

Previously, Van Tol (2001) and Van Tol (2007) analysed failure costs related to deep excavations in the Netherlands, ranging from relatively small damage to the work itself to very severe impact on the structure or on the adjacent buildings. In this paper, the main conclusions of the analyses regarding the type of failures and the causes of failures are described and an update is made for the most recent 5 year period (2007-2012). The authors focus on the possible role of risk assessment to counteract the ongoing stream of failing deep excavations and touch upon the role of education of geotechnical professionals.

Table 1 lists fifty claims in construction sites with some brief descriptions. The first column indicates the project number, the second column the characterization of the damage, the third column the construction aspect that played a role, and the fourth a brief indication of the cause of the failure costs is given. Claims described in this table include all events that occurred during construction, which were not foreseen and generally caused significant cost increases. This does not always mean actual damage such as settlement or cracks occurred in adjacent buildings. All necessary additional measures to prevent damage that had not been foreseen at the start of the project have also been included in the definition of failure costs in this paper.

The failures are either related to the retaining wall, the bottom seal of the excavation, the foundation or the surroundings. For example, the foundation may have insufficient capacity or intolerable displacements. If buildings or infrastructure surrounding the project are mentioned in the main aspect "surroundings", the cause of the failure is related to the state or perceived influence from the deep excavation. This does not mean other aspects did not influence adjacent buildings or infrastructure. Sometimes a combination of aspects is mentioned, but only the most significant causes are listed.

Table 1. Deep excavation projects with description of damage and possible measure

#	Type damage	Aspect	Primary cause	Predictability	Possible measure
1	Unforeseen injection and additional retaining walls	Bottom seal	Locally absent bottom clay layer; site investigation insufficient	Knowledge present	Risk analysis
2	Change of pile type during construction	Surroundings	Damage (feared) due to densification by vibrations	Qualitatively present, not quantitative	Improve site investigation (SI)
3	Additional jet grouting to seal wall	Retaining wall	Press-in sheet pile wall with interlock openings	Knowledge not present	Monitoring, improve methods to check locks
4	Failure tension piles	Foundation	Capacity vibrated piles insufficient	Knowledge present	Education, design review
5	Damage to buildings	Retaining wall	Presence of loosely packed sand at installation of secant pile wall	Qualitatively present, not quantitative	Additional research in topic
6	Settlement of utilities and street level	Retaining wall	Bad connection in corner of the retaining wall	Knowledge present	Education, design review
7	Jet grout retaining wall failure	Retaining wall	One row of columns proved insufficiently waterproof	Knowledge present	Risk analysis
8	Jet grout arch failure	Bottom seal	Insufficient cover on arch, uplift	Knowledge present	Risk analysis
9	Well leads to erosion and failure retaining wall	Bottom seal	Chemical injection eroded and clogged dewatering system	Knowledge not present	Additional research in topic
10	Jet grout block fails, inflow of sand and water	Bottom seal	Jet grout block not waterproof	Knowledge present	Risk analysis
11	Leakage Jet grout wall	Retaining wall	Jet grout wall not waterproof	Knowledge present	Risk analysis
12	Damage buildings	Surroundings	Combination: type foundation not known, retaining wall too flexible	Knowledge present	Improve site investigation
13	Jet grout wall fails, inflow of sand and water	Retaining wall	Jet grout wall not waterproof	Knowledge present	Risk analysis
14	Failure historic building	Surroundings	Type foundation not investigated	Knowledge present	Improve site investigation

#	Type damage	Aspect	Primary cause	Predictability	Possible measure
15	Failure free standing retaining wall	Retaining wall	Retaining wall too short, soil parameters too optimistic	Knowledge present	Risk analysis
16	Piles cracked due to bottom heave	Foundation	Reinforcement in piles insufficient (too short)	Knowledge present	Education, design review
17	Damage surroundings	Surroundings	Combination: quality foundation not known	Qualitatively present, not quantitative	Improve site investigation
18	Piled wall cracked, inflow of sand and water	Retaining wall	Leakage caused erosion, failure of adjacent foundations	Knowledge present	Risk analysis
19	Additional seal for retaining wall	Retaining wall	Press-in sheet pile wall disconnected, openings	Knowledge not present	Monitoring, better methods to check locks
20	Displacement adjacent building	Surroundings	Combination	Knowledge present	Improve site investigation
21	Late design change from polder to closed floor	Retaining wall	Leakage retaining wall through clutches and openings	Qualitatively present, not quantitative	Better methods to check locks, improve water tightness clutches
22	Leakage diaphragm wall	Retaining wall	differential displacement corner panels	Knowledge present	
23	Displacement adjacent building with shallow foundation	Retaining wall	Retaining wall not sufficiently stiff, installation effects anchors	Qualitatively present, not quantitative	Improve project processes
24	Displacement adjacent building	Retaining wall	Retaining wall not sufficiently stiff	Knowledge present	Risk analysis
25	Hardening of bentonite-cement wall	Retaining wall	Slow and fast hardening of mixture due to ground water pollution	Knowledge not present	Analysis of mixture with ground water from the site
26	Additional anchor to limit horizontal displacements	Retaining wall	Vibratory installation of sheet piles	Qualitatively present, not quantitative	Additional research in topic
27	Leakage Retaining wall	Retaining wall	Retaining wall to flexible, openings in press-in sheet pile wall	Knowledge present, knowledge not present	Risk analysis, Monitoring, better methods to check locks

#	Type damage	Aspect	Primary cause	Predictability	Possible measure
28	Settlement due to vibratory installation	Retaining wall/ Foundation	Densification and settlement during installation	Knowledge not present	Additional research in topic
29	Ground water lowering in surrounding area	Retaining wall	Leakage Retaining wall through clutches	Qualitatively present, not quantitative	Better methods to check locks, improve water tightness clutches
30	Settlement due to vibratory installation piles	Foundation	Densification and settlement during installation	Knowledge not present	Additional research in topic
31	Additional recharge wells	Retaining wall	Leakage Retaining wall through clutches	Qualitatively present, not quantitative	improve water tightness clutches
32	Additional tension piles	Foundation	Capacity tension anchors insufficient	Knowledge present	disseminate existing knowledge
33	Replacement installed foundation due to excavation damage	Foundation	Large soil deformation unsupported excavation in very soft soils	Knowledge present	Education, design review
34	Additional secondary piled wall	Retaining wall	Leakage piled wall	Knowledge present	Risk analysis
35	Uplift bottom, No vertical equilibrium	Bottom seal	Locally deeper excavation	Knowledge present	improve project processes
36	Additional permanent wall	Retaining wall	Leakage piled wall	Knowledge present	Education, design review
37	Extreme water discharge	Bottom seal	Unknown presence of drainage pipes	Knowledge present	Improve site investigation
38	Damage adjacent building	Retaining wall	Vibratory installation sheet pile wall	Knowledge present	Risk analysis
39	Failure adjacent building	Surroundings	Type of foundation not investigated	Knowledge present	Improve site investigation
40	Failure Retaining wall	Retaining wall	inferior designed sheet pile wall	Knowledge present	Education, design review
41	Displacement retaining wall	Retaining wall	Insufficient support, design too complicated	Knowledge present	Education, design review
42	Severe damage to adjacent buildings	Retaining wall	inflow of sand and water through defects in diaphragm wall	Knowledge not present	Additional research in topic

#	Type damage	Aspect	Primary cause	Predictability	Possible measure
43	Delays due to displacements of adjacent buildings	Foundation	Installation effect piles within excavation	Knowledge present	Education, design review
44	Accident with loss of life due to liquefaction behind wall	Retaining wall	Interlock openings in sheet pile wall and subsequent inflow of soil and water	Knowledge present	Risk analysis
45	Considerable delays due to deformation of existing railway	Foundation	Compaction of loose sands during vibratory micro pile installation	Qualitatively present, not quantitative	Additional research in topic, monitoring
46	Large surface settlements due to leakage of D-wall	Retaining wall	Inflow of sand and water through defects in diaphragm wall	Qualitatively present, not quantitative	Additional research in topic
47	Severe delays after installation of secant pile wall	Retaining wall	imminent damage to monumental museum	Knowledge present	Design review, education
48	Delays and additional costs related to piling	Foundation	Densification during pile driving and loss of casings	Qualitatively present, not quantitative	Design review
49	Settlements of high-rise building	Retaining wall	Settlements during vibratory installation sheet piles close to and under existing foundations	Knowledge present	Design review
50	Additional struts and change of pile system	Retaining wall	Large displacements compartment wall due to lacking support	Knowledge present	Design review

The conclusion drawn from table 1 is that in over 60% of the fifty cases there is something wrong with the retaining wall of the deep excavation, in 12% of the cases the bottom seal is unfit and in 10% of the cases damage occurred due to unexpected behaviour of adjacent buildings. In 17% of the cases, damage was related to the new structure's foundation. These aspects are discussed in more detail in the next section.

In 34% of the cases some form of damage was actually caused to adjacent buildings, the other main aspects of failure include time delays, requirement for additional measures to prevent deformations and requirement for additional measures to prevent leakages. Figure 1 shows the full composition of the cases analysed.

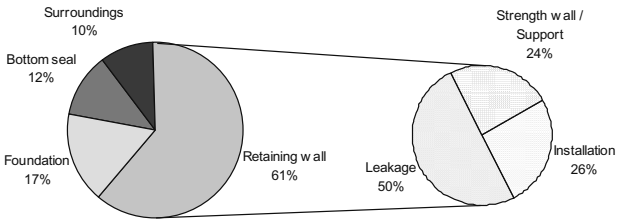


Figure 1. Decomposition of deep excavation failures

Deep excavation works relating to failures

In 61% of the cases discussed the wall of the deep excavation caused damage. This is a remarkably high percentage. In a quarter of these cases, the installation of the wall played a major role and in another quarter excessive deformation occurred due to problems with the stiffness (mostly cantilever types) or strength of the wall. The function of the deep excavation wall is to retain ground and (ground) water, its stiffness and strength must keep the wall deformation within acceptable limits. In half of the cases the water retaining function was compromised, so that leakage from a steel sheet pile wall, secant pile wall, diaphragm wall or jet grouted wall took place. Damage to the adjacent buildings is mainly caused by inadequate performance of the wall (including the anchoring system) or by damage during the installation of the wall.

Sometimes the deformations caused by the excavation were not larger than foreseen, but an incorrect assumption was made about the type, dimensions or quality of the foundation of the adjacent buildings. This was the major cause of failure in 10% of the total number of projects.

Deep excavations are usually sealed by natural and artificial layers below the excavation level. In 12% of the claims, this seal was the main source of the failure costs. Sometimes the naturally available sealing layer in the soil was absent locally, which had not been identified before construction due to insufficient ground investigations. In urban areas, it is not always possible to perform ground investigations as buildings to be relocated in a later stage are still present. In such a case, it is known in advance that there is an increased risk that the soil profile might deviate locally. In several cases, the excavation was deeper than designed for, usually for local elevator pits. Sometimes even the vertical equilibrium, including all the 3D effects was not sufficient to prevent uplift. Artificial seals such as injection layers are commonly known to be sensitive to construction practice and might not be water proof, but merely water resistant to a certain degree. If this is taken into account in the design, this need not be a problem.

Foundation piles in or adjacent to deep excavations may be damaged during the excavation. This can be caused by horizontal deformation, especially in very soft soil or by swelling. Damage due to the latter can generally be prevented by adequate pile reinforcement. Other failures are related to anchored concrete floors. Many floors are anchored with micro piles, which are very sensitive to installation procedures. Relatively few accidents have

occurred, mainly due to the increasing awareness of the sensitivity and the subsequent implementation of test programs and pile tests.

Design versus workmanship

In cases like these, an important question usually is whether the problem is caused by the design or the execution of the project. Although not mentioned for each case, in most of the cases from Table 1, the design played a crucial role in the origin of the failures. The designer of the wall (which can either be an engineering firm or a contractor in an integrated contract) often disregards the installation effects of retaining walls and foundations. A lot of effort is put into the design of the wall stiffness and related assessment of possible damage to properties, but the installation and the associated deformations are often excluded. The reluctance of designers to take issues of constructability into account is often due to perceived liability issues. The authors would like to emphasize that this aspect is likely to become more important for designers and construction parties alike, as restrictions on vibrations are presented (and observed!) more and more. Several developments have taken place to improve and validate prediction models for settlements due to installation effects, such as Hergarden and Van Tol (2001) and Meijers (2007). Also in cohesive soil, especially for sloped terrain, settlements may occur. In about 10% of the cases, poor workmanship caused the failure, usually related to the installation of diaphragm wall, piled walls or anchors, which is a specialist job.

Predictability of failures

The claims in this article are categorized by size and the extent to which the event was predictable or not. It is stated here that predictable adverse events do occur, due to insufficient knowledge of the designers. Despite the deployment of appropriate personnel, risk assessments and design reviews, such cases occur. Safety and robustness are important in design and efforts should be made to improve predictability through knowledge and model development. This paper shows whether the required knowledge was sufficient to, in principle, predict the damage or not. Also potential measures are indicated.

Bea (2006) made a distinction between predictable and unpredictable events when analysing hundreds of construction projects in the U.S., where major damage occurred in the field of geotechnical engineering. He concluded that in many cases the errors that arose directly related to (lack of) knowledge and development. In his work, he speaks of "unknown knowables" and "unknown unknowables". Regarding the first category, he argues that the knowledge (information) exists but is ignored, not used, was not known or incorrectly applied. Bazerman and Watkins (2004) call this category 'predictable surprises'.

The fifth column of Table 1 shows whether the required knowledge was sufficient to, in principle, predict the damage or not. In the last column, potential measures are indicated: if the knowledge exists but is not applied, the main measure should be training of personnel and / or design reviews. If the event was not predictable, specific improvement actions are described. Based on the work mentioned by Bea (2006) the cases from this paper have been categorized into "Unknown Knowables" (UK) or "Unknown Unknowables" (UU) as well, as shown in Figure 2.

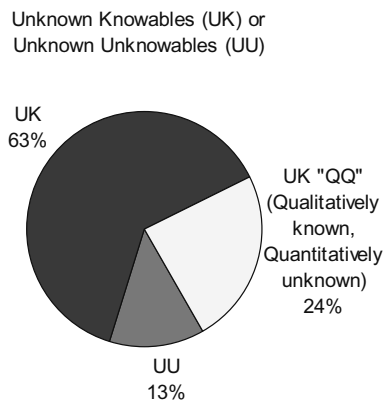


Figure 2. Subdivision in known and unknown knowledge

In more than 60 % of the cases the failure was due to not (correctly) applying existing knowledge. This is amazing but confirms Bea's findings. Moreover in 24% of the cases the cause of the failure is qualitatively known, meaning that these kinds of failures are known, but it cannot be predicted exactly when and where they will occur. An example of this category is: it is known that the interlocking of sheet pile walls may fail during installation (declutching). This is known, but it is hard to predict where and when this will occur. But using clutch indicators it can be monitored and corrected during installation. In only 13% of cases was unknown knowledge the main reason, all the others were "predictable surprises" as described by Bazerman and Watkins (2004).

The findings above are important because they can subsequently be used to answer the question how to improve the performance of geotechnical engineering. In figure 3 another subdivision is shown explaining that 27% of the failure cases could have been avoided with a proper training and education of the geotechnical staff and another 27% by proper (or improved) risk management. In 13% of the cases lack of information (e.g. the length of piles, the depth of a shallow foundation, etc) regarding the adjacent foundations was the main reason for failure. Often such information is extremely hard to obtain, in particular in the design stage of a project. The related risk of failure due to the lack of such data can however be managed by carefully analyzing all possible consequences. This may be achieved by identifying possible events, assessing the importance of the event and defining the necessary countermeasures (and related costs) that eventually should be taken. The same applies for the lack of ground information. In many cases, it is impossible to do proper soil investigation because existing buildings are still present on the jobsite. Then just at the start of the project the existing buildings are demolished and CPT's and borings are performed often leading to surprises and additional costs. In addition, these surprises can be avoided with a proper risk assessment during the design process of the project.

Therefore, it can be concluded that in 77% of the cases failure could have been avoided if proper risk management had been applied during the preparation of the project. Risk management also may have identified the lack of knowledge of the staff working on the project. Therefore the challenge for the education of geotechnical engineering is to not only teach a good theoretical base but also to convince the students to apply risk management during the entire design process. Especially the young geotechnical engineers without

experience should be familiar with a risk-based approach for the field of geotechnical engineering in which practice and experience play a major role.

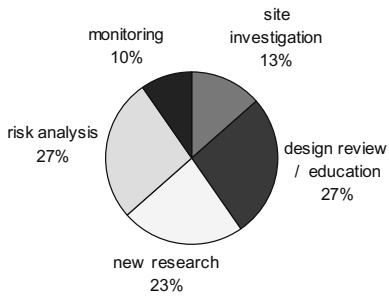


Figure 3. Focus of possible measures to improve performance

Trends

Although the number of cases in the period 2007-2012 is not significantly large, the authors feel like the following conclusions may be drawn based upon their experience. In the last 5 years, a clearly more significant interest has been put into preventing damage to surroundings. In the Netherlands, several research initiatives have started to quantify and predict the influence to surrounding structures. For example, the GeoImpuls program which aims to halve failure costs in geotechnical engineering by 2015 has started. This program introduces geotechnical risk management (GeoRM) as a standard such as described by Van Staveren (2006). The program also includes a research project into the quality of diaphragm walls and other cast-in-place elements. Promising results from this project are shown by Spruit (2011). Furthermore guidelines for monitoring deep excavations (CUR, 2010) and design of deep excavations with a strong focus on preventing damage (Korff and Roggeveld, 2012), as well as a guideline for micro piles (CUR, 2011) have been published recently. In the authors experience it seems that the average quality of the knowledge available for the design and construction of deep excavations has increased. Nevertheless, some very serious incidents have happened recently (e.g. project numbers 42 and 44 in Table 1), calling for further intensification on the topic. Projects seem to be becoming ever more complex, which means that failures generally will increase in severity. Risks of low frequency but high consequence are more difficult to handle in the process of geotechnical risk management due to the psychology of dealing with unlikely events. Geotechnical engineers may learn to deal with those risks by observing other professions with more experience in low frequency – high consequence risks, such as for example in the aviation industry.

Conclusions

Based on 50 cases of deep excavations in The Netherlands, it appears that in a large majority (60%) of claims the knowledge to prevent problems existed beforehand, but for various reasons was not used. The responsibility of design should be assigned to the right people and general training of staff must be the highest priority, if we want to reduce failure costs for deep excavations. Conducting design reviews can also be a means to prevent such damage. If the phenomena that play a role are identified, a risk analysis may help in making the right decisions.

The present gaps in knowledge are generally not related to completely unknown phenomena, but more likely to well-known phenomena that are hard to predict. Many of these types of

claims (more than 60%) relate to the installation (e.g. vibratory sheet piling) and or performance (e.g. leakage through secant pile or diaphragm walls) of the earth retaining structures. In about 10% of the analysed cases, the problems arose from misunderstanding the behaviour of the adjacent buildings. Research is especially needed into methods to prevent and detect leakages.

Since 2007, it seems that the awareness with regard to limiting damage to adjacent buildings and failure related to deep excavations has increased. Not enough projects in the period 2007-2012 can however be evaluated to obtain general conclusions on a possible increase of project quality since 2007. At the moment, the authors recommend to study other low frequency – high risk industries to learn how to deal with those risks specifically.

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Forensic Engineering: Informing the Future with Lessons from the Past
ISBN 978-0-7277-5822-4

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doi: 10.1680/feng.58224.225

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IMPACT OF ADJACENT CONSTRUCTION ON EXISTING PILE FOUNDATIONS

Yang, Hanwei Taiwan High Speed Rail Corporation, Taipei, Taiwan
Ho, Shu-Ken Sino Geotechnology, Inc, Taipei, Taiwan

Introduction

Ground settlement or lateral soil movements due to construction activities are well understood. Ground movement associated with nearby construction activities may have adverse impacts on existing piles, including geotechnical or structural pile failure due to movement of the soil in relation to, as well as to additional shear or bending moments acting on, the pile structure. O'Neil (1983) showed that pile-soil-pile interactions consist of "installation" effects and "mechanical" effects. Installation effects, which refer to the alteration of soil stress states, densities, and grain size distributions due to pile installation, can be different for groups of piles and single pile. Mechanical effects refer to the alteration of soil strains and failure zones due to simultaneous loading of closely spaced piles. The mutual interaction of installation and mechanical effects makes rigorous analysis difficult. Huang (2001) summarized data from field tests including the standard penetration test (SPT), cone penetration test (CPT), and dilatometer test (DMT) on a full-scale lateral pile load test project in Taiwan to evaluate the effects of construction on the behavior of laterally loaded pile groups. This study showed that pile group installation, bored or driven, tended to disrupt the material near the ground surface and lower the in situ lateral stress. Bored pile group construction appeared to loosen the soil surrounding the piles, whereas driven pile construction apparently caused densification.

The Taiwan High Speed Rail (HSR) Project is a key component of transportation development in Taiwan that involves the construction of an approximately 345-km-long railway between terminal stations at Taipei in the north and Kaohsiung in the south. The southern section from TK193 to TK345 is constructed on an alluvial plain and nearly all alignments are essentially on viaducts and bridges. Typical foundation conditions are deep alluvial or colluvial materials, often overlaid by soft clay of varying thickness that require piles to provide sufficient capacity and minimize the possible settlement.

To provide access between HSR stations and nearby urban areas, highway authorities constructed three sections of highway parallel to the HSR alignments. The nearest distance

between the edges of the HSR viaduct structure and highway structure is 5 m. In general, the structural type of a highway is an embankment 2–3 m high. However, viaducts with a ramp structure of up to 6 m high were adopted for sections crossing over a river. For these reasons, many nearby construction activities such as piling, earthwork, and excavation of pile caps from the highway construction project were undertaken between 2004 and 2005. The effects of nearby construction activities on existing HSR pile structures have been monitored and documented (Sino Geotechnology Inc., 2006, 2007). In this paper, we first present two cases of pier settlement, namely of Piers P627 and P1224, caused by adjacent construction activities. Subsequently, we investigate the underlying causes by monitoring data.

Observed pier settlement: P627

On 10 June 2004, a visual inspection of the track alignment around pier P627 north of the Yuanshuixi River area was undertaken following a verbal report of misalignment of the tracks in this immediate area. The inspection confirmed the presence of displacements for both north- and south-bound tracks. Immediate steps were taken to carry out a detailed survey of trackwork bollards. The results indicated that a vertical settlement of around 55 mm had taken place at some time between April and June of 2004. The horizontal alignment was also displaced by around 50 mm.

The affected HSR structure is a standard 30-m concrete box girder viaduct simply supported on four bearings on top of the pier head. The pile cap is 11 m × 11 m × 2.8 m and supported on four 2-m-φ piles 58 m long. The piles are designed as friction piles; they were built entirely within the alluvial layers and therefore not founded in rock. This is normal for all piles within the southern section of the railway alignment.

We noticed that construction work was underway for a new highway that straddles the existing HSR viaduct alignment and includes a 385-m-long bridge across the Yuanshuixi River. This bridge is composed of twenty-six 25- to 35-m-span box beams and supported by 26 piers and abutments. Within the Yuanshuixi River area, the highway contractor was constructing large bridge foundations. At the time of discovery of the track misalignment, pile construction work was underway adjacent to pier P627. Figure 1 show the layout of structure for HSR and adjacent highway. The highway contractor has reportedly been encountering problems with the piling work at this location.

Excessive loading, insufficient pile capacity, and impact from adjacent highway construction are three potential causes of the unexpected settlement of HSR piers. The loading from the superstructure and available pile capacity were thoroughly assessed by considering the site conditions and construction activities. The current loading was only about 16% of the pile

capacity. Hence, insufficient design capacity is excluded as the likely cause of the observed settlements. At the time the settlements of the piers were recorded, construction work on the foundation for the adjacent highway was underway; in addition, these piers are rather close to those of the HSR infrastructure. Therefore, the possibility that these two events were connected is reasonable.

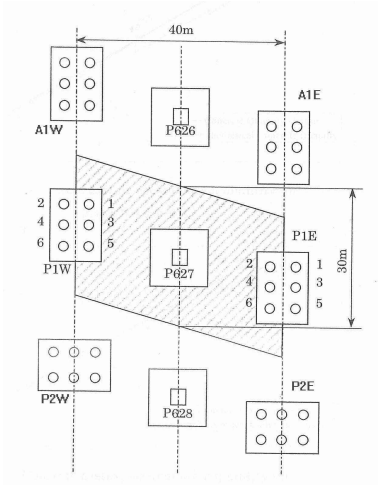


Figure 1. Structural layout

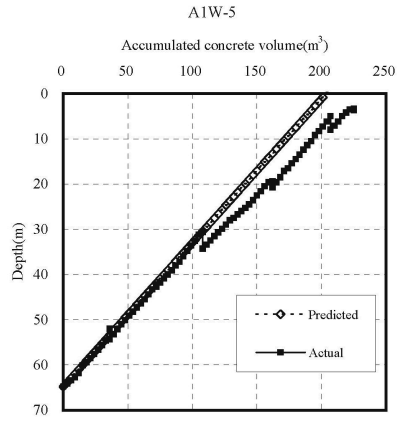


Figure 2. Concrete consumption record

Experience shows that during construction of the cast-in-place piles, the possibility of over-excavation and/or cave-in or lateral sand flow in the cohesive soil layer can be identified by monitoring the quantity of concrete used in the filling stage. Monitoring records indicated that during construction of the foundation for the four piers of the adjacent highway (A1W, P1E, P1W, and P2E), noticeably excessive amounts of concrete were pumped. Figure 1 shows the structural layout of the adjacent highway. Most of the over-use occurred between the depths of 35 to 50 m, where a cohesive soil layer was present. Figure 2 gives the typical curve of concrete consumption along the length of the pile for pier A1W. The four above-mentioned piers under construction are located near P626–P629, of which P627 is positioned in the middle. The total amount of over-used concrete was around 450 m^3 for these four piers. Therefore, estimating that the total volume of over-excavation, cave-in, and lateral sand flow will be equal or somewhat greater than 450 m^3 is reasonable. We note that the period of construction for these four piers was between April and June of 2004. The effects of these over-excavations on the site soils are difficult to assess. The presence of the thick cohesive soil layer located above the 35- to 50-m sand layer would most likely mask the effects on short-term bases. This is probably why the supplementary site investigation borings located near the pier of the HSR did not reveal any significant disturbances in the soil layers. If the

construction resulted in disturbances in the sand layer and caused loosening of the sand soils, the overburdening pressure acting on the sands might even re-compress the sand so that subsequent N-values of SPT in situ would not detect significant density-associated changes. Nevertheless, the effects of each over-excavation were greatest at the pier location and could be accumulative in nature; hence, the combined effects would be greatest and most obvious at or near the center of these four newly constructed piers. Given that the disturbance, re-compaction of the sand layer, and subsequent downward movement of the cohesive soil above the sand layer would be progressive, they may not be noticeable immediately after the over-excavations. This could have caused the track distortion initially observed on June 2, 2004.

The construction of pier P1E-1 started on June 3, 2004. Major difficulties were encountered during construction of this pier. An incident that involved upward movement of the steel cage (reinforcements) suggested that serious lateral sand flow had occurred. Reinsertion of the casing was stopped near the depth of 41.1 m, for 40 hours; this may have resulted in additional over-excavation and/or cave-in of the sand soils.

Construction records also indicated that on June 10, 2004, an impact hammer was used to free the casing. These impact loadings may increase pore water pressure in sandy soil and cause additional disturbances and subsequent re-compaction of the sands. Figure 3 is a photo that shows significant cracks on the ground after this incident. This indicates a serious disturbance of the surrounding soil.

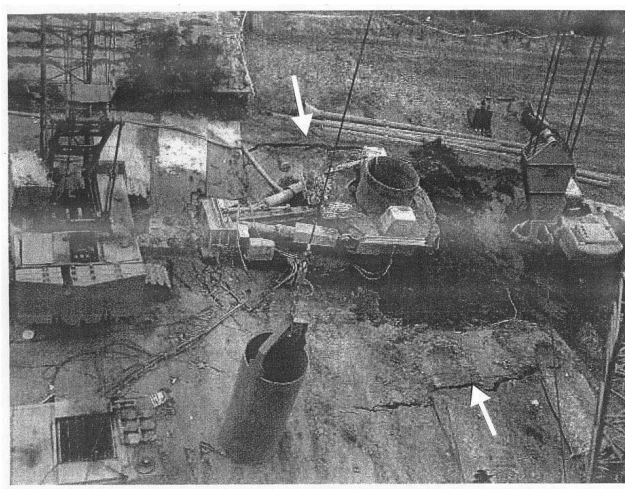


Figure 3. Ground cracks after incident

Quantitatively, when subsidence of the upper soil layer causes negative skin friction acting on

the pile, the safety factor for the pile capacity is reduced. This factor becomes one when the sum of the dead load and the magnitude of negative skin friction are equal to the residual friction resistance and tip resistance combined. A safety factor of one usually results in large vertical displacements. According to the original design parameters, the dead load on each pile of the four-pile group is 438 tons (i.e., ¼ of 1750 tons). The amount of negative skin friction that could develop from subsidence of the upper 41 m is about 1272 tons for each pile. The residual positive skin friction from the soil below 41 m would be 969 tons. According to the above discussion, at least 840 tons of tip resistance (78% of the original design tip resistance of 1072 tons) is needed to maintain a safety factor of one for the pile. Experience with the construction of cast-in-place piles by using the reverse circulation suggests that there is mud at the base of the piles constructed at the site; however, based on the observed settlement, we may conclude that the actual tip resistance is 840 tons or less.

We note that the discussion above was based on examining available data and performing pile capacity analyses to evaluate the likelihood of each possible cause. Therefore, the conclusions made are qualitative in nature; any quantitative assessments presented are approximations engineering practice.

According to the latest survey data, no additional settlement has occurred since June of 2005; some minute upheavals have been recorded, although this is within the acceptable range of survey tolerance. This indicates that the piles have reached an equilibrium state and that the reduction of pile capacity due to negative skin friction should decrease with time and lead to full recovery of the friction pile capacity.

Observed pier settlement: P1224

On Dec. 2, 2004, twisted rails were reported in the vicinity of pier P1224 in the Chiayi area. The suspected cause of rail movement was settlement of the piers, which in turn may have been caused by the adjacent highway construction work underneath the viaduct. There were excavations adjacent to the HSR piers in order to construct pile caps and a foundation for a retaining wall. The soil from excavation was piled up along the site up to 5 m high in some areas. The settlement profile in the vicinity of pier P1224 since June of 2003 indicated that settlement (a few centimetres) had occurred between May and December of 2004. The records between June of 2003 and May of 2004 did not reveal any significant settlement of piers. Consideration of the site conditions suggests that the settlement of piers supported by piles is initiated by uncontrollable soil movement due to the placement of fills, excavations, or pile installations by the construction of a nearby highway. However, further evidence is

required to support this hypothesis.

Field monitoring program and data interpretation

In January of 2005, the foundations for the elevated section of the highway project were completed. The remaining ramp structures located on the north and south sides of two waterways, namely the Xinpi Channel and the Puzhi Creek, were expected to cause further ground settlement that was likely to make a significant impact on the existing HSR infrastructure. For this reason, a comprehensive monitoring program was set up prior to commencing construction of the remaining ramp structures.

The monitoring program consists of three parts. The first includes four extensometer-monitoring wells and two inclinometers installed in the area between the highway ramp structures and the HSR infrastructure. The second involves the survey monuments on the retaining structure for the ramp highway. The third is comprised of the survey monuments built on the pier of the HSR viaduct.

The construction activities for the ramp structure include earthwork and retaining wall seating on piles. The height of fill for a ramp structure varies from 2 to 6 m. Two rows of piles with a diameter of 1 m and a length of 24–32 m were constructed to support the retaining wall. Construction began in January of 2005 and was completed in June of 2005.

Data from a leveling survey for HSR pier elevation showed HSR piers in four areas that underwent significant settlement since December of 2004. These four areas north of the Xinpi Channel and south of the Puzhi Creek are located coincidently near the ramp structure for the highway. Figure 4 shows the pier settlement time history of the four most affected piers and nearby construction activities from 12/19/2004 to 08/11/2006. During this time, two stages of significant settlement occurred during the periods from mid-March of 2005 to mid-June of 2005 and from mid-December of 2005 to mid-April of 2006. A similar trend was noticed from the data of piers located south of the Puzhi Creek. Pile installation for retaining wall and fill work for ramp embankment were the only construction activities during the same time frame.

The data from the extensometers in Figure 5 show similar displacement behaviors during the same period. Data from SIS4 (Figure 6) indicate that a distinct lateral displacement of soil at depths ranging from 20 to 30 m below the ground surface was recorded during the pile

installation. The data from SIS4 and EXT4 both indicate that major soil deformation occurred at soil layers 20–35 m below the ground surface during pile installation. This clearly demonstrates a mechanism of pile settlement due to soil deformation adjacent to the pile structure. In addition, data from EXT4 (Figure 7) indicate 30 mm of accumulated settlement due to the 6-m-high fill.

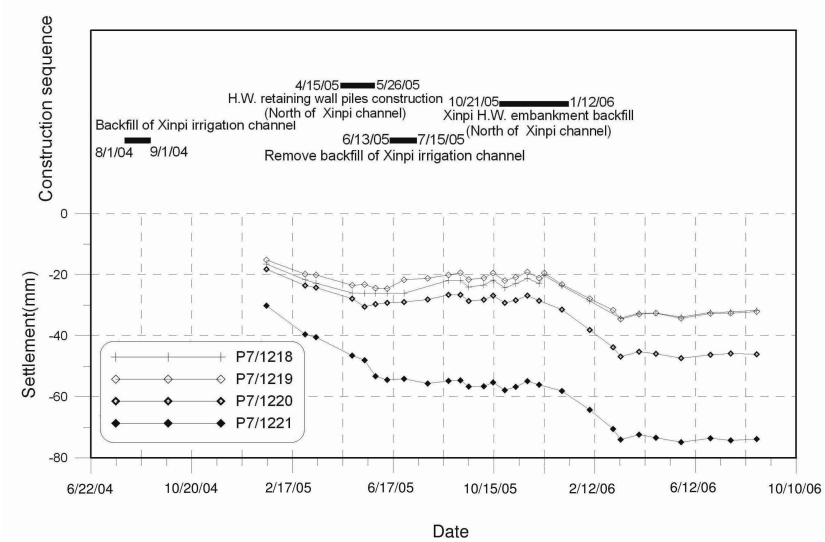


Figure 4. Settlement histories for four affected piers on north side of Xinpi Channel

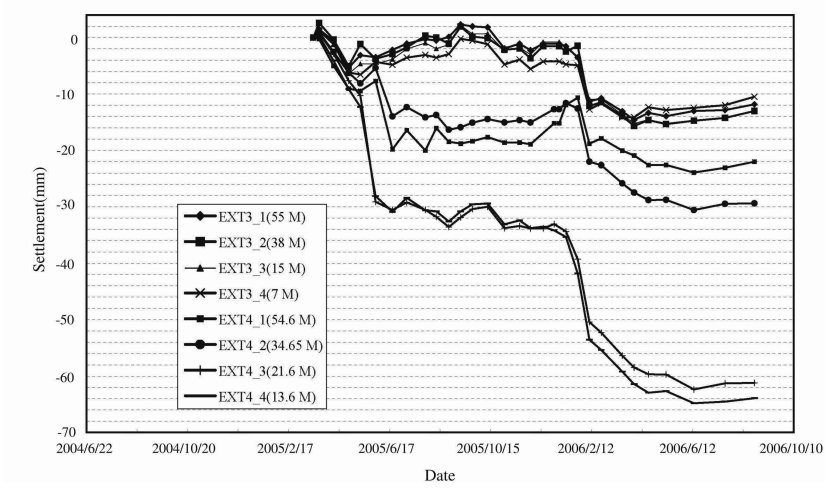


Figure 5. Data from extensometers: north side of Xinpi channel

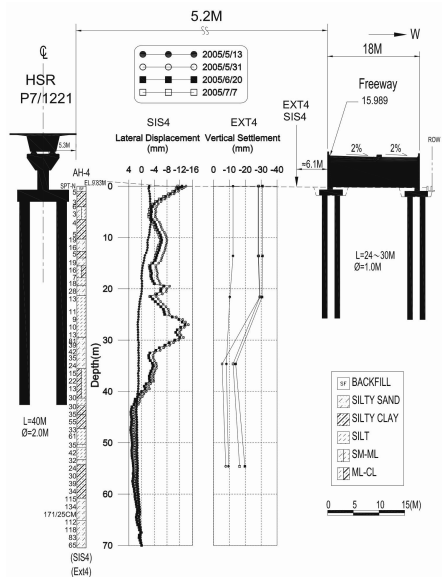


Figure 6. Lateral soil displacement observed by SIS 4 and EXT 4 during pile installation

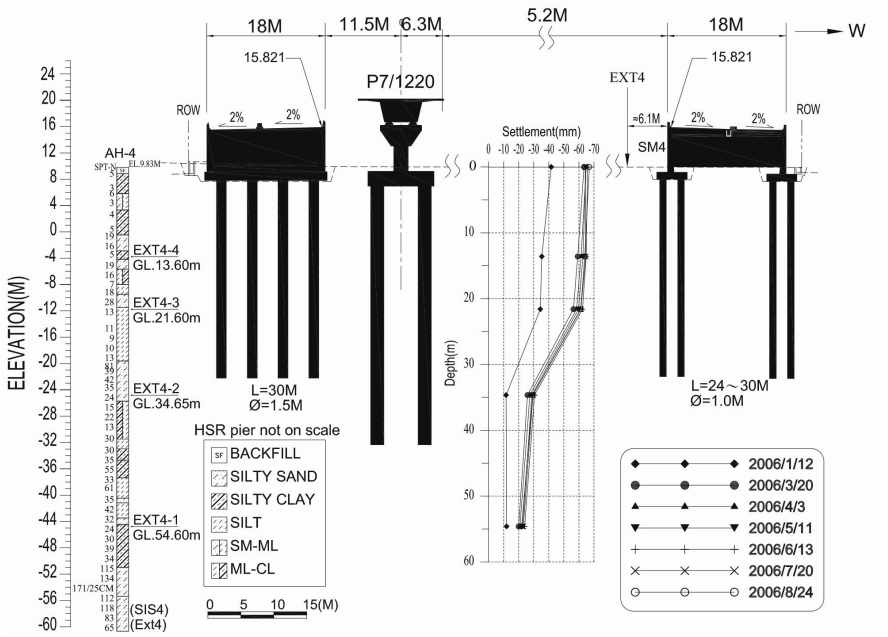


Figure 7. Soil deformation observed by EXT 4 during fill work for ramp embankment

Table 1. Summary of pile capacity re-evaluation

HSR Pier	Pile Top (EL.m)	Pile Bottom (EL.m)	Pile Length (m)	Single Pile Design Capacity (kN)		Disturbed Soil Layer Depth (m)	Single Pile Capacity (kN) (Negative Friction Condition)		
								CASE-1	CASE-2
P7/1224 (8 Piles)	1.0	-50.5	51.5			34~37	Pfn=	9888	9888
				Qs=	25582		Qps=	13392	13392
				Qb=	8492		Qb=	8492	0
				Qult=Qs+Qb=	34073		Qult=Qps+Qb-Pfn=	11996	3504
P7/1225 (4 Piles)	3.8	-50.0	53.8			31~36	Pfn=	9083	9083
				Qs=	26030		Qps=	14854	14854
				Qb=	8492		Qb=	8492	0
				Qult=Qs+Qb=	34522		Qult=Qps+Qb-Pfn=	14263	5771

Re-evaluation of pile capacity

Piles for two piers, P1224 and P1225, suffered significant settlement due to adjacent construction were selected in this paper for re-evaluation of pile capacity. Referring to the unit skin friction mobilization curves from the pile load test conducted in this area, we find that the displacement required to mobilize ultimate pile skin resistance is 20–25 mm. The observed accumulated ground settlement is within the range of 2–4 cm which is sufficient to mobilize ultimate pile unit skin friction. That negative skin friction can be generated on piles and result in a large reduction of pile capacity and pier settlement may be reasonably expected. Table 1 gives the dimensions and design ultimate pile capacity for the piles of each pier.

It is assumed that the neutral point (zero relative movement between soil and pile shaft) is located at the disturbed soil layer. Pile negative skin friction (Pfn) could be mobilized by the soil layers along the pile structure above the neutral point. The ultimate pile capacity (Qult) with consideration of end bearing capacity (Qb) for Pier 1225 and 1224 are 11,996 kN and 14,263 kN respectively (CASE-1) which are less than the dead load from the superstructure combined with the live load from the train (17,150 kN). If no end bearing capacity can be mobilized (CASE-2), the deficient gap of pile capacity between demand and supply is even higher. The results of pile capacity re-evaluation from both cases indicate that the pile capacity may not be sufficient to resist the loadings from the current conditions. It can therefore reasonably conclude that the significant settlement of piles during adjacent construction was resulted from in-sufficient pile capacity due to pile negative skin friction acting on pile structure. Details of the assessment results of pile capacity are summarized in Table 1.

Conclusions

Here we reviewed two cases of pier settlement. We examined and assessed the possible causes, including excessive live/dead loads, insufficient pile capacity, and construction of an adjacent highway. We found that piles sitting on an alluvial plain were easily affected by nearby pile installation and fill work. The main adverse impact of nearby construction on existing piles is the densification of loose and sandy material after disturbances.

Monitoring data provide direct evidence of significant lateral soil movement and settlement caused by bored pile installations with casings. In addition, we observed the squeezing of loose material underneath a ramp structure built with fill. During the same period, the settlement of HSR infrastructure piers supported by piles occurred.

The lesson we learned from these two cases of pier settlement is that insufficient pile capacity due to generation of negative skin friction, which resulted from the relative movement between the soil and existing piles caused by adjacent construction activities such as piling and fill work, is particularly critical for construction sites seated on the alluvium. Zones that are free from the impact of piling/fill construction should be evaluated and defined prior to construction. If construction outside of these zones is unavoidable, engineering schemes to minimize the impact from adjacent construction should be adopted. Monitoring should be conducted jointly by the contractor tasked with new construction and the owner of existing structures in order to clarify the liabilities for unexpected settlement that occur on existing structures during construction.

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SECTION 5:
TEMPORARY STRUCTURES

Forensic Engineering: Informing the Future with Lessons from the Past
ISBN 978-0-7277-5822-4

ICE Publishing: All rights reserved
doi: 10.1680/feng.58224.237

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Bragg Revisited: Keeping UK's Temporary Works Safe in the Dawning Eurocode Era

Eur Ing **Bill Hewlett** MA CEng FICE, Costain Ltd, London, United Kingdom

Andrew Jones BEng CEng MICE, RMD Kwikform Ltd, United Kingdom

Eur Ing **Stuart Marchand** BA CEng FICE MIStructE, Wentworth House Partnership, London, United Kingdom

Brian Bell MA MSc DIC CEng FICE FIStructE, Bell Johnson, Royston, United Kingdom

Introduction

40 years ago the UK construction industry was enjoying a hey-day of prosperity. Motorway construction in particular was in full swing. It was a period also of considerable physical danger. In the 1960s and early 1970s the falsework to a number of bridges collapsed during construction and the government was moved to investigate whether the industry was fit to deliver the construction programme. In consequence, Professor Stephen Bragg was commissioned to investigate, and his final report (Health and Safety Executive, 1976) (the 'Bragg Report'), helped set the standard for Temporary Works design and management in the UK.

Although the Bragg Report is now the most widely remembered document from the period, other documents were also influential, in particular a report on falsework (The Concrete Society and the Institution of Structural Engineers, 1971).

The terms of reference for a new code of practice for Falsework (BS 5975) had been set out prior to the final publication of the Bragg Report. Notwithstanding this, Bragg's recommendations were, by and large, incorporated, together with the falsework report prepared by the joint committee. The impact of BS 5975 (BSI, 1982), influenced as it was by the Bragg Report, has been profound: since its adoption by the industry, there have been no major falsework or excavation failures in the UK to compare with the failures of the early 1960s and 1970s.

The hazard that Temporary Works[(a)] presents, however, remains with us. Major project failures occurring internationally remind us of the dangers, for example the Nicoll Highway Collapse in Singapore in 2004, and, more recently, the collapse during erection of the FC Twente stadium roof in Holland[(b)], the failure of a falsework supporting a bridge in Andorra[(c)]

[a] BS 5975 defines temporary works as "*parts of the works that allow or enable construction of, protect, support or provide access to, the permanent works and which might or might not remain in place at the completion of the works.*" In this paper, we also consider temporary works design to include consideration of the temporary stages through which the permanent works may pass. BS 5975 defines the subset, falsework, as "*temporary structure used to support a permanent structure while it is not self-supporting.*"
[b] *FC Twente stadium roof collapse – learning from the fatal consequences*, SCOSS Topic Paper, October 2012.
[c] *Bridge Falsework Collapses as Workers Pour Concrete*, Nov 2009, accessed Sept 2012: http://enr.construction.com/infrastructure/transportation/2009/1111-bridgefalsework.asp

and the collapse of an excavation in Cologne[(d)]. The UK has not been without its incidents but these have been relatively minor and fortunately without fatality.

The questions we must answer are how we continue, in the UK, to avoid a recurrence of the sort of failures that occurred in the period before Bragg, and how we can improve on the present situation. The present moment is well chosen to consider these questions. There are two catalysts for change which we need to manage.

The first catalyst is the potential for loss of 'collective' memory in the industry, as to the hazard of Temporary Works failure. This memory actually resides in individuals, not organisations, and our senior engineers in the UK now, who started their careers at the time of the pre-Bragg failures, are retiring.

Engineering managers remain influential in major UK infrastructure projects and their memory of what failure looks like guides their actions. For schemes like Crossrail and the London Olympics, the safety of Temporary Works is at the front of management attention, and the schemes are exemplary. As this generation retires, the level of regard for engineered safety amongst our most senior managers might be learnt, or perhaps it might not be; what we can say is that it will not have been absorbed as a part of the collective experience of the 1960s and 1970s.

The second catalyst is the arrival of the new EuroNorms and the subsequent withdrawal of conflicting national standards. The style of the EuroNorms is unfamiliar to those used to British Standards as they are less prescriptive, more numerically based and contain very little procedural advice.

It should be noted that unlike the majority of other standards BS 5975 has not been withdrawn and exists in parallel with the EuroNorm EN 12812 (BSI, 2008). This relationship is emphasised by the forewords of each standard referring to the other. The reasons for both standards existing in parallel are:

- EN 12812 introduces two classes of falsework, A and B, specifying limit state design methods for the latter. Class A is specified as small simple construction "which follows established good practice which may be deemed to satisfy the design requirements". As BS 5975 provides established good practice, it is necessary to retain it.
- Even if BS 5975's promotion of permissible stress for steel design (although not for concrete) is seen to be in conflict with EN 12812's of limit state, this is for Class B only: the permissible stress approach is 'established good practice' and at least relevant to Class A.
- EN 12812 does not provide any procedural control, an area which Bragg highlighted as being critical for the safe execution of Falsework and which forms a central part of BS 5975, emphasised by the consolidation of its procedural control in 2008.

Unless specified by the contract, for the design of falsework for Works in the UK, designers are currently free to choose (for steel) either a limit state approach as set out in EN 12812 or the permissible state approach in BS 5975. There is a danger that if only EN standards are specified in contracts the procedural control will inadvertently be left out. Regardless of the

[d] *Köln - speculation and anger in aftermath*, March 2009: accessed Sept 2012: http://www.tunneltalk.com/Cologne-collapse-Mar09-Deadly-collapse-in-Cologne.php

design approach adopted, the procedural controls given in BS 5975 should be used, as highlighted in its foreword.

Lessons from a past generation can inhibit us; on the other hand they can be the wisdom that sustains us. In planning a way forward we must accept that some old habits must be laid aside. Our contention, however, is that this should be done knowingly and in full recognition of the hazards and risks with which we are dealing. In the construction phase of projects, there are few greater hazards than a major Temporary Works failure. For this reason we consider it is well worth while to look back at the Bragg Report and consider what the new generation of engineering leaders must take forward from it in the Eurocode Era.

What did Bragg say?

The Bragg Report gathered an extensive volume of evidence to do with falsework, which it takes to be the structural support of wet concrete. Examples are drawn from the UK and overseas, and from civil engineering and building construction.

The nub of the report lies in the 27 Principal Recommendations which cover design, site management, regulation, research, published data, testing, training and education, certification of operatives, client role and the need for authoritative publications. The most celebrated are No. 17, which is the need to appoint a TWC[e] and No. 3, which requires the designer to consider a horizontal load of 1% of vertical plus calculated horizontal loads or 3% of vertical, whichever is greater (BS 5975 uses 2½% in place of 3%).

In many ways it is a pity that Bragg was asked to consider only falsework. In principle all that he concluded applies equally to all Temporary Works and the industry has in fact adopted this approach. It is perhaps telling that his Principal Recommendations call for a "Temporary Works Coordinator", not a Falsework Coordinator, so maybe this was in his mind. Although the procedures in the first revisions of BS 5975 were ostensibly for Falsework – with their use of "Falsework Coordinator" – they were adopted across the Temporary Works sector, a fact that was recognised in the 2008 revision when it was renamed "*Code of practice for Temporary Works procedures and the permissible stress design of falsework*". The standard was re-ordered to give more prominence to the procedural control and the role of Falsework Coordinator was renamed Temporary Works Coordinator.

The TWf (Temporary Works Forum, 2012) has recently reviewed Bragg's 27 Principal Recommendations and reconsidered them in the light of progress made since 1975, the industry as it is today and EN 12812. The TWf's primary recommendations are that the management practice of the Bragg Report, later adopted in BS 5975, is retained as the operational protocol, and that the significance of the serviceability limit state, in addition to the other limit states, to temporary works design is recognised.

A particular point of comparison between BS 5975 and EN 12812 is that while BS 5975 is a self-contained code sufficient for complete designs, EN 12812 considers only the above-ground falsework and cannot be used without reference to the Eurocode suite.

Based on the TWf's review, the table below has been developed. It has been ordered so as to group Bragg's recommendations into Design, Procedure, Research and Information, and Training. The third column 'Today's Perform.[ance]' is an assessment of the degree to which each Bragg recommendation, in the UK, has been implemented as we stand today (10 = fully

[e] TWC: Temporary Works Co-ordinator, as referred to in the foreword of BS 5975

implemented, 0 = not implemented at all). The fourth column 'Impact of ENs' is an assessment of the likely impact on today's performance should EN 12812 (with the Eurocode suite in support) be applied without further intervention and without the use of the procedures set out in BS 5975.

Bragg Ref	Paraphrase of Bragg's Recommendation	Today's Perform.	Impact of ENs	Comments
Design				
3	Lateral stability must be considered in all directions with a minimum considered horizontal load equal to the greater of; the calculated horizontal loads plus 1% of the vertical load, or 3% of the vertical load.	9	Slight reduction	Written into BS 5975 but with the second condition reduced to 2.5%. Has become the norm in TW design. EN 12812 derives bracing requirements from consideration of both strength and stiffness. At this stage, completed designs to EN 12812 appear similar to those derived from BS 5975.
4	Overall factor of safety for the falsework should not be less than two.	8	Similar	BS 5975 is based on BS 449 (BSI, 1969) and therefore has a general FOS of 1.7 on yield. It also suggests a FOS of 2 on failure for items proved by testing. It only requires a FOS of 1.2 on overturning. EN 12812 uses a partial material factor of 1.1 and load factor of 1.35 and 1.5 which will give a similar FOS to BS 5975. The FOS against overturning is about 1.65.
5	Design to allow for erection tolerances.	8	Similar	BS 5975 gives values of acceptable erection tolerance. Modern falsework systems tend to force accurate positioning and alignment. EN 12812 states that actual tolerances be taken into account.
7	Special attention to beam grillages and failure due to rolling over or web buckling.	9	Large reduction	Excellent advice written into BS 5975. While the Eurocode suite gives design standards, practical advice is absent.
12	Designer to consider the effect of second hand equipment on the design.	7	Similar	Allowed for in BS 5975 but only in respect of reduced section area and increased initial out of straightness. A greater proportion of hired equipment is now used which generally has a plated finish and good quality control on return from site.
Procedure				
1	Recording of any falsework collapse required by Statute.	8	No change	Addressed in RIDDOR, which makes all falsework collapses 'reportable'. The extent of actual reporting, especially of minor incidents, is not complete, neither are the reports readily available to the public.

Bragg Ref	Paraphrase of Bragg's Recommendation	Today's Perform.	Impact of ENs	Comments
2	Extend the regulations for periodic examination of access scaffold to falsework.	5	Risk of large reduction	Although not implemented explicitly, the management aspects of BS 5975 and the framework of health and safety legislation, especially the CDM Regulations and the Work at Height Regulations, provide an adequate framework of management and control for falsework. If the Permit to Load requirement in BS 5975 is overlooked, safety risk would increase dramatically.
6	All falsework must be designed. A proper written brief should be produced.	7	Similar	Virtually all medium/large schemes and hire equipment from main suppliers are designed to written briefs. Small schemes with builders' own equipment are still an issue. EN 12812 is explicit on requirements.
10	Suppliers of proprietary equipment to provide basis of testing as well as safe working loads.	5	Similar	There is no standard test regime for proprietary materials and no requirement for this recommendation to be followed, or for reporting. Good quality design information is published by some suppliers, who can also provide more detail if asked. However, this is not uniform. The EuroNorms are more explicit on test requirements, but still do not go far enough.
11	More vigilance of materials; testing on new and used to check properties.	5	Similar	A greater proportion of equipment is now supplier owned and they have better controls than contractors' plant yards. There is an increasing issue with 'copycat' systems and inferior components such as scaffold fittings being imported from overseas into the UK market.
13	Design to be approved by fully qualified engineer.	7	Reduction	BS 5975 requires that all designs are checked by a competent person and that there is a robust system for ensuring competence. It also gives guidance on different categories of required check. See also comment to recommendation 2. The EuroNorms do not require one party to coordinate all aspects of brief, design, design check and pre-load inspection.
14	Novel designs to be checked by independent assessor.	7	Reduction	Category of checking in BS 5975 covers this situation: see comment on recommendation 13. The EuroNorms do not mention novel designs.
15	Falsework design to be reviewed by permanent works designer.	2	Reduction	The effect of intermediate stages, and of the temporary works, on the permanent works should be checked by the permanent works designer: however, the view that permanent works designers are the most competent to carry out design checks on temporary works has changed. Procedures to ensure adequate checks are given in BS 5975. EuroNorms do not mention novel designs.
16	Rigorous design and check procedures.	7	Reduction	Procedures to ensure adequate checks are given in BS 5975 and are generally well supported across the industry.
17	Appointment of TWC.	9	Risk of large reduction	Written into BS 5975 and has become standard practice on medium/large projects. Not a requirement in EuroNorms.

Bragg Ref	Paraphrase of Bragg's Recommendation	Today's Perform.	Impact of ENs	Comments
18	Improve communication between designer and site.	7	Similar	Written into BS 5975, CDM has brought change of culture. EuroNorms reinforce best practice.
25	Construction organisations directly responsible for falsework must keep a register of certificated operators they employ on each site.	3	No change	Did not become a specific requirement of BS 5975. However industry has moved in this direction with increased use of certification of roles.
26	The Government should insist that all falsework used on public sector projects is carried out by companies with an adequate training system.	0	Similar	Did not happen. See also comment to 25.
Research and Information				
8	Field study of actual applied loads.	2	Similar	Apart from a few exceptions no work has been done and there isn't a consistent programme of investigation. It is possible the EuroNorms, as a European initiative, will promote co-ordination and an increase in research.
9	Research into falsework.	2	Similar	See comment to 8.
27	A falsework handbook and a falsework textbook should be produced.	7	Similar	A number of very good publications exist in the field of Temporary Works; BS 5975, 'The Formwork Guide To Good Practice' (The Concrete Society, 2012), 'TG20' (National Access and Scaffolding Confederation, 2008) and 'Temporary Works - Principles of Design and Construction' (Grant, M and Pallet, P. 2012) for example. The introduction of the EuroNorms may lead to an increase in publications addressing the wider European audience.
Training				
19	Include safe working procedures in all falsework courses.	6	Similar	Is included in most courses although the number of courses is small.
20	Falsework should form part of all Civil Engineering and Architectural courses.	1	Similar	Although there are a few exceptions, has not been adopted.
21	An understanding of falsework should be part of Professional Reviews.	1	Similar	Has not been adopted, and is left to the discretion of the Reviewers. The Attributes examined by ICE require an understanding of engineering principles: this may or may not have reference to falsework (or other temporary works) depending on the discipline of the candidate.
22	Short courses in falsework should be available to practising engineers.	2	Similar	Very few courses are available. Those that there are focus on contracting organisations.

Bragg Ref	Paraphrase of Bragg's Recommendation	Today's Perform.	Impact of ENs	Comments
23	Practical courses in falsework should be available to site operatives.	3	Similar	A few such courses exist. Apart from scaffolding there is no industry scheme for certification. Front line supervisors learn their skills either through supplier-run courses, or by experience. BS EN 1991-1.6 (BSI, 2005b) is the execution standard and should be taught.
24	The standard and content of the courses should be controlled by HSE.	0	No change	This did not happen and the current role of the HSE makes it inappropriate that they should certify.

The EuroNorms for Temporary Works

European harmonization of the structural design codes for construction has been taking place for around 30 years and is now viewed as substantially complete. The core advice for all structural design is contained within the 'Structural Eurocodes suite' (BS EN 1990 to 1999). These are supplemented by product standards, design guidance for particular applications and execution standards. The Eurocodes therefore provide a basis for Temporary Works design and are modified by guidance given in specific standards. All harmonized European design codes are given the designation EN, for 'EuroNorm'; in this paper we refer to them collectively as 'the EuroNorms'. The Structural Eurocodes suite is a subset of them.

It is the responsibility of the National Standard Bodies (e.g. the BSI in the UK) to implement the EuroNorms. They must contain, without alteration, the full text of the EuroNorm and its annexes as published by CEN. They can be preceded by a National Title Page and Foreword, and followed by a National Annex, which may be a separate document. All the EuroNorms use limit state principles and, as they are published, the conflicting national standards are obliged to be withdrawn[(f)].

The two main EuroNorms giving specific[(g)] design guidance relevant to Temporary Works are EN 12811-1 (BSI, 2003) for Access Scaffolding and EN 12812 for Falsework. In essence they state how the Structural Eurocodes suite should be applied to the particular applications. The introduction to EN 12812 states 'The information on structural design is supplementary to the relevant Structural Eurocodes'. They recognise that these types of Temporary Works require a different design approach using different partial factors. BS EN 1991-1.6 is the execution standard and includes loading recommendations during erection but does not distinguish between execution standards for temporary works and those for permanent works.

What Bragg might make of the 'Eurocode Era'

The success of the Bragg recommendations and their adoption in BS 5975 can be seen in the improved safety record of the UK industry. The falsework failures of the 1960s and 1970s have not been repeated in the UK.

However, if Bragg were reviewing his recommendations after the introduction of EuroNorms, there are several features which he might find caused concern.

[f] As noted above, BS 5975 has been retained and is referred to in the EuroNorm.
[g] Specific design guidance is not yet given for other types of temporary works e.g. formwork, refurbishment, façade retention, bridge propping, and ground retention.

1. **Lack of clarity on required procedural control.**

   BS 5975 has been retained: but, if it is not specified for the procedural control, there is a possibility that it will not be followed when EuroNorms are used for the Temporary Works design.

   The Eurocodes and EuroNorms which relate to Temporary Works, so far issued, do not directly address temporary works management practice. The Basis of Structural Design to the Eurocodes, EN 1990 (BSI, 2005a), is indeed fundamentally different from BS 5975: in the BS the TWC is required specifically to coordinate and take an overview, it being recognised that the different parties to the Temporary Works design and construction need this; however, EN 1990, at Section 1.3 'Assumptions', sets out who should do what and appears to establish a culture where discrete parties should be able to work together without coordinating management. Although the CDMC[(h)], appointed under the Construction (Design and Management) Regulations (HMSO, 2007), has an obligation to ensure communication between designers, there is no obligation to carry out the many other tasks which are defined for a TWC.

   It is worth restating that since the procedures outlined by Bragg (and now fully developed in BS 5975) have been adopted by industry, there have been no major falsework collapses in the UK, whereas they have continued worldwide on a, roughly, biennial basis. There is however no research into cause and effect. Because of the lack of temporary works management procedures in the EuroNorms, the pivotal role of the TWC is missing. The TWC will ensure that design, design checking and pre-load checks are complete and co-ordinated. This may require bespoke approaches specific to circumstances: the EuroNorm approach is more formulaic.

   Risk is added by the following:

   - the growing fragmentation of the industry;
   - the numerous interfaces created by procurement or management criteria;
   - the general reduction in the level of technical expertise employed at the front line on site; and
   - the progressive loss of the tradition amongst main contractors that they coordinate all engineering aspects of the construction process.

   Taken together with the potential loss of the TWC, we have a cocktail which Bragg would regard as potentially toxic.

2. **The industry is ill-informed and unready to apply EuroNorms to Temporary Works.**

   Bragg would be concerned at how unprepared and ill-informed the Temporary Works sector is in regard to the EuroNorms and would probably be disappointed that throughout the drafting and implementation of the EuroNorms for Temporary Works there has been very little engagement by the UK Temporary Works sector.

   It is the opinion of the authors of this paper and others within the Temporary Works sector that an immediate transfer to the EuroNorms within the UK would give significant difficulties and would lead to an increased risk of failures.

[h] CDMC = Construction (Design and Management) [Regulations] Coordinator

Currently the EuroNorms are insufficiently clear to allow a practising engineer to judge, in all cases, whether a given design complies with them. Because of the way some of the Temporary Works' standards have been drafted, it is difficult to interpret what the requirements are, which standards should be referred to (for different types of Temporary Works) and how the Eurocodes suite should be used. Without guidance the codes will be applied differently by different designers and checkers. This appears to be an issue across the European Union, given evidence from designers in Spain, Germany and Sweden.

The number of documents which have to be consulted to carry out a Temporary Works design to EuroNorms is legion. For a typical design involving timber in the formwork, steel for the falsework, concrete for the foundations and bearing capacity of the ground and wind loading, the designer needs to consult BS EN1990; 1991-1.1, 1.3, 1.4, 1.6, 1.7; 1992-1.1 or 1.6; 1993-1.1, 1.8; 1995 – 1.1; and 1997, together with all of their National Annexes. This is to ignore the NCCI documents available with each code[(i)]; and the plethora of safety specific legislation[(j)]. There is therefore a large time (and financial) investment required to enable first use of the EuroNorms, with a consequently long learning curve.

The EuroNorms emphasise performance requirements and a numeric approach, and place great reliance on statistical methods. The original British Standards were more akin to design guides and led the designer through the requirements. A lot of the good practical design advice in BS 5975 on issues such as effective lengths and bracing of girders is not in EN 12812 and has to be found elsewhere.

EuroNorms are written with complex formulae which require the use of a computer to be executed in a reasonable timescale. Whilst computers have brought and will continue to bring great benefits to engineering, the loss of the ability to carry out quick hand check calculations is a serious disadvantage. Although this can be overcome to some extent with the Non Contradictory Complementary Information (NCCI) published by various bodies, yet further documents have to be consulted.

The design approach given in the EuroNorms for Temporary Works is substantially different from the approach currently used in the UK and will be unfamiliar to many UK Temporary Works designers. It is currently unclear how the overall factors of safety given by the different approaches compare. The principal differences are the use of limit state design and second order analysis. The application of limit state principles to the design of Temporary Works has to date been very limited in the UK, with the vast majority of schemes being designed using permissible stress method as presented in BS 5975. Analysis of Temporary Works in the UK has traditionally used first order analysis with an augmentation factor to allow for second order effects. Design of Temporary Works seldom warrants the use of the more complicated second order analysis, which anyway needs to be used with caution, bearing in mind the underlying assumptions that will have to be made regarding, for example, lack of fit, straightness and joint stiffness.

The EuroNorms require that all limit states be considered. For a typical falsework system these might include SLS, ULS, Settlement, and Accidental load states. There are different load factors applicable to each state, and different combination factors. For simple falsework structures EN 12812 proposes a combination factor of 1.0 for all cases but does

[i] Compare this with BS 5975 which alone addresses the full design.
[j] See 'Designers' Health and Safety Map' published by the TWf: accessed Sept 2012: www.twforum.org.uk

not give advice for more complex situations. The partial factor regime is interpreted in a document published on the TWf website. The authors consider that the complexity of the regime is itself such as to introduce a safety hazard. Falsework design failures are generally attributable to a lack of understanding or knowledge on the part of the designer. The switch to EuroNorms should be regarded as a high risk activity in this regard.

3. **Inappropriate partial safety factors leading to reduced overall factor of safety.**

The EuroNorms deal in partial safety factors, intended to be derived from large volumes of test data. Whilst this is logical there is a complete lack of a database of loading information for Temporary Works on which to carry out statistical analysis to derive the appropriate load factors. Indeed, even the required research into permanent structure loads has yet to be conducted. Load factors for permanent works design have generally been reduced, based on current experience that structures designed historically have performed satisfactorily and the belief that some reduction is safe.

EN 12812 and EN 12811.1 give material partial factors for steel and aluminium in falsework and access scaffold which are greater than those in the Structural Eurocode suite. Since the majority of the load in a falsework system is live loading (a variable action), this leads to an overall factor of about 1.65. For the design of other temporary works structures, or for design in other materials, no such specific guidance is given, particularly as to whether higher factors are required. Defaulting in these cases to partial factors from the main Eurocode design suite, which are based on permanent works design, would result in a further reduction in the overall factor of safety for temporary works. There is no assurance that this is safe,

There are several significant differences between permanent structures and Temporary Works which lead to a requirement that Temporary Works partial factors are higher. These include:

- Temporary Works systems tend to have a greater proportion of variable actions and, whereas in permanent works the maximum variable loading can be expected to occur only exceptionally over its long design life, in many forms of Temporary Works (and in all forms of Falsework) the full variable loading is expected every time.
- Permanent structures tend to be more robust and have greater residual stiffness that is not taken into account in the design.
- There are frequently more unknowns in a Temporary Works system, such as lack of fit, eccentricities, corrosion and damage from previous use.

Bragg in his report recommends an overall factor of safety of 2.0 generally, with the calculations taking full account of the tolerances adopted, and, where there are novel features or methods, serious consequences of failure or lack of design staff training, a factor of 3.0. Since Temporary Works designers are currently unfamiliar with the EuroNorms, and they have to learn a large amount of information as they utilize many different materials, it could be said that for the next few years that the industry lacks trained staff. Given these unknowns and lack of trained staff Bragg would consider a factor of safety of 3.0 more appropriate than 1.65 or less. Notwithstanding, as authors, we would prefer to see effort put into training and the issuing of good authoritative advice than into simply bumping up the factor of safety.

4. Insufficient lateral stability.

Bragg clearly identified inadequate lateral stability as a major cause of failures and recommended a minimum design horizontal load of 3% of the vertical load be taken to act at the top of the falsework. Although this was later reduced to 2½% when it was incorporated into BS 5975, there have been no further failures attributed to insufficient lateral stability. EN 12812 treats lateral stability slightly differently and advises that all known horizontal forces plus 1% of the vertical load are applied horizontally to a model that takes the imperfections and second order effects into account, using partial load factors for static equilibrium. It is possible that Bragg would have been concerned whether this equivalent gave an adequate overall factor of safety, particularly as the analytical modelling of imperfections is itself far from perfect.

The relatively recent introduction of falsework systems that rely on external restraint to provide lateral stability would also have caused Bragg concern. The issue, termed "head restraint", was recognised in the 2008 revision of BS 5975. This gives recommendations for falsework depending on whether it is "restrained" or "not restrained" at soffit level. In particular, if a system is designed as restrained it is essential that the external restraint is adequate. No such recommendations are given in EN 12812.

5. **Insufficient design values provided by suppliers of proprietary equipment.**

Falsework systems use a lot of varied proprietary items and the lack of consistent published data caused Bragg concern. In his recommendations he stated that both the safe working load and failure loads be given, together with the conditions of testing. Despite 40 years having elapsed, this information is still not fully provided by suppliers to the industry. For example, tests on modern falsework system components have been carried out and safe working loads tabulated; however, in this case, the safe working loads apply only with the falsework heads laterally restrained, but this is not clearly stated in the literature, nor is it understood by some users. Another concern would be the quantity of 'copycat' equipment in circulation. At one time there were more than 8 different copies of 'Kwikstage' in circulation in the UK. Equipment therefore needs to come from an assured source, and preferably be readily identifiable.

Conclusions

Analysis of catastrophic events indicates that they occur when a number of errors coincide. This is likened to a 'Swiss cheese' (Reason, J. 1990): each slice of cheese has holes in it and occasionally the holes line up so that something can pass through the whole cheese. The larger or more numerous the holes, the more likely they are to align and allow something to pass through.

We are currently at a point in history where, for Temporary Works, it appears that the size and number of holes is likely to increase because:

- the generation brought up during the last series of major failures is retiring;
- the industry is poor at passing on knowledge;
- the industry has become highly fragmented;
- the recession in the industry in the 1990s has led to a shortage of engineers at the front line with extensive broad experience;

- changes in industry practice and procurement have led to the loss of the experienced general foreman who would often know there was a problem just by looking;
- the change to EuroNorms with more complex analysis which often can only sensibly be carried out by computer will lead to a loss of fundamental understanding of engineering behaviour; multiple load-cases also increase complexity giving rise to increased risk of errors; and
- buildings are becoming more complex and difficult to construct.

This implies that the risk of Temporary Works failures is increasing and we recommend that the 7 bullet points above are considered and where possible actions put in place to mitigate their effect. Notwithstanding, the following changes in the industry are helping:

- BS 5975 has been revised and retained so that the robust procedural controls remain in place and designers have the option of using permissible stress design if the EuroNorms are not specified.
- The profile of the procedural control within BS 5975 is higher than it has ever been and is actively indorsed by the HSE and TWf.
- Many leading clients are implementing excellent practice, especially the practice of co-location.
- Modern designs strive to reduce the number of people put at risk.
- A growing number of contractors now make safety their No. 1 priority; ethical concerns are genuine, coupled to the recognition that the financial and reputational consequences of major failures are potentially ruinous.
- The industry has combined to form the Temporary Works Forum (TWf).
- The Institution of Structural Engineers, the Institution of Civil Engineers and the Health and Safety Executive sponsor Structural-Safety, which combines SCOSS and CROSS.

The Temporary Works Forum (TWf) was formed in 2009 at an open meeting held at the Institution of Structural Engineers. The TWf is an independent, not-for-profit company which operates on a limited cost base; its primary asset is the influence of its members in the industry, and its primary value derives from their assembly on a quarterly basis. Most meetings are open (contact the Secretary for an invitation) and issues across the industry, especially the concerns of client bodies, are debated. The TWf is developing a range of expert guides published through its website.

Structural-Safety, which is sponsored by the Institution of Structural Engineers, the Institution of Civil Engineers and the Health and Safety Executive, is a body devoted to evaluating, and anticipating where possible, trends in the construction industry and issuing warnings where necessary. A confidential (anonymous) reporting system called CROSS broadcasts events which are believed should be more widely known. Many of these events are related to Temporary Works, or to lack of communication between designer and constructor. The reporting is hampered by the lack of notification, and the industry is encouraged better to publicise safety related events.

Leading UK client organisations have been outstanding in their approach to health and safety on sites. CrossRail and the Olympics Delivery Authority exemplify. During both the design and construction processes, both organisations have placed great emphasis on the CDM

aspects of design: eliminating (SFARP[k]) hazards and minimising (SFARP) the remaining risks. They have done this through co-locating the designers, through active campaigns (including toolbox talks) in the design office, using messages on the designers' screens, and by rigorous insistence on risk assessments. They have also scored the competing contractors, before their appointment, on their track record and attitude towards health and safety on site, as well as devoting whole sections of their organisation to site safety. To date, the record of safety on site of these two projects has been exemplary.

Proposals

The authors believe that the industry should see to it that the following actions are dealt with:

1. The procedural control given in Section 2 of BS 5975:2008+A1:2011 should be used for all Temporary Works. Client bodies should include it in their specifications and its application should be promoted across the construction industry particularly to the small and medium sized enterprises (SMEs).
2. Clients should be encouraged to specify that either EN 12812 or BS 5975 may be used for the design of Falsework.
3. A briefing document is produced, which states which ENs are appropriate for different Temporary Works applications.
4. EN 12812 is revised to make its requirements clear.
5. Research work is carried out to compare the overall factor of safety using the design approaches presented in the EuroNorms with those presented in BS 5975, for all types of Temporary Works.
6. An agreed summary of design parameters and requirements is produced, which sets out how Temporary Works of all types should comply with the EuroNorms.
7. Agreed design guidance and examples are prepared, which show how the parameters and requirements given above are to be applied.
8. Agreed guidance is produced, which sets out how product capacities issued by equipment suppliers are to be presented.
9. Training courses are made available to enable Temporary Works designers to become familiar with the new design methods, and to train permanent works designers and architects to identify and understand Temporary Works solutions.
10. Wider engagement with and support for SCOSS and CROSS, and the Temporary Works Forum (TWf) are promoted, especially with SMEs.

The authors gratefully acknowledge the assistance of Rob Thomas at the Institution of Structural Engineers in collating the references.

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[k] SFARP – So far as is reasonably practicable: refer to http://www.ice.org.uk/Information-resources/Document-Library/So-Far-As-Is-Reasonably-Practicable

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Forensic Engineering: Informing the Future with Lessons from the Past
ISBN 978-0-7277-5822-4

ICE Publishing: All rights reserved
doi: 10.1680/feng.58224.251

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Partial collapse of bridge falsework and salvage technique

Janusz Hołowaty West Pomeranian University of Technology in Szczecin, Poland

Introduction

As part of Poland's motorway and expressway programme, dozens of road bridges need to be constructed for every project. The majority of the new bridge structures have concrete decks either cast in situ, or precast concrete beams with an in-situ top slab. To construct these, modern formwork and falsework systems have been introduced. After several years of the accelerated motorway programme, only one serious falsework collapse with fatalities has occurred (Flaga, 2005); however, some falsework failures have been noted (Hołowaty, 2003). The paper presents the partial collapse of falsework along with a salvage project for precast beams in a road bridge constructed for one of the new expressways. The assembly of precast beams had been completed when a portion of falsework in one of the bridge spans collapsed.

S3 expressway project and the bridge

The S3 expressway project between Szczecin and Gorzów, in the north-west of Poland, was divided into three sections (contracts). The total length of the dual expressway is 81.6 km. The new expressway connects the A6 motorway via the Klucz interchange south of Szczecin with the North Gorzów interchange (Fig. 1). The number of constructed bridge structures is 77, including 43 overbridges and 34 underbridges. Four new interchanges were constructed. The S3 expressway was opened to traffic in three stages over the course of 2010. The last (southern) section was opened in December 2010. The total cost was over 530 million EUR.

The bridge, marked as PZ-56A where its falsework collapsed, is situated in the southern section of the expressway, close to Gorzów (Fig. 1). The bridge was built in 2009 as a two-span dual superstructure for two carriageways of

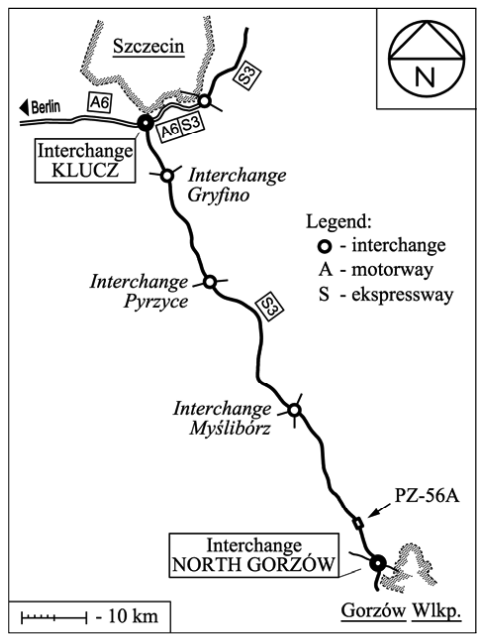


Figure 1. Site of S3 expressway project

the expressway. The structure is for environmental purposes, and gives passage for wild animals under the expressway in a forested area. The relevant section of the expressway and the bridge are located in a corridor cut through the forest. The bridge was designed, as are all expressway underbridges, for the highest class of traffic loading in Poland. For the construction, typical support and deck designs were drafted. The superstructures consist of two continuous spans with precast beams and cast-in-situ slabs and diaphragms. The length of the spans is 2×20.7 m. T-type precast beams with pre-tensioned strands (T21) were used in the deck superstructures. Precast piles driven through loose sands and soft clays were used in the foundation. The longitudinal section of the bridge at the construction phase is presented in Figure 2. The designed vertical clearance under the bridge is 3 m.

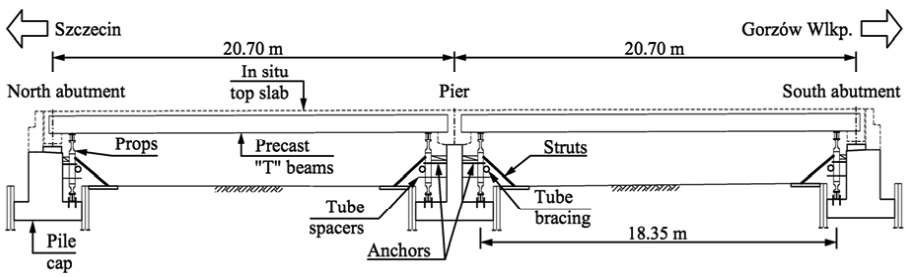


Figure 2. A two-span bridge with precast T-beams during construction phase

Erection of falsework and beams

To make the precast beams continuous over the intermediate support and to construct the end diaphragms, they were supported at the ends by temporary structures (Fig. 2). In a deck system, formworks are only needed for in-situ connections, end diaphragms and the edge sections of the overlay slab. Temporary props to support the precast beams are needed until the cast-in-situ concrete in the connections and the top slabs have hardened. Temporary supports from a commercial system, resting on horizontal steel beams on the permanent abutment and the pier foundation caps of the bridge, were used in construction. Heavy duty props of four vertical tubes in section were used. The permissible vertical loading on one prop was assessed at over 200 kN. The arrangement of the props in the cross section of the bridge near the abutment is presented in Figure 3. The same arrangement of props was used near the pier.

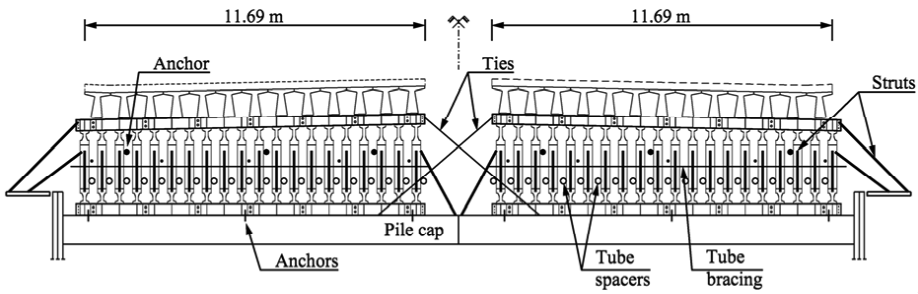


Figure 3. Falsework to prop precast beams near supports

The falsework design had been drafted by a falsework supplier using typical solutions for this system after adapting them to the existing alignment. The rows of props were braced laterally and longitudinally by external diagonal struts (Fig. 2 and 3). In the prop row plane only horizontal scaffold tubes were used. There were X-type diagonal tie bracings but only between the rows of props. The tops of the props were tied back to concrete walls of supports in longitudinal directions (three/five anchors for each prop row). These were intended to give some lateral stiffness to the prop rows but they were deformable in the row planes. In the construction of the temporary supports, brand new members in the falsework structure were used.
The execution of supporting falsework for the beams was finished and approved on 20 March 2009. After the approval of the falsework structure, the assembly of the T21 precast beams commenced. A total of 52 precast beams, with 13 beams for each span were used. The erection of the precast beams was completed on April 6. Over the next two days the formwork elements for the edge sections of the spans and the support diaphragms were assembled. The installation of the reinforcement was commenced, and bundles of stirrups in stocks were piled on top of the precast beams in the first span.

Partial collapse of the falsework

According to the contractor's staff, the temporary props supporting the beams of the first span near the pier collapsed during the night of 8/9 April. Fortunately, there were no workers on site overnight and the collapse of the falsework was claimed to have been discovered in the morning. There had been a moderate thunderstorm during the night with strong winds. A general view of the bridge structure after the partial collapse of the falsework is shown in Figure 4, with the scheme of collapsed props near the pier in Figure 5. One prop row lost its lateral stability; the props overturned and totally collapsed in an easterly direction. The props and external diagonal struts were damaged. In addition, some parts of the just-assembled formwork were damaged. A general view of the collapsed props near the pier is shown in Figure 6. The precast T-beams which the props supported relocated horizontally and vertically with the collapsed props. The beams relocated but did not overturn and were still supported by the top girder lying on the wrecked props (Fig. 5 and 6).



Figure 4. View of bridge during construction after partial collapse of falsework

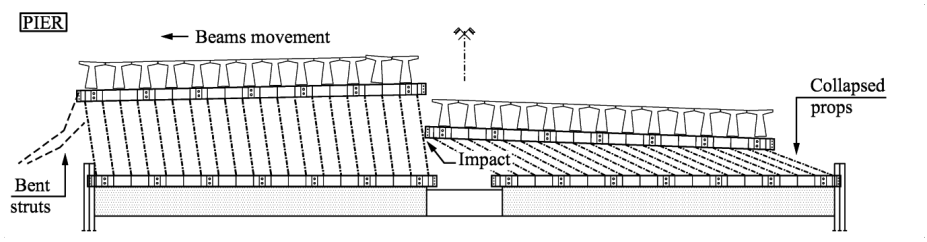


Figure 5. Scheme of the partially collapsed props near the pier



Figure 6. View of the collapsed and damaged props near the bridge pier

The opposing temporary props of the collapsed span, near the north abutment, survived but with a large lateral displacement. The collapsed props struck the props of the next dual span near the pier, causing a large horizontal displacement and the vertical props to sway; however they did not collapse (Fig. 5). The external diagonal struts were bent.

The two spans of the bridge from the southern (Gorzów) side did not collapse, although small imperfections in the verticality of the shoring props were noticed. After the collapse, the falsework was assessed not to have had adequate lateral bracing and was vulnerable to lateral horizontal loads. The lateral instability and imperfection in the assembly of the vertical props caused them to sway after the additional loading. This would have been due to the concentrated loading of the bundles of stirrup reinforcement, strong wind or other forces not described during construction, especially horizontal. However the trigger cause was not obvious. It seems unlikely that the strong winds during the storm at night would have made the props, loaded by heavy concrete beams, sway. Fortunately, the failure occurred before the concrete for the top slabs was placed in.

Failure investigation

After the partial collapse of the pier props, the construction site was closed to workers. An investigation was launched (Ratay, 2000) with an evaluation of the collapsed and still-standing props. The author of the paper was appointed to carry out an investigation into the failure. The first site inspection took one day. The configuration of the collapsed props was recorded along with the sway configurations

and those of the straight props. The collapsed props near the pier and relocation of one end of the precast beams forced the movement of the opposite props near the north abutment. The lateral movement of the top beam and the sway of the props was 50 cm. The same situation occurred with the sway in the props near the pier. The collapsing props hit the nearby row of props with their top beam, making a lateral movement of 37 cm (Fig. 5). The tie cross bracing between the rows of props which had swayed near the north abutment were bent and loose. The entire bridge structure, its temporary supporting falsework with bracings and the construction site were photographed and videoed. Deformations in the props, along with movements and rotations of the strut-supporting precast slabs were measured. Damaged members and elements of the falsework were recorded (Fig. 7). Drawings of the temporary supports near the pier and the north abutment which had collapsed and swayed were made. The falsework design was checked and verified. The concurrence of the falsework with the design was crosschecked. The stability and safety of the structure and the falsework were assessed. All possible on-site data, from the contractor, the falsework supplier and the inspector, were collected for the purposes of making a report on the possible causes of the failure and any contributing factors. One of the important findings was the presence of stirrup bundles on the collapsed span (Fig. 8).

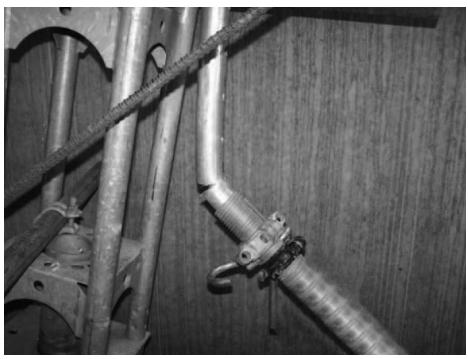


Figure 7. Broken strut near abutment



Figure 8. Stirrups on precast beams

Another (and the most important) finding was the absence of lateral cross bracings for the prop rows in the falsework design. The assembly of the falsework was in accordance with the design and no additional diagonal bracings were installed in the prop rows. Such cross bracings are used in similar commercial falsework. They are used to transmit lateral horizontal loads from the top beams to the bottom beams.

The bottom beams of the falsework were firmly anchored to the concrete foundations; however, there was no bracing for transmitting lateral loads from the top beams. There is some structural rigidity in the spindles at the top and bottom of the prop but they are not designed to transmit lateral loads. Therefore, the falsework was not designed to carry lateral loading, and when this appeared, the props lost lateral stability. Numerical verification of the stability of the props under vertical loads proved that with prop swaying there was a rapid increase in the horizontal force value.

The stability failure occurred in the northern section of the falsework; one row of props collapsed and three other rows of props swayed from 10 cm to 55 cm without collapsing. In the southern (Gorzów) section of the falsework, there were only small vertical imperfections in the props. However, there was a fear of further collapse in

the props which had swayed because they were not properly braced. The partially collapsed falsework had some relocation and movement in the precast prestressed beams, but the beams did not overturn and did not lose their vertical gravity loads balancing prestressing forces. The overturning of the precast beams might have broken them.
The final report, written by the author, found that the props were not braced adequately and lateral loads made them collapse or sway. However the trigger cause for the partial collapse was not discovered and might be external events that may have initiated the falsework collapse. The report identified that the southern part of the falsework (no sway) was at risk of collapse.
The report gave introductory instructions for stabilizing the falsework for safe and controlled dismantling of the precast beams and the falsework structure.

Salvage technique

The final report was accepted and gave the go-ahead to the contractor to stabilize the structure. A stabilizing design was drawn up, developing additional diagonal struts and cross bracings for safe dismantling. The schemes for stabilizing the partially collapsed falsework are shown in Figure 9. The same scheme of struts was used for the southern section of the falsework which had not swayed but was considered to be unstable and might lose verticality during the dismantling of the precast beams. In the stabilizing design the horizontal force in the top beams is calculated, taking into account the crossfalls of the top beams, wind forces and the sway of the props.
For carrying out the stabilizing and dismantling works, the contractor replaced the on-site work crew with a more experienced team. Strong wooden struts and standard pipe struts were installed to prevent further swaying and collapsing of the falsework. Firstly, the broken or bent system struts were replaced with new ones in the prop plane. Then diagonal wooden struts with firm supports were installed in the external parts of the falsework. These were stabilizing top beams in a lateral direction.

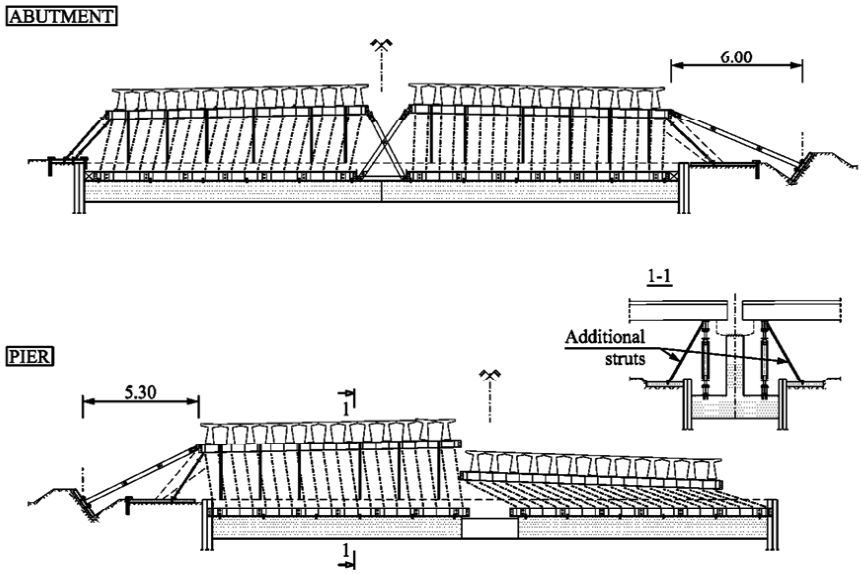


Figure 9. Schemes of stabilizing the falsework for dismantling



Figure 10. Stabilizing diagonal strut



Figure 11. Stabilizing cross bracing

Next, stiff cross bracings between the rows of props were installed. A view of an external diagonal strut near the abutment is shown in Figure 10 with a view of a cross bracing between rows of props in Figure 11. The wooden struts were prepared at a workshop as prefab members and delivered to the construction site. Some additional pipe struts were installed in a longitudinal direction. These also stabilized the top beams of the falsework (Fig. 9).
Successful stabilizing of the falsework enabled firstly the dismantling of the precast beams and then the falsework. Three mobile cranes were used in the precast beam dismantling operations. Two cranes lifted up a beam (Fig. 12) and placed it in a transition saddle near the bridge. Next, with the help of the third crane, the beam was moved to a temporary storage location (Fig.13).



Figure 12. Precast beam during the lifting process



Figure 13. Precast beams on temporary supports for inspection

Assessment of precast beams and bridge supports

The dismantling and temporary storage of the bridge precast beams ensured their verticality to prevent an unexpected release of loads. Verification and assessment of the beams following dismantling were carried out in May 2009, where it was

established that no serious damage had occurred and they would be easy to repair. Major and minor typical surface damage to the precast beams are shown in Figures 14 and 15. The same type of minor damage was discovered on the support walls.
The precast beams were re-erected on a new falsework of the same type but with appropriate cross bracings, and the bridge was completed in 2009.



Figure 14. The most serious surface damage



Figure 15. Scar in bottom edge

Conclusions

The paper presents the damage to the falsework and the initial results of the investigation immediately following the collapse. A strengthening method was applied to the surviving section of falsework to allow for the safe dismantling of the precast beams.
The paper has shown that failures can happen in typical bridge constructions adopting the accustomed falsework system. Scaffolding props with high vertical capacity may lose stability if they are not adequately braced. The adoption of the most common solution for simple falsework structures and making simplified calculations are not sufficient for safety considerations. Every bridge falsework, even that which is most typical, must be properly braced to transfer lateral loads to the ground. In this case, neglecting the horizontal forces at the design stage of the falsework resulted in the partial collapse of the system props. This happened despite approval of the falsework design and the supervision of the temporary works as if they were permanent works.
The safe bracing of temporary structures is critical in bridge construction. The correct brace must be in the correct place, taking into account the real expected forces and deformations. Load paths in falseworks must be recognized and understood to transfer vertical and horizontal loadings to the ground.

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Forensic Engineering: Informing the Future with Lessons from the Past
ISBN 978-0-7277-5822-4

ICE Publishing: All rights reserved
doi: 10.1680/feng.58224.259

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Dreamspace: Dreams turned into nightmares, a case study

Dr Rajen Kishor Lavingia, BEng, MEng, PhD, SMIIRSM, Cadogans, UK
Dr John McCullough, EurIng MSc PhD CEng CSci FIMechE FEI, Cadogans, UK
Iain Hall, BE (Mech) (Hons), CEng MIMechE, Cadogans, UK

Abstract

Dreamspace was an inflatable structure approximately 50m by 50m square and 5m in height. The structure, in the form of an artwork, was created by Mr Maurice Agis and was designed to be an interactive public attraction using colour, light and sounds, to create a spatial experience. Mr Agis had created similar structures in the past, under the name of Dreamspace and others.

The structure consisted of many sections that were glued together. The structure was inflated and secured in Riverside Park in County Durham on 22nd and 23rd July 2006. The structure was opened to the public, to walk around inside. Ropes were fed through the outer columns of the structure (and attached to pegs in the ground) which were then used to anchor the structure.

During operation on the 23 July 2006, the structure lifted up from the ground, broke free from its moorings, rose to a vertical position and travelled across the park. Unfortunately two adults were killed in the incident and numerous adults and children were injured. Mr Agis was charged with gross negligence manslaughter.

Health and Safety Executive (HSE) and Health and Safety Laboratory (HSL) carried out an investigation of the incident and prepared their reports. The reports addressed various aspects of anchoring the structure including ropes, ground anchor pullout strength. A meteorologist also prepared a report on the local wind and temperature conditions.

Cadogans examined the evidence collected by the police department, HSE and HSL, carried out analysis of the information available and prepared a report. It was argued that the theories put forward by HSE, HSL and the meteorologist did not account for all possible scenarios in their reports.

Mr Agis was convicted of a breach of Health and Safety at Work Act 1974 and fined for the same. However, the jury could not reach a verdict for gross negligence manslaughter charges.

This paper provides details of analysis carried out by Cadogans during its forensic investigation of the highly publicised incident.

1.0 The Incident

Dreamspace was an inflatable structure approximately 50m by 50m square and 5m in height (the Structure). The structure, in the form of an artwork, was created by Mr Maurice Agis and was designed to be an interactive public attraction using colour, light and sounds to create a spatial experience. Mr Agis had created similar structures in the past, under the name of Dreamspace and others.

The structure consisted of many sections that were glued together. The structure was inflated and secured in Riverside Park in Chester le Street, County Durham on 22nd and 23rd July 2006. Ropes were fed through the outer columns of the structure and attached to pegs in the ground anchored the structure.

The structure was opened to the public, to walk around inside. During operation on the 23 July 2006, the structure lifted up from the ground, broke free from its moorings, rose to a vertical position and travelled across the park until it became snagged by a nearby CCTV post. Unfortunately two adults were killed in the incident and numerous adults and children were injured.

2.0 Investigation

The Health and Safety Executive (HSE)[1] carried out an investigation of the incident and provided its opinion based on the Meteorologist's report and tests carried out by the Health and Safety Laboratory (HSL).

2.1 Ground anchors

There was a lack of clear evidence to show the exact number of anchors used to secure the structure. Investigators found 16 anchors in the ground during their investigations after the incident. Their locations are identified in Figure 1. However, the lengths of ropes sent to HSL for investigation suggest that there could have been 13 additional anchors securing the structure in position. These 13 anchors were thought to have been pulled out of the ground when the Structure lifted off.

HSL carried out Ground Anchor (peg) pull out tests around the incident locus on two different occasions, July and November 2006. The ground was dry in July whereas it was wet in November.

The pegs were inserted fully in the ground (Figure 2) to simulate the actual conditions. There is substantial scatter in the results (Figure 4). The July results show that if a peg is installed vertically and pulled out vertically the pullout load is about 560N ± 35%. If the peg is installed at an angle and is pulled out at an angle, the pullout load is about 1739N ± 45% for an angle of 145 to 165 degrees between the peg and pullout direction. For an angle of 125 to 144 degrees the pullout load is 2470N ± 30%. In other words, as the angle between the peg and rope decreases the load increases.

At higher loads there was a risk of bending the peg or distorting the ground into a mound. The load at which this happened varied between 1844 and 3365N.

In November, when the ground was wet, the pullout loads were generally lower than those observed in July. However, bending of the pegs occurred at between 2000 and 2700N apart from one that bent slightly at 990N. The loads obtained in November were not considered in the analysis carried out by Cadogans.

2.2 Ropes

The HSE relied upon the HSL to draw conclusions as to rope strength and concluded that the ropes were of lower tensile strength than would be expected for new rope. For example, new polypropylene rope of the type used should have a breaking force of 4.28kN, whereas those that were recovered from the incident had a breaking force of between 1 and 2kN.

Other selected opinions expressed by HSE in relation to the ropes used to anchor the Structure are as follows.

a) The rope strength would have been reduced by the inclusion of knots in the loops and by tying to the steel pegs when compared to a rope without knots.

b) As the rope anchor at each location consisted of two loops, this would have resulted in it being weaker than had it consisted of one loop.

c) The ropes showed signs of having been exposed to an outdoor environment for some time.

d) The ropes elongated under load by about 80mm over a 1m (1000mm) test length.

e) A total of 29 lengths of rope measuring over 10m in length were recovered and so could have formed 29 of the longer loops. However, when shorter lengths were joined by knotting there could have been a maximum of 42 larger loops.

The ropes at pegs 14, 15 and 16 (Figure 1) were thought to have been cut during active service prior to the incident. However there was no conclusive evidence to this effect.

[1] Evidence of Mr A. Hoyland given at Newcastle Crown Court, England.

3.0 Cadogans engineers' analysis[2]

3.1 Anchoring issues

The pullout tests carried out by HSL were considered in the context of the failures as a whole. The results of these tests are summarised in Figure 4. Above a pull out load of 1.84kN some pegs showed signs of bending or a mound in the earth was formed.

No bent pegs were discovered after the accident. The number of ground mounds formed was not reported. About half the pegs were still in the ground and the ropes attached to them had probably broken. The other half had been pulled out of the ground without the rope breaking. The angles of installation of pegs were not reported.

As no bent pegs were recovered, it was likely that the maximum load on the rope loops at the start of the accident, which either caused the ropes to break or the pegs to pull out, was between 2.0 and 3.0kN. Had ground mounds formed then the value could have been higher.

It was also noted that those loads were the direct load on the peg, i.e. the load through the small loop (Figure 3 and Section 3.2.1.2). This load will tend to be taken evenly by both parts of the loop. The load in the longer loop depends on how loose the rope is and what angle it is pulled to at the point at which it joins the small loop. If the load transferred to the peg is between 2 and 3kN, the total load in the main rope will be above these levels but half will be taken by each side of the loop. It is therefore possible that the initial failures were of the rope which then put greater loads on the remaining anchors pulled out of the ground.

Thereafter, if the geometry of the installation changed as Dreamspace was lifted, it is possible that some pegs were pulled out in line with their direction in the ground, i.e. with combined angles (Figure 2) nearer to 180 degrees and pullout loads below 1000N (Figure 4).

3.2 Rope issues

At the Dreamspace incident some of the ropes failed by breaking and were shock loaded; those which did not break would have been overloaded and probably pulled pegs out of the ground. In addition, it was not known what happened to the ropes immediately after the incident and during the 2 month interval until most were delivered to the HSL at Buxton. If a rope is stood upon then particles of dust and grit get driven into the rope fabric to act as little blades and reduce the strength of the rope. It is possible that the ropes were stood upon or otherwise misused before testing at the HSL.

There were 16 pegs remaining in the ground after the incident, probably because the ropes failed first by breaking. Of those, 11 and their associated ropes were sent to the HSL. It is known that for correctly angled pegs the pull out load can be up to 3kN. As far as is known none of the 11 pegs sent to HSL was bent but it is not known whether earth mounds formed. Therefore, it cannot be concluded that the ropes did not have a breaking strength of between 2 and 3kN. Of the 29 (16 found anchors + 13 potential anchor ropes, Section 2.1), some 16 ropes broke (because the pegs remained in the ground). The remaining 13 pegs must have been pulled out of the ground and that must mean that those ropes did not break. So, those ropes could have experienced a load in excess of 3kN without breaking. Therefore it was opined that it is unsafe to conclude, as the HSE did, that the incident ropes had a breaking strength of between 1 and 2kN.

3.2.1 Specific rope-related issues raised by HSE

3.2.1.1 Strength

It is well known and accepted that a knot reduces the strength of a rope; the extent of the reduction depends upon the type of the knot formed. It is also accepted that if a rope is tied round an object like a metal peg, the strength will be reduced. During the HSL tests on knotted and tied samples the breakages did occur at the knots and ties to the pegs as would be expected.

[2] Expert witness report of Dr. J. McCullough dated October 2008

However, on the large loops from the incident which failed by breakage due to overload, the breaks did not occur at the knots. On the small loops that failed by breakage, there was no evidence whether the breaks occurred at the pegs. The condition of the ropes reported by HSL would suggest that breakage did not occur at the pegs. Failure of small loop was reported by HSL at only one location. However, there was no evidence if it occurred at the peg. Therefore, while knots should, ideally, be avoided, it was opined that they were not relevant to the cause of the incident.

3.2.1.2 Number of loops

The view that if the rope anchor at each location had consisted of one loop instead of two the anchor would have been stronger is based on the fact that the combined angle would have been lower and therefore the effective strength of the rope greater. That is the case but it is not as simple as that.
The situations of double and single loops are shown in Figure 3. A longer length of anchor rope (large loop) was wrapped around the cell of the structure which was connected to the peg by a small length of rope (smaller loop). While the angle for the single loop would be smaller, it results in a more vertical load applied to the peg. So, the effective strength of the rope could improve with a single loop having reduced number of knots among other factors, the resistance to pullout of the peg could reduce. In addition, of the 29 anchors, 13 pegs were pulled out of the ground; so their ropes did not break even though they were double looped.

3.2.1.3 Previous Exposure

The ropes were polypropylene and it is known that such ropes are sensitive to UV (ultra violet) light. So, if the Dreamspace ropes had been exposed to an outdoor environment for months some deterioration in strength would be expected. However, it was not possible to quantify any such deterioration partly due to the effects of overloading and potential driving in of dust and grit particles.

3.2.1.4 Elasticity

Elasticity of the ropes was not considered a significant factor in the cause of the accident and therefore it is not discussed further.

3.3 Wind and Air Temperature

The Meteorologist opined that the effect of the weather on the structure would be to cause the air inside it to heat up. The resulting reduction in air density would then encourage the structure to lift off. An internal temperature of 44°C was assumed in the HSE investigation, under normal circumstances with a very light structure like Dreamspace. However, at Dreamspace the blowers which kept the structure inflated were blowing air through the internal spaces. The resulting cooling effect would have reduced the internal temperature, probably to around the upper 20's or lower 30's centigrade. This view is supported by the fact that members of the public were willing to spend time inside Dreamspace. 44°C would have been extremely unpleasant.

The video evidence suggests rising at the south west and/or northwest corner at the start of the incident. Therefore, inability to secure the central part of the Structure would not be relevant providing the edges could be secured adequately.

3.4 Calculations for anchor loads and rope loads

Two different anchoring arrangements were considered during analysis – (i) 29 anchors equally distributed around the Structure and, (ii) 16 anchors with only 2 located on windward side. The latter case represents the arrangement presented by HSE.

Mean and gust wind speed considered during the analysis were based on the data supplied in the meteorologist's report which are as given Table 1.

Anchor loads and rope loads as calculated by Cadogans engineers are given in the Table 2.

The calculated results indicate all loads were below the assumed limits for anchor pegs (1.6kN) and ropes (2.5kN) with the exception of those shown in red. The red values shown for Case B indicate

that the rope and anchor limits were exceeded in the two anchor points on the windward side at an assumed wind speed of 7.3 m/s. However, if it is assumed that 4 (rather than 2) of the 16 anchor points are located on the windward side (there was an inconclusive evidence that 4 anchors were provided on windward side), this reduces the loads to marginally above the assumed anchor limit and below the rope load limit i.e. anchor peg and rope loads of 1.7 kN and 2.4kN respectively.

Table 1 Wind data used in the analysis of anchor loads and rope loads

Wind data used in Cadogans' analysis		
Height from Ground	Mean wind speed (m/s)	Gust wind speed (m/s)
2.5m	3.3	5.7
5.0m	3.9	7.3
Wind data used in HSE's analysis		
	3.6	8.2

Table 2 Anchor loads and rope loads, as calculated by Cadogans engineers

Wind Speed	m/s	5.7	7.3
CASE A			
Total Number of Anchors		29	29
Anchor Distribution		Equally Distributed	Equally Distributed
Anchor Load	N	355	622
With Safety/Design Factor (x1.5)	N	533	934
Maximum Rope Load	N	503	880
With Safety/Design Factor (x1.5)	N	754	1,320
CASE B			
Total Number of Anchors		16	16
Anchor Distribution		2 Anchors only on Windward Side	2 Anchors only on Windward Side
Anchor Load	N	644	1,319
With Safety/Design Factor (x1.5)	N	966	1,979
Maximum Rope Load	N	911	1,866
With Safety/Design Factor (x1.5)	N	1,367	2,798

The Meteorologist's report identified the possibility of thermal lift in the proximity of the Structure. Anchor loads and rope loads for the thermal lift as calculated by Cadogans engineers are given in the Table 3.

It is noted that the anchor loads and rope loads indicated for this case are lower than those for the conventional wind load case (at the same wind speed). However the loads are significant and it can be seen that the anchor peg load and rope loads limits are exceeded for Case B_thermal.

4.0 Further analysis by Cadogans engineers

4.1 If the anchor pegs had been driven into the ground at an angle as intended (and there is no evidence that they were not) and rope load was applied at an angle (and not in line of the peg), which it would have been, then the pullout load or resistance of the anchor pegs (in dry ground as on the

day of the incident) would have been sufficient. For a combined angle of between 125° and 144° the load or resistance would have been about 2.5kN and possibly as high as over 3kN per peg.

Table 3 Anchor loads and rope loads for the thermal lift, as calculated by Cadogans engineers

Wind Speed	m/s	7.3
CASE A_thermal		
Total Number of Anchors		29
Anchor Distribution		Equally Distributed
Anchor Load	N	333
With Safety/Design Factor (x1.5)	N	499
Maximum Rope Load	N	471
With Safety/Design Factor (x1.5)	N	706
CASE B_thermal		
Total Number of Anchors		16
Anchor Distribution		2 Anchors only on Windward Side
Anchor Load	N	1,207
With Safety/Design Factor (x1.5)	N	1,810
Maximum Rope Load	N	1,707
With Safety/Design Factor (x1.5)	N	2,560

4.2 It is generally accepted in climbing and mountaineering that if a rope is shock loaded it will be weaker and unreliable. The pre-incident rope strengths would have been greater than those determined during the HSL breakage tests and would have been adequate (Section 3.2). Factors such as possible mishandling of the ropes prior to the tests and shock loading during the incident among others would have reduced the strength of the ropes tested by HSL. The rope breaking strength could have been between 2 and 3 kN and possibly above 3 kN for those ropes that pulled pegs out of the ground, i.e. which did not break.

4.3 With the wind speeds that pertained at Riverside Park on the day, around the time of the incident, and assuming 29 anchor locations deployed around the structure, there was insufficient wind load to cause Dreamspace to lift off.

4.4 HSE calculated an anchor load of 1.9 kN each for a wind speed of 11.1 m/sec. Using the same method, Cadogans engineers estimate the value to be about 1.5 kN

4.5 HSE also estimated, using BS 6399, that 80 anchors would have been required for a wind speed of 11.1 m/sec. Using BS EN 14960, which was not available at the time of the construction of the Structure, HSE arrived at 108 anchors for 11.1 m/sec wind speed. The numerical accuracy of the latter assessment was agreed but not the velocity of 11.1 m/sec which, although part of the standard, did not pertain on the day. Using examples of lower wind speeds which were around at the time, the following values were obtained.

Table 4 Number of anchors required for a given wind speed

Wind Speed (m/sec)	No. of Anchors Required
8.2	60
6.7	40
4.8	20
3.3	10

The wind speeds of 8.2 and 6.7 m/sec were gusts and there was no evidence of whether they occurred at the time of the incident. Using the lower values as suggested by the Meteorologist, it can be seen that 29 anchors (which roughly match the number of large loops and pegs recovered) would have been adequate. So Dreamspace should not have lifted off; but it did.

4.6 The HSE was of the opinion that the ropes at pegs 12 and 13 failed first due to wind load and the resulting increased load transferred to the remainder causing them to unzip. An alternate view was formed that HSE assumed a pessimistically large value of the combined angle, too low a rope strength and too few anchors on the west side (i.e. there could have been at least 1 or 2 more between pegs 12 and 13). Therefore, it was opined that HSE's conclusion was not a safe one. The video evidence did not include the entire structure and in particular the north west corner (i.e. where peg 13 was located) rendering it inconclusive to suggest an initial failure somewhere in that region.

If the ropes at pegs 14, 15 and 16 had been cut whilst the ropes were in active service as anchors (say due to vandalism), the loads they should have taken would have been transferred to the remainder, possibly causing further failures and unzipping. However, this theory has the problems of low wind speed and for those 3 pegs, wind direction. The Meteorologist's report indicated that the nearby buildings and trees could have caused local wind variations. If there had been a sudden change of wind direction, with or without a gust, the load that should have been taken by the cut ropes would have been transferred and could have initiated further failure.

4.7 The HSE also assessed the effects of buoyancy considering an internal temperature of 44°C and 16 anchors. With a more probable internal temperature of 34°C (as explained previously), Cadogans engineers calculated a lift due to buoyancy equivalent to 20% of the weight of the structure. However the weight of the structure plus the holding down force of some 30 anchors exceeds the buoyancy forces by just over an order of magnitude (just over a factor of 10). So, buoyancy of itself cannot be a cause of lift off but it could be a contributory factor.

4.8 Thermal lift is a different phenomenon caused by uneven heating from the sun of the ground and/or structures. It creates local pockets of hot air which rise and are known as thermals. The Meteorologist is of the opinion that thermal lift could have been present at the Riverside Park on the day of the accident.

It was also regarded as possible that wind loads played a major role in the failure mechanism; but as discussed previously, the anchoring provisions were probably both strong enough and of sufficient number to resist the wind conditions on the day.

4.9 Therefore, a possible alternative explanation had to involve thermal lift (air flow) and buoyancy in addition to the wind. The wind was light and the sun had heated the Dreamspace structure and its immediate surroundings to a greater extent than Riverside Park generally. The structure could have reached the sol-air temperature of over 40°C. A thermal air flow as described previously was initiated resulting in lift and horizontal forces on the structure. Combined with the buoyancy force, this caused some of the ropes to become overloaded. For example, regardless of whether failure was initiated around pegs 12 and 13; as 14, 15 and 16 had been cut, all the load on the north face would have transferred to 18, the only other remaining anchor on that side. When, in turn, 18 failed by overload, the load would have transferred to anchor 1, the only one on the east face; it too would have failed by overload. The failure would then have spread to the remaining anchors on the south side. As the Structure began to rise, it became exposed to the increased wind speed at higher level. By now a sufficient number of the remaining anchors had unzipped to allow the Structure to be transported by the wind until it became snagged by the CCTV post.

If there had been 29 anchors located around the structure, a similar failure mechanism could have occurred but the resistance to it would have been greater. The probability of this depends on the wind speed that could be induced by thermal air movement above the Structure.

The video evidence was of interest; it showed Dreamspace starting to move or rise vertically, which could suggest thermal lift; it seems to show (but as all the structure is not included one cannot be sure) initial anchor failure at the north west corner, as suggested by HSE; it then indicates failure spreading down the north face where the cut ropes were located and spreading along the east face

and finally to the south face. Therefore, the influence of thermal lift and buoyancy was considered a credible alternative explanation to the simple wind theory proposed by HSE.

5.0 The Trial and Verdict

Mr Agis was charged with gross negligence manslaughter. He was convicted of a breach of the Health and Safety at Work etc. Act 1974 but the jury was unable to reach a verdict on the manslaughter charge.

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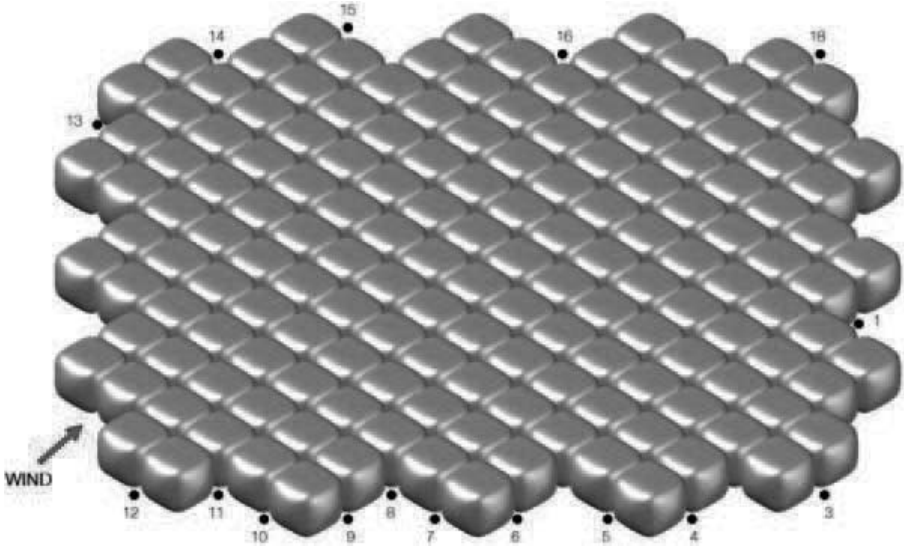


Figure 1 Position of Anchors (Pegs) as found in the ground in relation to position of Dreamspace Structure prior to the Incident

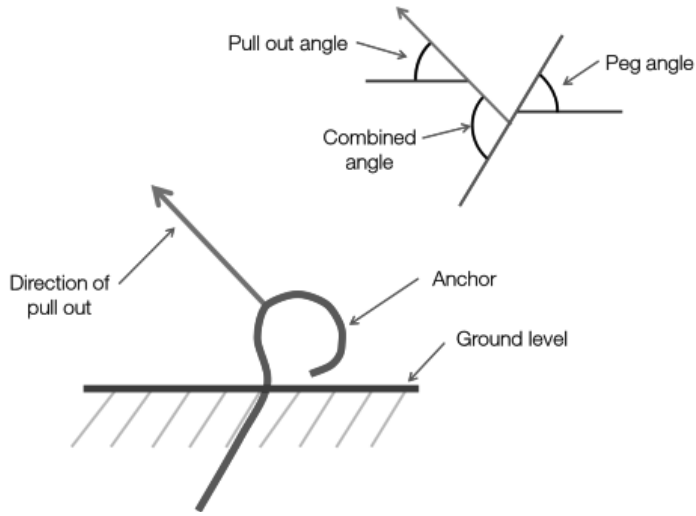


Figure 2 Definition of Ground Anchor Angles

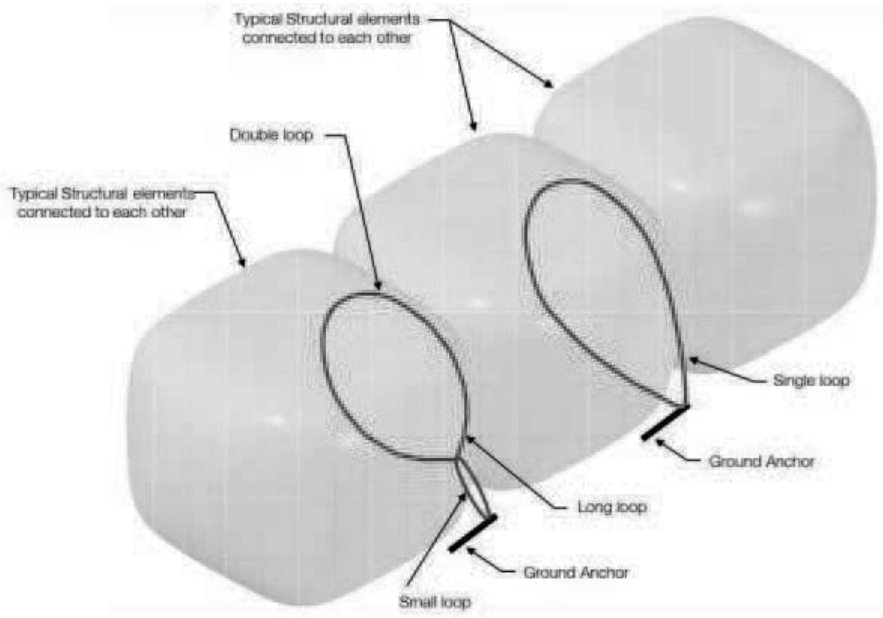


Figure 3 Explanation of single loop and double loop, all the anchors were connected by a double loop

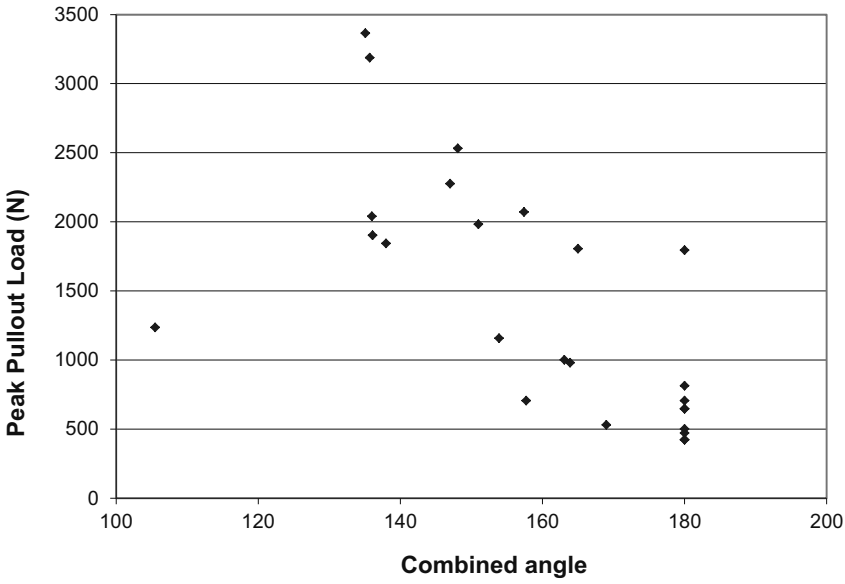


Figure 4 Relationship between pullout load and combined angle between the rope and the peg

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SECTION 6:

EDUCATION AND CONTINUING PROFESSIONAL DEVELOPMENT IN FORENSIC ENGINEERING

Forensic Engineering: Informing the Future with Lessons from the Past
ISBN 978-0-7277-5822-4

ICE Publishing: All rights reserved
doi: 10.1680/feng.58224.271

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Forensic Engineering Education Workshops: U.S. and International Experiences

Kenneth L. Carper, R.Arch., M.S.CE, M.ASCE, Prof. Emeritus, Washington State University, USA
Paul A. Bosela, P.E., Ph.D., F.ASCE, Prof., Cleveland State University, USA
Norbert J. Delatte, P.E., Ph.D. F.ASCE, Prof., Cleveland State University, USA

Abstract
The Committee on Education of the ASCE Technical Council on Forensic Engineering promotes inclusion of lessons from forensic investigations in the undergraduate and graduate professional engineering curriculum. As part of this effort, the Committee presents workshops for civil engineering faculty. The workshops are aimed at integrating failure case histories into existing courses. These workshops have been successfully presented at seven locations throughout the United States to more than 100 college and university professors. The workshops provide easily adapted case modules, visual aids, and proven teaching strategies. Recently, the project has been extended to the international arena, with presentations in the UK, China and Latin America. This paper reviews the background, intent, and content of the forensic engineering education workshops, with a focus on the international activities.

Introduction
Following the success of a series of forensic engineering case history workshops presented to civil engineering educators throughout the United States, the workshops have more recently been offered in several international forums. This project has evolved under the direction of the Technical Council on Forensic Engineering (TCFE) of the American Society of Civil Engineers (ASCE). The workshops, presented by the TCFE Committee on Education, have as their principal goal the integration of lessons from failure case histories into undergraduate and graduate professional education. The workshops provide case descriptions and visual aids, along with suggested strategies for their effective use.

To date, nine workshops have been presented in the United States to more than 100 college and university professors. Similar workshops in the UK, China, Costa Rica and Ecuador have been well received. Future international workshops are under discussion.

Construction materials, methods and project delivery organizational systems vary from country to country. Dispute resolution techniques and litigation procedures also differ substantially in the international arena. However, many of the lessons that can be learned from past forensic investigations apply universally. Because of this universality, engineering educators throughout the world find the information and teaching strategies presented in the workshops to be helpful.

ASCE Technical Council on Forensic Engineering
The ASCE Technical Council on Forensic Engineering (TCFE) was founded in 1985 following a series of high-profile structural collapses in the U.S. that attracted widespread media and governmental attention. The accompanying introspective review within the design community determined that the professions had been remiss in failing to develop appropriate strategies for learning from our failures. The TCFE's mission is to improve the practice and professional status of forensic engineering, including the quality of forensic investigations, ethical conduct for expert witnesses, and effective failure information dissemination throughout the industry. A full review of the TCFE formation, committee organization, and scope is given in published references (Carper 1998, 2003, 2011a).

The current standing committee structure of TCFE includes:

- The *Committee on Dissemination of Failure Information* (CDFI) develops and implements means for disseminating accurate and complete information regarding the nature and causes of structural failures in civil-engineered projects. CDFI also acts as a vehicle for coordinating and assisting in the dissemination of forensic engineering information generated by other organizations, both within and from outside ASCE.
- The *Committee on Practices to Reduce Failures* (CPRF) develops objectives for change in the design and construction industry specifically targeted to reducing the incidence and severity of failures in civil-engineered projects. CPRF also works toward implementation of these changes and acts as a liaison between ASCE and other professional societies and organizations to pursue the prevention of failures.
- The *Forensic Practices Committee* (FPC) works to enhance the competent and ethical practice of forensic engineering. FPC develops practice guidelines, seminars, tutorials and conference sessions aimed at elevating the understanding of competent and ethical practice by professionals engaged in the investigative and judicial arenas.
- The *Committee on Education* aims to improve the practice of Civil Engineering by promoting the study of failure case histories in educational activities. The committee develops course materials and acts as a source of referral for educational materials with forensic engineering emphasis.
- The *Committee on Forensic Investigation* develops and defines procedures for conducting forensic investigations on constructed facilities.
- The *Publications Committee* reviews manuscripts, technical notes and discussions submitted to the Council for publication in the *Journal of Performance of Constructed Facilities (JPCF)* and is responsible for publication of papers sponsored by the Council. (The first author of this paper was the founding chief editor for JPCF and continues to serve in that capacity.)

Accomplishments of TCFE include enhancing the quality and safety of constructed facilities and improving the image and practice of forensic engineering (Carper 2005, Delatte and Carper 2010). Products of TCFE committee activities include the *Journal of Performance of Constructed Facilities*, a number of books and monographs (see ASCE bookstore website under “Forensics”), and six international Congresses with published proceedings (Minneapolis 1997, San Juan, PR 2000, San Diego 2003, Cleveland 2006, Washington, DC 2009, and San Francisco 2012). Over the past two decades, TCFE has established liaisons with a number of international professional organizations, including the Institution of Civil Engineers (ICE), Institution of Structural Engineers (IStructE), and Standing Committee on Structural Safety (SCOSS) in the UK; the Chamber of Forensic Engineers of the Czech Republic; and the new Association of Forensic Engineering in Italy. Forensic engineering organizations are also emerging in Ireland, Taiwan, China and India. Through these liaisons (and through related JPCF activities) several international delegations of TCFE practitioners and scholars have been funded.

Most of the TCFE standing committees involve educational activities. For example, the Forensic Practices Committee regularly presents workshops on ethical and competent forensic engineering practices and publishes educational documents relating to practice. Likewise, the Committee on Dissemination of Failure Information organizes continuing education conference sessions for design practitioners that present information from forensic investigations. These sessions influence improvements in design practice. The TCFE Committee on Education, however, is focused on enhancing “failure literacy” as part of the professional education of future engineers. The Committee on Education was one of the first standing committees formed by the TCFE Executive Committee. Founding members of the Council recognized the important task of including failure related information in the civil engineering curriculum, so that our failures are not repeated by the next generation of civil engineers.

TCFE Committee on Education: Educator workshops project

Activities of the TCFE Committee on Education encourage the integration of failure case histories into the undergraduate and graduate education of civil engineers. The Committee publishes failure case histories and monographs, organizes conference sessions and presentations for engineering educators, and maintains Internet sites prepared specifically for educators and classroom access.

For a more complete review of TCFE Committee on Education activities and products see Carper, Delatte and Bosela (2008) and Delatte and Rens (2002).

It is in this context that the educator workshops are presented. Between 2003 and 2010, workshops were offered in Birmingham, Alabama (2003), Cleveland, Ohio (2004, 2005, and 2006), Denver, Colorado (2007), Pittsburgh, Pennsylvania (2008), Austin, Texas (2009), and Louisville, Kentucky (2010). A similar event is planned for San Francisco, California in 2012. The U.S. workshops are funded by ASCE and by the U.S. National Science Foundation (NSF). The purpose of the workshops is to provide assistance to educators who wish to integrate failure case histories into their civil engineering and engineering mechanics courses. More than 100 college and university professors have attended the workshops, influencing the content of courses throughout the United States.

The typical workshop occupies one day (or one-half day) of instruction. Early workshop participants were provided with a CD containing numerous failure case histories organized for implementation in the classroom and a binder of case study materials. Since 2009, they have been given a copy of the book, *Beyond Failure: Forensic Case Studies for Civil Engineers* (Delatte 2009). Additional resources are presented, such as the ASCE/TCFE Committee on Education monograph, *Failures in Civil Engineering: Structural, Foundation, and Geoenvironmental Case Studies* (ASCE/TCFE 1995, 2012), and useful failure case history websites including: *http://matdl.org/failurecases* and *http://failures.wikispaces.com*. Interactive discussion on proven strategies for use of these materials in the classroom forms a valuable part of the workshop, especially in the longer workshops. The one-half-day workshops are limited in this respect.

TCFE international delegations and international outreach activities by the *Journal of Performance of Constructed Facilities* have resulted in invitations to present educator workshops internationally. Successful workshops have been presented in London, Costa Rica, China, and Ecuador. Ongoing discussions suggest future workshops in Colombia, India, Taiwan and Ireland, among others (Delatte and Carper 2010).

The ICE sponsored the *Fourth International Conference on Forensic Engineering: From Failure to Understanding*, in London, December 2008 (Carper, Delatte and Bosela 2008, Carper 2009a). Prior to the Conference, three members of the TCFE Committee on Education presented a half-day workshop, “Embedding Forensic Engineering into the Undergraduate Curriculum,” closely patterned on the workshops offered in the United States. The London workshop was attended by more than 40 participants, including both educators and practitioners representing 11 countries. This was the first workshop to introduce the book, *Beyond Failure* (Delatte 2009), which was very well received. Following the workshop, ICE conference staff collected reviews from the participants and prepared a statistical summary. Despite the brief format of the workshop, it was considered a very successful event by all participants and by the presenters.

Four TCFE members presented a forensic engineering education workshop, “Taller de Ingeneria Forense para Profesores de Ingeneria,” in Cartago, Costa Rica, July 23-24, 2009 (Carper 2009b). The workshop was hosted by the Instituto Tecnologico de Costa Rica. Approximately 60 engineering educators and practicing engineers from throughout Costa Rica attended. The workshop was held over two full days, which permitted time for a very productive panel discussion session to conclude the second day. The workshop was fully funded by the Costa Rican hosts. In addition to the education component of this event, discussions were initiated with leading engineers in Costa Rica regarding the potential establishment of a forensic engineering organization in that country, patterned after the committee structure of ASCE/TCFE.

In March 2010, six ASCE/TCFE members traveled to three universities in China during a two-week period (Carper 2010). Two-day forensic engineering education workshops were presented at Shandong University, Jinan; at Zhejiang University, Hangzhou; and at Hohai University, Nanjing. ASCE provided airfare. The costs of lodging, meals, and travel within China were provided by the three host universities. Following the well-attended and interactive workshops, formal meetings were held at each university with faculty, engineering practitioners, and industry representatives. Current research activities were reviewed, including tours of facilities. Opportunities for future joint projects, such as potential exchanges and joint sponsorship of international conferences, were discussed. The Chinese hosts expressed interest in forming a professional committee patterned after the TCFE in order to facilitate future collaborations. One member of the delegation was invited back to Hohai

University in 2011, and to Southeast University in 2012, both located in Nanjing. Further collaborations are under discussion, including a substantial participation of representatives from China to the Sixth ASCE Forensic Engineering Congress in San Francisco, October 31-November 3, 2012.

Guayaquil, Ecuador, was the site of a workshop, January 7-8, 2011 (Carper 2011b). The event attracted 54 attendees, including 40 professional engineers and educators, and 14 students. Topics included an introduction to forensic engineering practice; investigation techniques; the engineer as expert witness; dispute resolution; the ethics of forensic practice; forensic engineering education; and numerous case histories involving failures of buildings, bridges, dams, and other constructed works. The workshop was presented by the same team as the Costa Rican event, although the program was modified slightly from that in Costa Rica to include a wider range of topics. It was supported by a number of professional and educational organizations. Guayaquil is the largest city in Ecuador, a small country located between Colombia and Peru, in northwestern South America, and it is home to the ASCE Ecuador Group. Prior to this workshop, very few people in Ecuador had heard about the discipline of forensic engineering.

Discussions during and following each of these workshops revealed major differences in professional licensure requirements, specific materials and methods of project delivery, and substantial differences in dispute resolution techniques, including litigation procedures. Despite these differences, some topics were clearly universal, such as engineering principles, lessons from failures, and proper failure investigation techniques. The participants and the presenters alike learned much from these discussions, made possible by the longer duration of the two-day format.

Evaluation of workshop effectiveness

Until recently, measuring the effectiveness of including failure-related information in classroom instruction has been limited to subjective assessments and anecdotal evidence, although such evidence has been consistent and impressive. While the value of such material in the classroom is clear to those who have introduced case histories at appropriate student maturity levels, the hard evidence has been lacking. Similarly, the effectiveness of the forensic engineering educator workshops has only recently been measured. A U.S. National Science Foundation (NSF) grant funded a four-year project at Cleveland State University that concluded in 2009, titled, “Assessing the Impact of Case Studies on the Civil Engineering and Engineering Mechanics Curriculum.” The project demonstrated the positive impact of these strategies. The results were also incorporated into the workshops.

A third, larger NSF funded assessment project, “Implementation and Assessment of Failure Case Studies in the Engineering Curriculum,” is a four-year project that started in 2009. For this project, the work was expanded beyond Cleveland State University to include eleven other universities. These projects, and their results are discussed more completely in a companion paper to be presented at this conference (Delatte, Bosela and Bagakas 2013). Additional reports have also been published on this work (Delatte, Bosela and Bagakas 2009, and Delatte, Roberts, Ralston, Brady, Zoghi, Hagerty and Yu 2010).

Statistical surveys and comments received from educators who attended past U.S. and international workshops are yielding the same results. For example, following the 2008 half-day workshop presented at the Fourth London Conference on Forensic Engineering, numerous positive comments were received from the more than 40 participants representing 11 countries. A survey conducted by the ICE conference staff reported that 100% of the participants rated the event as “Excellent” or “Good” (78% gave it the highest possible rating, “Excellent.”) 100% of the respondents wrote that they would attend a future event on this topic, with many preferring a full-day format. Unfortunately, economic considerations prevented either a repeat or an expansion of this workshop in conjunction with the Fifth London Conference on Forensic Engineering.

Use of failure case histories in professional education

The value of including failure-related case histories as a formal component in the professional engineering curriculum has been well documented (Delatte and Rens 2002, Carper 2005, Carper, Delatte and Bosela 2008, and Delatte and Carper 2010). Presentation of case histories can serve to illustrate technical topics. As students contemplate the consequences of failure, their motivation for

mastering theoretical principles is enhanced. The historic importance of some of the more influential failures can develop in students an appreciation for the roots of their chosen profession, and an understanding of the role failures have played in the evolution of design standards and construction practices. Presenting the ethical dimensions involved in failures encourages discussion of many critical non-technical aspects of project engineering. Most importantly, studying failures can impart tangible evidence of the solemn public responsibilities inherent in civil engineering practice.

Despite the recognized benefits of including this information in the curriculum, there remain substantial obstacles (Carper, Delatte and Bosela 2008). Educational institutions and programs are sometimes susceptible to inertia that resists modifications or the introduction of new ideas. There is the very real problem of the crowded curriculum, with many competing demands for subjects to be covered. This crowded curriculum condition makes it nearly impossible to introduce new stand-alone required courses.

The most successful strategy for introducing this material is through case history modules that can be integrated seamlessly into existing courses. The ASCE/TCFE Committee on Education has developed case history resources that enable educators to accomplish this task with less effort, along with proven classroom techniques to maximize their benefit.

Prominent engineering practitioners, from Robert Stephenson in 1856 to Jonathan Wood in 2012, have been vocal advocates for inclusion of failure case histories in the education of the next generation of civil engineers. In 1856, while serving as President of the Institution of Civil Engineers, Robert Stephenson addressed a meeting of the ICE with these observations:

> "Nothing is so instructive to the younger members of the profession as the record of accidents in large works, and of the means employed in repairing the damage. A faithful account of those accidents, and of the means by which the consequences were met, is really more valuable than a description of the most successful works. Older engineers have derived their most useful store of experience from the observation of those casualties which have occurred to their own and to other works, and it is most important that they should be faithfully recorded in the archives of the Institution." (R.
> Stephenson quoted in Hammond 1956)

And from Jonathan Wood (2012):

> "Information from the rigorous investigation of failures provides the best basis for improving design and construction practice…Each new generation of engineers must learn 'the history so far' if they are to innovate successfully and avoid the repetition of mistakes of previous generations."

In the United States, similar recognition for the concepts presented in this paper has come from practitioners, in their support for the ASCE "Body of Knowledge" project that defines accreditation standards more clearly. Many of the standards can be most effectively met through the study of engineering failures.

Some of the practitioners who attended the London forensic engineering education workshop in 2008 noted, with regret, the absence of leading UK academics. If needed changes in educational programs are perceived by practicing engineers, they can play an important role in influencing those changes. Their input is usually welcomed as educational objectives evolve in academia. Their regular contributions in the classroom are also valued, especially as related to project delivery discussions, where human frailties are often more prominent than the technical issues as the root causes of failure.

Conclusions

Forensic engineering education workshops for civil engineering educators have been presented throughout the United States and in several international forums. They have been well received and future related international events are planned. These workshops equip engineering educators with resources and strategies for integrating lessons from failure case histories into classroom instruction, principally as modules in existing courses. The lessons reinforce technical concepts taught in the classroom. They also stimulate critical discussion on the non-technical aspects of project delivery,

such as professional ethics and the importance of clear communications among the various disciplines contributing to successful project completion. While specific materials, construction methods, and litigation procedures vary widely throughout the world, the success of the educator workshops demonstrates that the causes of failure have common roots and that the lessons from failure investigations can be applied universally.

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Forensic Engineering: Informing the Future with Lessons from the Past
ISBN 978-0-7277-5822-4

ICE Publishing: All rights reserved
doi: 10.1680/feng.58224.278

ice
Institution of Civil Engineers
publishing

Implementation and Assessment of Failure Case Studies in the Engineering Curriculum

Norbert Delatte, Professor and Chair, Civil & Environmental Engineering Department, Stilwell Hall Room SH 107, Euclid Avenue at East 24th Street, Cleveland State University, Cleveland, Ohio 44115, USA
Paul Bosela, Professor, Civil & Environmental Engineering Department, Stilwell Hall Room SH 121, Euclid Avenue at East 24th Street, Cleveland State University, Cleveland, Ohio 44115, USA
Joshua Bagaka's, Professor, Curriculum and Foundations Department, Cleveland State University, Cleveland, Ohio 44115, USA

Introduction

Lessons learned from case studies have substantially affected the practice of civil engineering and other engineering disciplines. The history of the development of practice in many engineering disciplines is, in large part, the story of failures, both imminent and actual, and of the changes to designs, standards and procedures made as the result of timely interventions or forensic analyses. In addition to technical issues, concepts such as professional and ethical responsibilities are highlighted by the cases. Pilot studies have been carried out over several semesters in order to assess the use of failure case studies in civil engineering and engineering mechanics courses at Cleveland State University (CSU). Student learning has been assessed through surveys as well as focus groups, led by researchers from the CSU College of Education and Human Services. Students were asked specifically about the technical lessons learned, as well as their response to the case studies. Case study questions were included on homework assignments and examinations. Survey questions linked student achievement to learning outcomes. The focus groups identified additional benefits to the use of case studies.

A new project extended the work of implementing and assessing case studies from CSU to eleven other university partners as well as the Materials Digital Library (Kent State University) for a total of thirteen universities participating in the project. The participating universities and faculty members (professors) are listed in Table 1.

Each of the 11 subcontractors (Kent State University excepted) was asked to:

1. Use case studies in at least 2 courses during the subcontract period.
2. Collect student survey data for students participating in the courses and provide to Cleveland State University (CSU), using forms developed at CSU and modified during this project.
3. Collect student survey data from underrepresented groups in engineering, specifically women and minorities. Since few subcontractors, if any, have enough women and minorities in engineering to be statistically significant, aggregating the data will make it possible to determine overall trends.

4. Participate in faculty surveys during the project
5. Participate in at least one faculty focus group during the project.
6. Contribute at least one new case study to the case studies web site http://matdl.org/failurecases/Main_Page (Delatte, 2012b)

Table 1: Participating Universities, Faculty, and Programs (Delatte et al., 2010)

University	Faculty Member(s)	Programs
Cleveland State University	Norbert Delatte, Paul Bosela, Joshua Bagaka's	Civil Engineering, Engineering Mechanics
University of Wisconsin Platteville	Matt Roberts, Keith Thompson	Civil Engineering
University of Louisville	Patricia A Ralston, Donald J. Hagerty, Jim Lewis, Dave Wheatley	Engineering Fundamentals, Civil Engineering
Southern Polytechnic State University	Gouranga Banik	Construction Management
Case Western Reserve University	Xiong (Bill) Yu	Civil Engineering
Lamar University	Mark C. Bourland	Civil Engineering
Pennsylvania State University	Kevin Parfitt, Andrew Scanlon	Architectural Engineering, Civil Engineering
California State University, Fresno	Manoochehr Zoghi	Construction Management
University of North Carolina at Charlotte	Shen-en Chen, Tara Cavalline	Civil Engineering
California Polytechnic State University	Pamalee Brady	Architectural Engineering
Colorado State University	Becki Atadero, Marvin Criswell	Civil Engineering
University of Alabama	Philip Johnson	Civil Engineering
Kent State University	Laura Bartolo	Materials Digital Library, Center for Materials Informatics, College of Arts & Sciences

Case studies require students to synthesize the facts and engineering principles they have learned, and combine them with their broader education in the arts, humanities, and sciences. Case studies tie together technical aspects, ethical issues, and procedural issues and require students to undertake higher order thinking in order to synthesize the relevant issues. The case study products of this research will help civil engineering educators improve their teaching of specific technical topics within the discipline.

The university partners in this project include public and private institutions, undergraduate-only programs and universities with doctoral programs, and programs in civil engineering, engineering mechanics, engineering fundamentals, architectural engineering, and construction management. All were in the U.S. In U.S. practice, a program is a course of study leading to the awarding of a degree. Involving this broad range of universities and programs will help spread the word about the beneficial impacts of case studies. The surveys conducted will also verify and strengthen the argument for using case studies.

Education Activities

The American Society of Civil Engineers Technical Council on Forensic Engineering (ASCE TCFE) Committee on Education has a focus on integrating failure cases and forensic engineering into the undergraduate and graduate engineering curriculum, both in the U.S. and internationally. One of the early products of this committee was a monograph entitled Failures in Civil Engineering: Structural, Foundation, and Geoenvironmental Case Studies (Shepherd & Frost, 1995). The monograph had approximately 40 short case studies. This report is currently out-of-print, and was revised by the Education Committee with an addition of about 10 new cases for publication in October 2012 (Bosela et al. 2012).

A 1989 survey of 175 accredited civil engineering programs by the Education Committee showed that 85% of the schools included failure examples in technical courses. Only 9% had a stand-alone course devoted to engineering failures. 15% held regular seminars on failure-related topics. Of the respondents, 63% indicated they would teach more failure-related courses if suitable resource materials were made available. Twelve courses taught at ten major research universities were reviewed in articles summarizing this survey (Bosela, 1993, Rendon-Herrero, 1993). In 1998, a second survey of 300 accredited programs found that the majority recognized the importance of including failure-related information, but the consensus was that the undergraduate curriculum was too crowded to require additional courses devoted to the topic (Rens et al., 2000). Many responses indicated that integration into existing courses is the only option available. A number of alternative methods for integrating the material efficiently into the curriculum were suggested.

Subsequently, the U.S. National Science Foundation funded a series of projects on integration of failure case information into undergraduate courses, and on assessment of the results on student engagement and mastery of course material. As part of this work, the TCFE Education Committee conducted a series of annual workshops at various locations throughout the United States (Delatte et al., 2007, Carper et al., 2008). The locations included Birmingham, Alabama (2003), Cleveland, Ohio (2004, 2005, 2006), Denver, Colorado (2007), Pittsburgh, Pennsylvania (2008), Austin, Texas (2009), and Louisville, Kentucky (2010). The workshops provided assistance to educators who wished to integrate failure case histories into their civil engineering and engineering mechanics courses.

To date over 100 college and university professors have attended these workshops. The workshops were either a day long or a half day (since 2009). Workshop participants were provided with a CD with dozens of prepared presentations summarizing case studies. The presentations were designed to be either used directly in the classroom, or modified by the instructor. Until summer 2008 participants also received a binder of case study materials. In subsequent workshops, they received copies of the book *Beyond Failure* (Delatte, 2009) instead. Similar workshops have also been delivered internationally, in London (2008), Costa Rica (2009), China (2010), and Ecuador (2011). Additional details on international activities are provided by Delatte & Carper (2010).

There are a number of web sites on failure case studies. Two sites that have been developed by members of the TCFE Education Committee include the Materials Digital Library site "Failure Case Studies: Civil Engineering and Engineering Mechanics" (http://matdl.org/failurecases/) (Delatte, 2012b) and Professor Kevin Parfitt's site "Failures Wiki – Building, Architectural and Civil Engineering Failures and Forensic Practices"

(http://failures.wikispaces.com/) (Parfitt, 2012). The Wiki site was first set up to support a course in the Architectural Engineering program at Pennsylvania State University.

Through an NSF grant, a consortium of 12 U.S. universities agreed to test the use of failure cases in the classroom and to report and analyze assessment data. This work has led to a diversity of approaches to teaching with failure cases, which can be shown through alternate ways of teaching the same case. Detailed examples are provided by Delatte (2012a).

Student Survey Results

The surveys and focus groups have been carried out in classes at Cleveland State University and elsewhere since 2007. As an example, student survey responses from the two Spring 2008 courses are presented in Tables 2 and 3. The two courses were a mechanics of material course at the sophomore level and a construction planning and estimating course at the senior level. The sophomore students were in the second year of a four year Bachelor's degree engineering program, and the senior students were in the fourth year.

Table 2 suggests which ABET student outcomes may be considered to be strongly supported by the failure case studies. ABET "student outcomes describe what students are expected to know and be able to do by the time of graduation. These relate to the skills, knowledge, and behaviors that students acquire as they progress through the program." (ABET, 2011). In the US, ABET accredits engineering programs. The scale ranged from 1 – strongly disagree to 5 – strongly agree. Each class was between 20 and 30 students in size. Since the surveys and focus groups were held in class, virtually all of the students enrolled participated.

In 2007, the students in both classes rated ability to apply knowledge of mathematics, science, and engineering; understanding of professional and ethical responsibility; and knowledge of contemporary issues at 4 or higher on average. The sophomore students also rated the broad education necessary to understand the impact of engineering solutions in a global and social context above 4.

The senior students rated ability to function on multi-disciplinary teams; ability to identify, formulate, and solve engineering problems; recognition of the need for, and an ability to engage in life-long learning; and ability to use the techniques, skills, and modern engineering tools necessary for engineering practice at 4 or higher. All average results were 3.33 or higher. This suggests that the failure case studies can be important for enhancing learning of all ABET student outcomes.

With the 2008 results, shown in Table 2, the averages in both classes for ability to apply knowledge of mathematics, science, and engineering; understanding of professional and ethical responsibility; and knowledge of contemporary issues were all 3.81 or higher except for one value of 3.56. All averages were 3.50 or higher. Overall, these 2008 results are similar to the 2007 findings.

The students were also asked to rate the relative contributions of the textbook, lectures, homework, projects (if any), exams, and the case studies to their interest and understanding of the course material. In 2007, case studies were ranked at 4.33 (highest) and 3.88 (second highest) for interest by the sophomores and the seniors respectively, and at 4.0 for understanding (tied for third highest, highest). These show strong reinforcement of the specific course technical material. The 2008 results, shown in Table 3, ranked case studies at

4.04 for sophomores and 4.50 for seniors for contribution to interest. These were the highest values. 2008 contributions to understanding were 3.73 for sophomores and 4.50 for seniors. The 2007 and 2008 results were roughly comparable.

Table 2: Student Survey Responses related to ABET Outcomes (Spring 2008) (Delatte et al., 2010)

	Strength of Materials Sophomore Students				Construction Planning and Estimating Senior Students			
The case studies contributed to:	Ave	SD	High	Low	Ave	SD	High	Low
my ability to apply knowledge of mathematics, science, and engineering;	3.89	0.89	5	2	3.81	0.54	5	3
my ability to design and conduct experiments, as well as to analyze and interpret data	3.56	0.75	5	2	3.81	0.40	4	3
my ability to design a system, component, or process to meet desired needs, using the principles of equilibrium;	3.80	0.87	5	2	3.94	0.77	5	3
my ability to function on multi-disciplinary teams	3.54	0.98	5	1	3.50	0.82	5	2
my ability to identify, formulate, and solve engineering problems;	4.19	0.57	5	3	3.88	0.72	5	3
my understanding of professional and ethical responsibility	4.52	0.70	5	3	4.88	0.34	5	4
my ability to communicate my problem solutions effectively;	3.85	0.91	5	2	4.00	0.73	5	3
the broad education necessary to understand the impact of engineering solutions in a global and social context	4.08	0.74	5	3	4.38	0.62	5	3
my recognition of the need for, and an ability to engage in life-long learning	4.16	0.69	5	3	4.63	0.50	5	4
my knowledge of contemporary issues	4.15	0.78	5	3	4.00	0.73	5	3
my ability to use the techniques, skills, and modern engineering tools necessary for engineering practice	4.00	0.89	5	2	4.00	0.63	5	3

The summary statistics indicated that students benefited most from case studies in the following ways:

1. Making the course more interesting.
2. Helping students broaden their understanding of the impact of engineering solutions in global and social contexts.

3. Making students aware of their professional and ethical responsibility.
4. Increasing students' ability to apply knowledge of engineering to real life situations.

Since these pilot studies have been completed at CSU, the research team has been gathering additional data from other universities for analysis.

Table 3: Student Survey Responses on Relative Contributions (Spring 2008) (Delatte et al., 2010)

How well did each of these elements contribute to your INTEREST in the course material?								
Course	Strength of Materials Sophomore Students				Construction Planning and Estimating Senior Students			
	Ave	SD	High	Low	Ave	SD	High	Low
Textbook	3.00	1.13	5	1	3.00	1.00	4	1
Lectures	3.92	0.84	5	3	4.38	0.96	5	2
Homework	3.69	0.88	5	1	3.50	0.63	5	3
Projects	3.26	0.92	5	1	3.15	0.90	4	1
Exams	3.42	1.06	5	2	3.56	0.73	5	3
Cases	4.04	0.87	5	2	4.50	0.63	5	3
How well did each of these elements contribute to your UNDERSTANDING of the course material?								
Textbook	3.54	1.07	5	1	3.18	1.17	5	1
Lectures	4.08	0.74	5	3	4.50	0.89	5	2
Homework	4.00	0.85	5	2	4.13	0.81	5	2
Projects	3.09	0.95	5	1	3.18	1.54	5	1
Exams	3.54	1.14	5	2	3.88	0.72	5	3
Cases	3.73	0.87	5	2	4.50	0.63	5	3

Summary and Conclusions

Over the past three decades, the Technical Council on Forensic Engineering (TCFE) of the American Society of Civil Engineers (ASCE) has worked hard to establish and promote forensic engineering as a respected profession. Efforts in the past decade have been strongly focused on engineering education. This paper has reviewed the development of TCFE in the U.S. with particular emphasis on educational activities. Considerable work has been done in the area of integrating failure case histories, failure case studies, and forensics into graduate and undergraduate engineering education. In particular, TCFE members and colleagues have developed and tested innovative methods of teaching with case studies.

Case studies require students to synthesize the facts and engineering principles they have learned, and combine them with their broader education in the arts, humanities, and sciences. Case studies tie together technical aspects, ethical issues, and procedural issues and require students to undertake higher order thinking in order to synthesize the relevant issues. The case study products of this research project will help civil engineering educators improve their teaching of specific technical topics within the discipline.

The university partners in this project include public and private institutions, undergraduate-only programs and universities with doctoral programs, and programs in civil engineering, engineering mechanics, engineering fundamentals, architectural engineering, and construction management. Involving this broad range of universities and programs can help spread the word about the beneficial impacts of case studies. The surveys conducted also verify and strengthen the argument for using case studies. Data have also been collected and analyzed to demonstrate the benefits of including failure case information in courses. The primary investigative methods were student surveys and focus groups.

Details have been reported by Delatte et al. (2009, 2010) and Delatte (2012a). Delatte et al. (2009) reported that the assessment instruments and processes developed during the project had yielded useful results. The students enjoyed the case studies and believed that they contributed to learning the course material. The case studies stimulated their interest. Furthermore, most faculty who had participated in the one-day case study workshop and who responded to the survey had made at least some use of the cases in their courses. Although the range of responses was wide, responses indicated that it was not particularly difficult to include case studies in courses, the workshop materials were helpful, and the usefulness to students was high. However, they also reported that a significant time commitment was often needed to incorporate case studies. All fourteen respondents that had used case studies believed that the benefits justified the effort required. A number of them requested additional case study materials. Delatte et al. (2010) reported similar observations.

Acknowledgements

This work was sponsored by the grant "Implementation and Assessment of Failure Case Studies in the Engineering Curriculum," National Science Foundation Project DUE-0919487, $ 500,000, August 1, 2009 – July 31, 2013. Opinions expressed are solely those of the authors and not of the National Science Foundation. This work was also sponsored by a number of groups of the American Society of Civil Engineers.

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Forensic Engineering: Informing the Future with Lessons from the Past
ISBN 978-0-7277-5822-4

ICE Publishing: All rights reserved
doi: 10.1680/feng.58224.286

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Failure case databases for teaching risk in civil engineering

Denys Breysse, University Bordeaux 1, I2M- Civil and Environmental Engineering Dep., CNRS UMR 5295, France

Abstract
Risk management in construction has only recently become a topic of interest for French engineers or researchers. It was only in 2003 that MR-GenCi the first French national organisation for Risk Management in Civil Engineering, was founded by the author. It now brings together more than 20 partners (authorities, builders and consultants, academics, insurers…). The appreciation of failure analysis has developed progressively among the community of experts and engineers. It is now important to open a new perspective by showing the value of a shared failure database. Since there is no common failure database for the moment, the author took the initiative of developing a first prototype version. This database now contains more than 830 failure cases. It has been developed from various sources: books dedicated to failure analysis, technical reports on case studies, already existing databases, and information provided through the Internet. For many reasons this prototype database,cannot yet be considered as a research tool, but merely as a demonstration tool to show what can be done, once organisational problems are identified and solved.
This database can be used (among other purposes) for teaching, both using specific failure cases and at a more synthetic level. Information contained in the database is useful for incorporation in a range of courses like geotechnics, materials science, tunnelling, bridges, reinforced concrete etc. It also has direct application for teaching risk management in civil engineering..

Keywords: acceptable risk, case studies, database, failure analysis, statistics, teaching,

Introduction: failure as a source for success in civil engineering
Collapse and structural failure cannot be considered as just an “Act of God”, since rational reasons can always be found. Often these relate to the sociological, contractual and economical context of when and where they happen. In construction, explanations for failure often focus on material properties or loading: failure happens when the load has more severe effects than the material can bear, without forgetting possible deterioration with time (Breysse, 2009a). However, even if this “mechanical logic” would seem to explain the accident, human factors at any stage (design, building or service) need to be considered for full understanding (Breysse, 2003). Human error can come from bad decisions, insufficient knowledge, understanding or communication problems… Failures can also be the result of risks which are consciously considered to be acceptably low (in one case out of four), either because their elimination is technically impossible, or economically not acceptable (Schneider, 1997).

Case studies on past failures makes it possible to identify their causes (certain, probable or possible), to analyse them and to draw some conclusion (Bordes, 2005). These conclusions enrich collective expertise. Feedback justifies the gathering process so that lessons can be learnt from an unforeseen (and often dramatic) event. Feedback can be qualitative or quantitative, based on case studies or on statistics of similar failures. The theory of the learning process after failures and catastrophic events has been studied by (Duffey, 2005). The detailed investigation requires both a high level of expertise and extensive means (laboratory tests, numerical simulations...). Rigorous investigation is essential

when the judicial/legal process examines questions of responsibility. Developments in the dispute resolution process, mainly in Anglo-Saxon countries, gave birth to forensic engineering (Carper, 2007). Forensic engineering can be seen as comparable to medico-legal analysis: it is a (*post-mortem*) retro-analysis after the failure, to identify the root causes, and unravel the logical chain of causes and consequences which finally resulted in the collapse. This is important for courts considering the allocation of responsibility, but of greater importance for adding to engineering knowledge.

Figure 1, drawn after (Wood, 2008), illustrates how forensic engineering can be seen as participating in a whole process: the « forensic dance ». The four possible uses of failure analysis are:

- legal matters and responsibility,
- research and improvement of knowledge,
- education and learning,
- improvement of standards and practices.

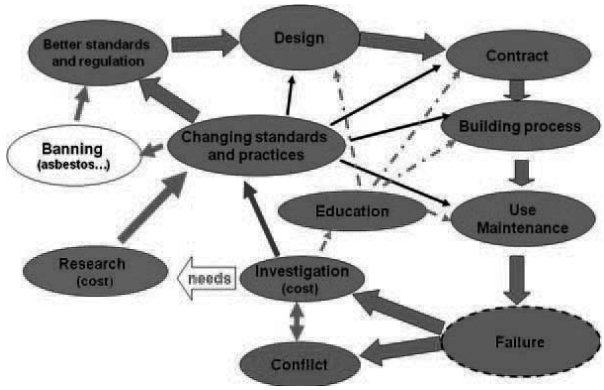


Figure 1. Forensic dance (after Wood, 2008).

In France, the interest in analysis of structural failures emerged only recently. It was in 2003 that a first national structure, named MR-GenCi, for Risk Management in Civil Engineering, was founded after the author's initiative. It now brings together more than 20 partners (authorities, builders and consultants, academics, insurers…). In April 2011 its first conference in Paris was devoted to forensic engineering (Breysse et al, 2011).

However, France has not been free of structural collapses. The most famous collapse case in France is that of Malpasset dam, whose sudden failure, on 02/12/1959, caused 423 fatalities in the town of Fréjus. This case is known worldwide as one of the most devastating collapses in Western countries in modern times. The subsequent investigations led to a general improvement in dam management strategies. The design could not be faulted on the then state of the art, as there was then a lack of knowledge of the risks induced by high pore pressures in the fractured rocks below the dam. Finally, because the failure and its consequences were comprehensively documented, further analysis is still possible, for instance for calibrating flooding models after a sudden dam collapse.

Significant collapses have occurred more recently like: (a) the Auguste Perret, Meteor tunnel collapse, in Paris, on 14th February 2003, (b) the Roissy airport, Terminal E, roof collapse, on 23rd May 2004 (4 fatalities), (c) the Saint-Etienne River bridge collapse, on La Réunion island, in the Indian Ocean, on 25th February 2007. These three collapses were technically analysed by experts nominated by the Ministry of Transport and the experts' report was made public (even if briefly and then withdrawn in the second case). However this new initiative of public authorities was not followed by any collective action (either from the government or from academics or from the construction industry itself) which would focus on the analysis of failures for improving knowledge and practice. We will discuss here how the development of a database has been undertaken and how the information it contains can be used for teaching purpose.

Potential usefulness of failure databases

Failures or accidents can be analysed and used at various levels:

- at a (micro) qualitative level, when a case is strong enough to draw conclusions which cannot be forgotten. Many spectacular failure cases, mostly is the U.S., have been the topic of such analysis (Saint Francis and Teton dams, Ronan Point-London, Skywalk in Hyatt Regency, WTC buildings, New Orleans levees after Katrina, etc...);
- at a meso level, when apparently isolated cases reveal after thorough analysis that some patterns may be generic. This makes it possible to go from the particular to wider conclusions with practical applications. This is a usual approach in earthquake engineering, in order to assess the vulnerability of buildings, but it has also been used for bridges (George, 2010), silos (Carson, 2000), geotechnics (Gue et al, 2000), snow loads on roofs (O'Rourke, 2008), etc...
- at a (macro) statistical level, when repeated occurrences of failure events may enable the identification of trends and quantification of occurrence probability and risk indices. This approach has been applied to earthquake engineering (Lee et al, 2002), bridges (Bailey et al, 2001; Imhof, 2004), tunnelling (Shin et al, 2009), fire safety in buildings (Brassell and Evans, 2003), reliability of wind turbines (Kozine and Rademakers, 2004), scour development and bridge failure (Briaud and Hunt, 2006), etc...

Past worldwide experience and existing databases

The development of failure databases facilitates the organisation of information and records it for the future. It helps to disseminate information worldwide. It has particular value in identifying factors which give rise to disproportionate risk which only become clear from the analysis of larger scale data as distinct from individual cases. Table 1 provides an overview of existing databases of failure events. Three criteria distinguish between these databases: (a) the field, which can be a specific type of structure, all types, or even wider, extending to other fields of engineering or to other types of disasters, (b) the level as discussed above, which can be that of a simple case-study or more general, (c) the objectives, which can be that of a simple catalogue/list or more ambitious.

Field	Level	Objectives	References
D: Bridges	1	Catalogue of failure cases (part of a wider database)	[A] Brueckenweb
D: Bridges	1	Catalogue (short description)	[B] Bridge Forum
D: Light structures	1	Catalogue	[C] Eilon
C	1	Detailed analysis of failures, teaching	[D] Delatte
C	1	Detailed analysis of failures, teaching	[E] MRGenCi
C	1	Detailed description of accidents, with historical sources	[F] GenDisasters
B	1	Detailed description of accidents, with historical sources	[G] Engineer's aspect
B	1	Collection of selected resources about major types of failures	[H] Engineering failures
B	2	Knowledge database, detailed analysis of failures, looking for scenarios of failure	[I] Hatamura
C	3	Knowledge base for dissemination and analysis	[J] Scoss-Cross
C	2	Interactive knowledge database	[K] Failures Wiki
A	2	Interactive knowledge database	[L] TerritorioScuola
A	3	Data repository	[M] NIST

Content: A: disasters (among which structural collapse), B: engineering disasters, C: construction (various types of structures and infrastructures), D: specific; Level: 1 = micro, 2 = meso, 3 = macro

Table 1. Overview of some databases available for studies on failure events in construction.

Delatte [D] develops his Web database as a companion support to his book (Delatte, 2009) where he provides innovative course materials for teaching engineering design and analysis through failure case studies. This project intends integrate research in forensic engineering into undergraduate education (Delatte, 2000). Every year from 2003 to 2010, faculty workshops on teaching with failure case studies have been held. The database documents about 40 cases. Several course sections are based on the analysis of these failures covering: Statics and Dynamics, Mechanics of Materials, Structural Analysis,

Design of Reinforced Concrete Structures, Design of Steel Structures, Soil Mechanics and Geotechnical Engineering, Fluid Mechanics and Hydraulics, Construction Materials, Management, Ethics, and Professional Issues.

MRGenCi [E] database follows a similar objective. It has been developed in the frame of a collaborative project between French academics, devoted to teaching Risk in Civil Engineering (Niandou et al, 2009). Full courses have been made public on the Internet, among which a small part is devoted to failure case studies.

Gendisasters [F] is an extensive catalogue about disasters (including those due to structural collapses) viewed as "tragic accidents that became a part of history and our genealogy". It presently documents about 400 cases of disasters, which are classified by type and US State. Engineer's Aspect [G], the Nanette South Clark's blog, contains a lot of information presented from the engineer's aspect. Failure cases are only a part of the content, which contains a historical review, with a copy of original press clippings and a detailed description of the cases. The lack of rational classification however makes this information sometimes difficult to find. Engineering failures [H] is a collection of selected resources about major types of failures and incidents. It is intended to be a starting point for research, not an exhaustive list. It is however of very limited size (9 bridge cases, 6 building cases, 2 dam cases).

Hatamura's work [I] was very specific. In the frame of a Japanese research project, he developed a database, named "failure knowledge database" (FKD), covering various fields of engineering, among which construction. The database contains 1136 failure cases. The information related to each case is the following: case name, pictograph, date, place, location, overview, incident, sequence, cause, response, countermeasures, knowledge comment, background, incidental discussion, scenario, sources, deaths/injuries, field, author. Hatamura asked: *"is it possible to develop useful and consistent models for failure, from the analysis of unique cases, since they can be many but also remain specific?"*. That issue of bridging the gap between a descriptive and naturalistic approach to case studies and a generic approach has no easy answers. The knowledge is modelled on the basis of scenarios which are a logical chain of causes, actions or countermeasures and consequences (Hatamura, 2007). Hatamura built a model for each of these three steps by organising the possible options inside "mandalas". For instance, the mandala of causes organises them according to whom is responsible (typically one individual, the organisation or nobody), see fig. 2. Associative research was carried out later in order to analyse the failure reports and to find the "most analogous accidents" (Nakao et al, 2008).

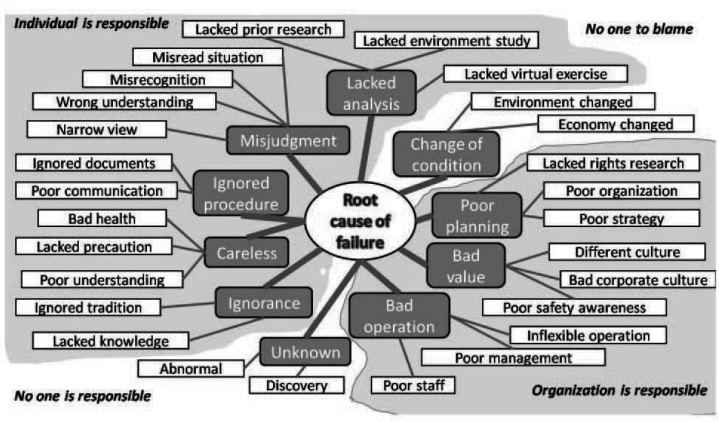


Figure 2. Mandala of causes, after Hatamura,

The CROSS reporting system developed in the UK, under the auspices of SCOSS (Standing committee for the structural safety), which disseminates useful data while keeping sources confidential, and offers an attractive way to add value to feedback [J]. The CROSS process (« confidential report on structural safety ») was initiated to reduce the number of accidents (or near-

accidents) on structures, and to improve their reliability throughout their construction and service life. The method involves obtaining feedback from professional engineers (through confidential reports) and writing synthetic reports to identify risks and to influence practice. The database facilitates searching by topic or keywords.

Failures Wiki [K] is an interactive repository for the dissemination of failures information and educational materials related to building structures, architectural systems and civil engineering infrastructure. The site was created at Penn State in 2009 and has been seen by the American Society of Civil Engineers as a means of encouraging the educational dissemination of failures information to practitioners, educators and students. The knowledge database not only contains case studies but also analyses on specific topics (progressive collapse, legal issues, ethics, etc…). TerritorioScuola [L] is a project recently developed project by Wikipedians in order to better organise information in articles related to Disaster Management (construction collapses are, among industrial accidents, as one source of disaster). The database contains bridge failure cases (more than 1500), dam failure cases, building failure cases and others. Information for failure cases is the following: location, country, date, construction type, use of bridge, reason, number of death/injuries, damage and comments. Three features are of particular interest:

- it is a collaborative project, to which anybody interested can contribute by expanding and/or correcting information,
- knowledge is really organised as in a database: as the project promoters argue: "*There is a lot of duplication of efforts and confusion of terminology surrounding disaster management on Wikipedia now. […] The current categorization is also far from great, lacking in structure and logic. The task to clean up in this domain is immense, but it has to be done*." The categorisation is by: (a) natural disasters, (b) accidents, (c) man-made disasters. Accidents are then categorised into: (a) transport, (b) industrial, (c) health. It is among industrial accidents that one finds structural failures (which include buildings, bridge, dam, levees, mast and towers).
- The project promoters also point the need to assess the data relevance. Each article is then assigned two scores:
  - importance (or priority), assessed on a 4-level scale, from *low* to *top*, dependant of its relevance regarding the project objectives,
  - quality, assessed on a 8-level scale, from *item in a list* to *featured article*, dependant on the amount and relevance of the information.

  On 2012, October 31, among the 2823 articles of the database, only 89 were assigned one of the three best scores on the importance scale, and only 129 were assigned one of the two best scores on the quality scale.

A new project has been launched recently (Sunder, 2011), under the auspices of NIST [N] with public funding of 1.8 million US$. It will be developed in two phases. Phase 1 has just been completed and includes data from NIST's six-year investigation of the collapses of three buildings at New York City's World Trade Center (WTC 1, 2 and 7) as a result of the terrorist attacks on Sept. 11, 2001. Phase 2 will include a larger collection of information on hazard events such as earthquakes, hurricanes, tornadoes, windstorms, etc… and man-made hazards (accidental, criminal or terrorist).
Data will be analysed to improve the understanding of hazards, the real-world performance of buildings and infrastructure during disaster and failure events at both the component and the system levels. The results of disaster and failure studies will lead to recommendations to provide disaster-resilience at the structure and community levels through improvements to building codes, standards, and practices and identification of gaps in current knowledge about buildings, infrastructure, emergency response, and human behaviour.

Challenges for failure databases

The databases summarised in Table 1 illustrate a large variety of contexts and methods, even if a common purpose is to collect, analyse and disseminate information on failure events, in order to improve professional practice. Teaching is only one means for this improvement and very few databases (mainly [D] and [E]) have been designed with a teaching purpose. Several questions must be kept in mind during the definition, implementation and development of a database:

- to what purpose the data will be used? Will this information be prepared for students? For experts? for general audience?
- What resources will be available for the development and maintenance of the database? There is a large difference in databases developed from an individual initiative, those responding to a public demand or those developed by a group of experts. The durability of the

database, even when the initial funding is public is a real question, since nothing ensures that sufficient means will be available on the long-term. Several examples among those at Table 1 show that, after an initial stage of development, everything can stop once funding disappears. From this point of view, the collaborative development of databases on Wikipedia may be a very practical possibility.

- The main weakness of such collaborative projects, which do not correspond to a well-defined programme, is that they may lack of organisation. A database must be organised so that it is not only easy to find information, but also to use it for further studies and analyses.
- Another essential issue is that of data validation. The fact that data is validated through some process (e.g. review by experts in [J], collective judgment in [L]) is important. A huge amount of information of variable reliability is available on failure cases, consisting of press clippings for most part of new cases, and one must consider how this information can be combined and distinguished in a database which can also contain validated cases
- Last, one must consider how representative the database is of the variety of existing cases. Even if data is gathered methodically, collecting biases always exist. This is less important as long as data are used as catalogues or for illustrative purposes, but it can bias results when carrying out statistical analyses.

Development of a prototypal failure database

Gathering of information

In France there was little interest in analysing failure cases in Civil Engineering until the research group Risk Management in Civil Engineering (MR-GenCi) was founded in 2003 (http://www.mrgenci.u-bordeaux1.fr/). It now groups more than 20 partners (authorities, research teams, insurers) involved in risk and safety issues. This group has been directly involved in the development of teaching resources on risk and safety in civil engineering (see [E] and (Niandou et al, 2009)) and in the organisation of the first national event on forensic engineering in 2011 (Breysse et al, 2011; Breysse, 2011). However, there is not yet a collective effort for gathering failure or near-failure data and analysing it. The civil engineering profession and construction industry are only just beginning to consider these issues. There is a substantial reluctance to share and spread information about problems and failures. Information is said to be confidential, which is sometimes true because of judicial matters. Since no common failure database exists for the moment, the following considerations are based on a first initiative promoted by the author.

Statistical picture of the information collected

The prototype database is developed through various sources: technical reports on case studies, already existing databases and information provided through the Internet. It does not follow any formal gathering process, which prevents the database being fully representative. Some specific arbitrary choices have been made and some types of failure cases are not documented: those due to earthquakes or accidental actions like gas explosion (in fact few cases which seem to be interesting may be included, but their number is very limited). Each case is first coded in an Excel® spreadsheet, with the following fields: location, country, date, type of structure, fatalities/injuries, main and secondary cause, position in the life cycle (construction, service, maintenance, demolition), miscellaneous, references. In addition, the full information is also stored, for further analysis. It may consist into press clippings, technical and scientific papers or reports, pictures or videos...

The database currently contains more than 830 cases. Table 2 indicates the geographical distribution of cases for the 2007-2011 period and Figure 3 shows how the number of fatalities varies on the last decade.

Year	Europe	America	Asia	Africa	Oceania	total
2007	8	19	20	6	-	53
2008	11	17	20	7	-	55
2009	20	23	49	9	1	102
2010	17	35	53	12	3	120
2011	8	17	46	17	1	89

Table 2. Number of post-2007 cases contained in the database according to the geographical origin.

It can be seen that the statistical distribution between geographical areas or types of structures is relatively stable across the years. Both Table 2 and Figure 3 show the main weakness of the database: (a) a more systematic effort after 2005-2006 for collecting data leads to a significant increase in the number of documented cases, therefore of fatalities, (b) since these efforts focus on sources in French language (author's native language) or English, this leads some regions to be under-represented. It is the case for instance for South-American countries or Eastern Asia. The high number of cases from Asia is mainly due to the contribution of China and India where respectively 49 and 74 failures have resulted in 681 and 602 victims respectively. During the same period the failure/fatality ratio is 36/36 for France and 59/52 for U.S. This shows that while the average number of fatalities per failure for France and U.S. is about unity, it is between 8 and 12 for China and India. This shows that: (a) failure events are more dramatic in these countries, (b) the gathering process tends to discriminate the sampling, since events with fewer fatalities are more likely to be reported in some Western countries than in India or China.

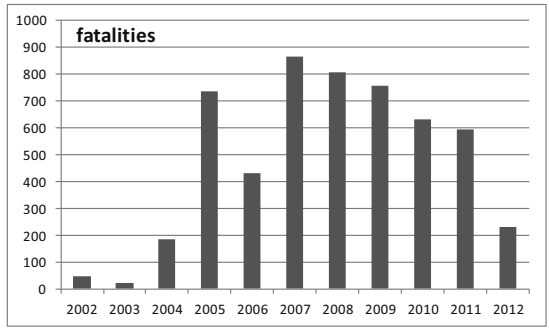


Figure 3. Number of fatalities (including those missing) in the documented cases,

2002-2012 period (2012 until end May)

What could be done with the database?

Descriptive statistics on failure causes

Databases are often used to assess the frequency of causes for failure. This information is documented, even if the quality of technical information available is very low (this is typically the case for press clippings, even in Europe). The database contains 282 cases of bridge failures, of which 212 are failures during service, 62 failures are during construction and 8 are unexpected failures during repair or demolition works. For the first category, Figure 4 details statistics regarding the primary cause of failure, as it can be identified from available data.

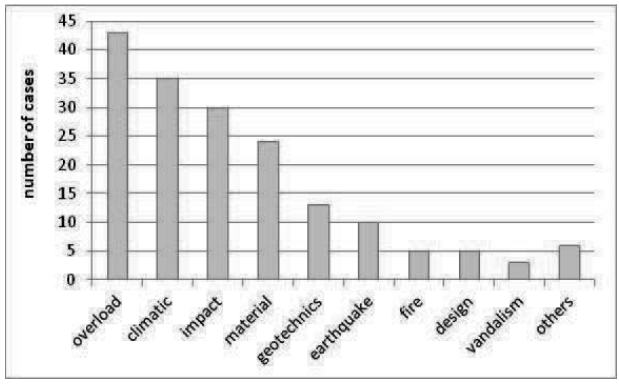


Figure 4. Statistics about the primary causes of failures for bridges in service

The main causes are excessive loading, due to climatic causes (wind, snow and flood), overload, or impact. Material failures also contribute significantly (it covers mainly problems of fatigue and ageing: corrosion, creep, decay...). These conclusions are somehow different from those given by (Briaud and Hunt, 2006) who highlighted the large group of scour failures in data compiled by the Structures Division of NYSDoT. From 1502 documented bridge failures between 1966 and 2005, 58% were due to scour; far behind on the list (less than 10%), were collisions and overload. Once more, this highlights the effect of the sampling process on the statistical results. In any case, further analyses would be welcomed since, in many cases, a failure results from the conjunction of an excessive load and a given material condition. However lack of maintenance or defective design can also contribute to this collapse. In any case, even if statistics are only indicative, the large variety of causes can be noted. This is particularly valuable for teaching.

Feedback on failure cases and acceptable risk

The fatal accident rate (FAR) is often used as a mean of estimating how risks are accepted by a population at risk (PAR). It is calculated by dividing the number of fatalities by the population at risk. The higher the FAR the greater accepted risk. The concept is that if the FAR is not acceptable, developments in attitudes, law and/or regulations, would have been implemented to reduce risks. FAR values for different types of risk or different populations may reveal significant differences in current accepted risk levels. Information from the database can be used to compare FAR values for different geographical areas, like those summarised in Table 3 for the 2002-2011 decade.

Area	Europe	Asia	Africa	N. America	Central and S. America
Population (in millions))	731	4030	965	339	572
Number of fatalities	589	2699	901	101	617
FAR (10^{-6}/y)	0.081	0.067	0.093	0.030	0.057

Table 3. FAR for all types of construction failures, 2002-2011

Because of the sensitivities to the sampling process discussed above there are biases, at this stage of development of the database, in the values for different areas. This can be illustrated with statistics from India, where the National Crime Records Bureau provides yearly statistics about death causes from natural or un-natural causes. The number of fatalities from structural collapse was about 2600-2900 for 2009 and 2010, with a large majority in individual housing or buildings and only few tenths of fatalities in dam and bridge collapses (NCRB, 2011). Thus it is clear that all number corresponding to low intensity events are widely underestimated in the database.

The results are also very sensitive to extreme events. Statistics for 2001 would have taken into account the WTC 9/11 disaster, thus increasing the FAR for Northern America by a factor 3. However, these FAR values are all below 0.1 10^{-6}/y, the number that is currently considered reasonable for structural failure (Breysse, 2009b; Ellingwood, 1999). FAR can also be calculated for different types of structures (buildings, dams, bridges, tunnels) for which it is also interesting to build the F-N curve, where N is the number of fatalities (gravity of the collapse) and F the frequency or probability (Breysse, 2009b). Figure 5 plots the F-N curve for all countries and all types of structures for the 2002-2011 decade. The probability value for each gravity level is calculated by dividing the cumulated number of victims due to events with at least N victims by the number of years (10) and the PAR.

The left part of the curve reaches a plateau below 10^{-7}, which corresponds to about 700 victims each year. This value is clearly an underestimation of the true number, since low gravity events are more sensitive to sampling biases (NCRB statistics cited above lead to a FAR between 2 10^{-6}, and 6 10^{-6}, for the three most risky Indian States). However two points can yet be noted: (a) the shape of the curve, since its slope is related to the risk-aversion (b) the comparison between the various types of structures, which may indicate their relative dangerousness. It would be possible to change the perspective by considering an “exposure factor” since the PAR is not the same for all types of structures. These results, based on the analysis of 236 events which have caused a total of 4660 fatalities must be seen as a very first indication of what could be obtained, if a more systematic collecting process is carried out. Many other issues could be looked at, by making a difference between collapses during construction or during the service phase (the FAR mainly concerns construction workers in the former case and full population in the latter), or by studying specific countries. Such curves can help regulators if they have to compare the real “safety level” between different contexts (different countries or types of structures) or if they want to compare performance (feedback) to safety objectives.

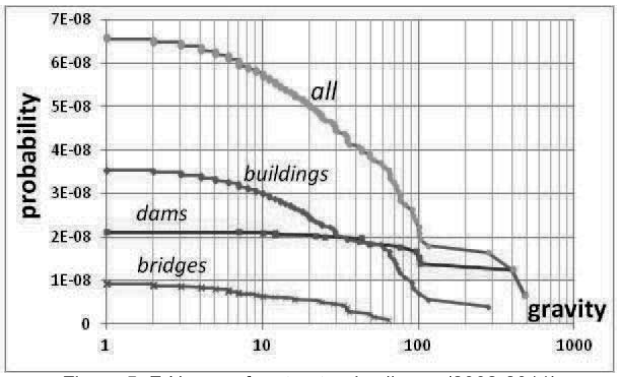


Figure 5. F-N curve for structural collapse (2002-2011)

Failure database and teaching risk

The development of the failure database at a « macro » level for statistical analysis, does not prevent other uses for teaching purposes, as briefly described below:

- Failure cases can be used to illustrate modes of failure and their consequences in representative situations, thus helping to teach to what limit state criteria/phenomena the designer must pay attention. This "learning through failure" is not matter specific and many examples can be taken from the database, either for specific types of construction (reinforced or precast concrete, masonry, steel, timber, earth structures…) or on specific issues (temporary stages, material ageing – corrosion, fatigue, crack propagation, creep… -). It is also interesting to show how in many cases building standards and regulations have been modified after failure events.

- Failure cases can also be used to teach risk management (in civil engineering or other fields of engineering). They often show that risk management is often an issue of communication and organisation. One can refer to the Hatamura's mandala (fig. 2) or to any similar model, to analyse the root causes of failure, the development of failure scenarios, and in trying to understand what countermeasures would be efficient. This is particularly interesting with cases which show that the failure results from a series of interacting causes. For instance the case of New Orleans levees failure during Katrina (Seed et al, 2008; Sills et al, 2008) has been used as a support with students in geotechnics, but many others can be appropriate.

- Other general concepts about risk can be taught. For instance, the database presently documents 21 cases of building collapses (including 3 during construction) in Nigeria between 2005 and 2012, with a total number of 267 fatalities. A more specific analysis of these cases can be undertaken related to the causes of collapses, and related to the response of society (What people at risk say? What media tell? What experts and public authorities do? How regulations change?). Issues such as socio-economical optimisation of risk-response can be discussed in relation to the concept of acceptable risk.

Conclusions and perspectives

We have discussed in this paper how the concern for risk and failure in civil engineering has progressively emerged during the last decade in France. The concept of forensic engineering is now becoming established. There is considerable potential benefit from the development of a failure database, documenting information on failures. A first prototype has been developed, and we have shown how, despite its present limitations (sampling biases, poor quality of information for many cases), such a database can make useful information readily available to experts, engineers, students.

The failure database can be used a micro level, when specific cases are used to illustrate some technical points regarding causes of failures, to improve the design and building practices.

It can also be used at a more global level, for teaching risk in engineering and introducing main concepts, like risk aversion, interaction between risk, sociological and economical dimensions in risk and safety.

Many other avenues for development can be exploited, like the analysis of scenarios for failure, a more rigorous estimation of the structure danger (through F-N curves), the development of an interactive risk register that could be used by engineers, etc…We think that this could be a collective task, at least at a national scale, and examples of other initiatives show that collaborative development through the Internet would probably be a good way forward..

Acknowledgements
The author gratefully acknowledges Pr. J. Wood for his interest in this work and his precious comments.

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Forensic Engineering: Informing the Future with Lessons from the Past
ISBN 978-0-7277-5822-4

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doi: 10.1680/feng.58224.297

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Structural incidents in The Netherlands: a comparison of three databases

Karel Terwel TU Delft, Delft, The Netherlands
Wouter Boot Volker InfraDesign, Woerden, The Netherlands
Mirjam Nelisse TNO, Delft, The Netherlands

Abstract
After several major structural incidents in The Netherlands, various initiatives have been started to improve structural safety. Research studies were initiated on the characteristics, causes and consequences of structural incidents. In this paper the results of three of these research studies are compared. Each study uses different data sources: a confidential reporting system called ABC registration, Dutch arbitration awards on 'structural' failures and a newspaper called Cobouw. It appears that the reliability of the research studies varies due to the quality of both source and analysis. In addition, a comparison of the results shows variations due to the type of sources used. However, general patterns have been derived from the three databases. From this study it can be concluded that contrasting and combining results of different sources has improved the overview of structural incidents and has provided an insight into trends.

Introduction
In the last decade some major structural incidents have occurred in The Netherlands. In 2003 five balconies of a residential building in Maastricht collapsed a few months after completion causing two fatalities (Mans 2010). In 2010 a temporary structure in Rotterdam failed during the casting of concrete resulting in five injuries (Joustra 2012) and in 2011 the roof of a new extension of the FC Twente stadium collapsed during construction resulting in two fatalities and nine injuries (Konijn 2012). These failures, amongst others, drew much media attention and necessitated a response from the Dutch building industry.

The building industry has tried to find answers to questions like: 'To what extent is a lack of structural safety an issue?' 'How can we avoid repetition of similar failures in the future?' Finding answers to these questions starts with the investigations of structural incidents. Usually this kind of research focuses on single incidents or on comparing incidents from a single source. The research question to be answered in this paper is: '*What patterns can be detected from the comparison of various sources of data on structural failures and near misses in the Netherlands?*'

Data sources
Various possible data sources of structural incidents are mentioned in literature such as insurance company files, forensic engineering firm files, newspaper articles, reporting systems of incidents and arbitration awards (Terwel and Waarts 2010). However, only the last three sources could be used for research, as they were publicly available. Most other sources require collaboration with companies that are reticent because of liability issues, bad customer

relationship and fear for loss of corporate image. In the present study a comparative analysis is performed on the results of the three available studies.

A Cobouw database was set up in around 2004 by TNO (Dutch Organization for Applied Scientific Research) based on 230 structural failures that were reported in the Cobouw (Dieteren and Waarts 2009), a leading newspaper for the Dutch building industry. TU Delft developed this into a more extensive database which elaborated the format used by TNO. It currently includes 401 incidents based on Cobouw articles between 1993-2009. The results of this database have only recently been made available.

ABC registration was initiated in 2008 by the Platform on Structural Safety. It is a confidential reporting system of mistakes in structural design, execution, use/maintenance and demolition, which was set up by TNO (Terwel, Nelisse et al. 2012). The essence of this system is similar to CROSS in the UK. Anyone in the building industry can report mistakes through a website. These mistakes are analysed by structural experts, who publish their results in periodic reports and newsletters. Although the last report dates from July 2011, consideration is being given to further reports.

The Arbitration database was developed in 2010 in a master's thesis containing 151 'structural' failures extracted from arbitration awards of Dutch arbitration institutes for construction disputes from 1992-2009 (Boot 2011). Arbitration is a common means of construction dispute resolution in the Netherlands.

The choice of boundary conditions, for instance the definition of failure, varied for the three studies, making a direct comparison of their outcomes somewhat challenging. However, after thorough analysis and minor adjustments of some categories, useful results were obtained. Unfortunately, not all categories could be compared as each source provided different types of data about failures.

Results from incident investigations

Table 1 presents an overview of the process of data gathering and analysis and of the outcomes of the three database studies on incidents.

Table 1: Summary of Main Results

	Cobouw	ABC registration	Dutch arbitration awards
Gathering process of data			
Source of data	(Near) failures collected by using search terms in a digital archive of newspaper Cobouw	Voluntary reports of building participants. Over 80% from local building control officers and structural engineers	Arbitration awards found by using search terms in an online database with arbitration awards
Definition failure case	Every case where the (probability of) failure of a (temporary) structure (potentially) endangers persons	Near misses without damage and failures with damage. A building mistake is defined as an error in design, execution, asset management or maintenance threatening structural safety	Cases with insufficient functional or structural performance. Usually damage has occurred.
Indication of reliability of data	Low	Medium	High

	Cobouw	ABC registration	Dutch arbitration awards
Analysis of data			
How?	Analysis as part of a PhD project	Analysis by professionals from TNO	Analysis as part of a master's thesis
Who?	Researchers from TU Delft	TNO	MSc student from TU Delft
Control by?	Approx. 50% was checked by another person than the one performing the analysis. Comparison was made with results from an earlier database of Cobouw-failures from TNO	TNO. Controlling person was different from analyzing person. Review by steering committee of the Platform on Structural Safety	Review by graduation committee
Publication analysis	Report on preliminary outcomes TNO database	Periodic reports and newsletters. Professional evaluation report. Peer reviewed conference paper	Master's thesis, article in magazine and peer reviewed journal paper
Indication of reliability of analysis	Medium	High	Medium
Indication of reliability of outcomes	Low	Medium	Medium
Outcomes			
Number of incidents	401	189	151
Years of detection failure	<1990-2009	2000-2011	<1990-2009
Type of structures	72% buildings, 8 cases unknown	97% buildings	91% buildings, 2 cases unknown
Function of buildings	38% residential, 2 cases unknown	40% residential	43% residential, 26 cases unknown
Material	28% concrete, 31% steel/metal, 62 cases unknown	24% concrete, 38% reinforcement	26% concrete, over 26% steel/metal, 32 cases unknown
Construction elements	24% facades, 15% floors, 1 case unknown (only buildings regarded)	23% foundations, 21% floors	19% foundations, 19% roofs, 2 cases unknown
Type of damage	51% (partial) collapse, 29% structural damage, 2 cases other	84% no damage	27% (partial) collapse, 33% structural damage, 33% insufficient functionality, 1 case other
Time of discovery	21% construction, 67% use, 2 cases unknown	27% design, 50% construction	25% construction, 67% use, 9 cases unknown
Phase of main cause	15% design phase, 30% construction phase, 23% use phase, 17% combination, 120 cases unknown	61% design phase, 31% construction phase	26% design phase, 30% construction phase, 19% combination, 16 cases unknown
Type of error	16% design error, 43% construction error, 16% combination, 128 cases unknown	65% design error, 35% construction error	34% design error, 33% construction error, 23% combination, 15 cases unknown

Discussion of results

Reliability of the incident investigations

The reliability of the incident investigations depends on the reliability of the data source and of the analysis of the data. Often the researchers had to deal with human errors, for which it is usually very difficult to find truthful answers. Especially in an arbitration parties try to avoid liability.
In addition, usually there are several mistakes needed to cause an incident. If a source identifies only one error as the cause, probably not all errors have been identified. Furthermore, from intensive studies of several major failures, it has been learned that experts can disagree on the main cause of the failure.

The reliability of the data from the arbitration awards is considered to be the highest of the three studies. An arbitration award usually provides detailed information about the structural failure. Arbitrators are impartial and independent structural experts. In contrast, ABC registration is dependent on only one source, although by calling every reporting person confusion about information is reduced. Therefore it has the lowest number of unknowns per category (table 1). For Cobouw there is a large variation in the reliability of the individual cases. In general the reliability of these cases has to be considered low.
The reliability of the analysis is supposed to be the highest for the ABC registration. The analysis is done by a structural engineer, the checking is done by another engineer and a review is done by a steering committee of senior professionals. The reliability of the analysis for arbitration awards and Cobouw is considered to be medium, because the reviewing process seems to be less thorough.
The reliability of the incident investigations can be considered as the lesser of the reliability of the data and the reliability of the analysis process, because a perfect analysis can never compensate for poor data. It appears that the reliability of the investigation of the ABC registration and Dutch arbitration awards are better than the reliability of the investigation of Cobouw cases.

Characteristics of cases

Number of incidents

In figure 1 incidents from all three studies are summarized. 54 of the arbitration awards are not depicted because the moment of occurrence of damage is not known.

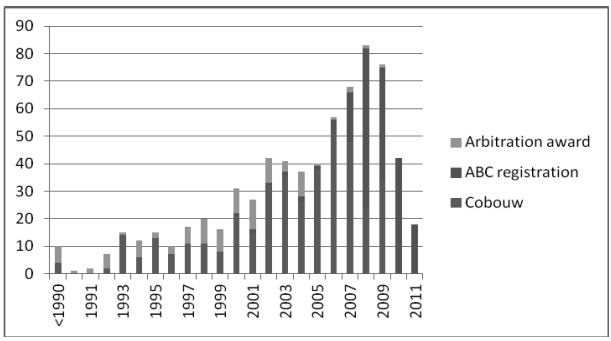


Figure 1: Number of incidents of all sources, depicted for the year of occurrence of damage or, if no damage resulted, for the year of the unsafe act.

Most of the cases from ABC registration did not result in damage and this system was only active from 2007-2011. In contrast, most of the cases from Cobouw and Dutch arbitration awards did lead to damage and the periods covered are similar. Therefore, to detect possible trends in the number of failures only the cases from Cobouw and Dutch arbitration awards are shown in figure 2. The chosen time span is 1993 to 2004, assuming there is an average period of five years between the damage and an arbitration award.

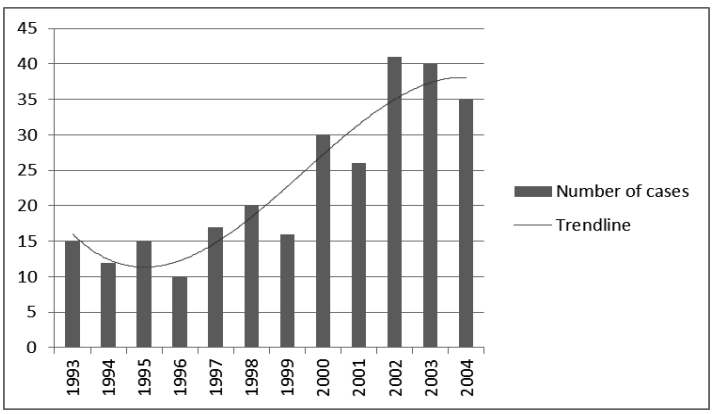


Figure 2: Number of cases Cobouw and Dutch arbitration awards 1993-2004

Figure 2 indicates an increase in the number of failures. This rise in failures over the years might be explained by an increase in media attention after major failures and a growing litigiousness. However, the increase is in line with an estimated increase of failure costs (USP_Marketing_Consultancy 2008). It seems a decline is starting after 2002. However, from the number of Cobouw cases it can be seen that maximum values were reached in 2006 and 2007, after the period presented in figure 2.

Type and function of structures

Figure 3 shows that approximately 85% of all incidents in the databases are related to buildings. In comparison, buildings count for approximately 74% of the yearly turnover in the construction sector (EIB 2012). This indicates that buildings seem slightly more vulnerable to failures than civil structures.

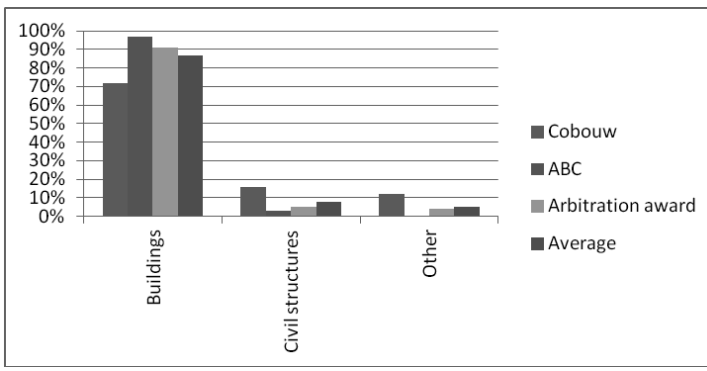


Figure 3: Type of structures

Figure 4 shows that approximately 40% of the cases have a residential function. About 55-60% of all buildings in The Netherlands are residential buildings. It is possible that, because there are many small houses, structural failures of individual houses are usually not worthwhile mentioning in newspapers or the damage costs are too small to start an arbitration procedure. Nevertheless, it seems reasonable to draw the conclusion that residential buildings suffer less often from structural failures.

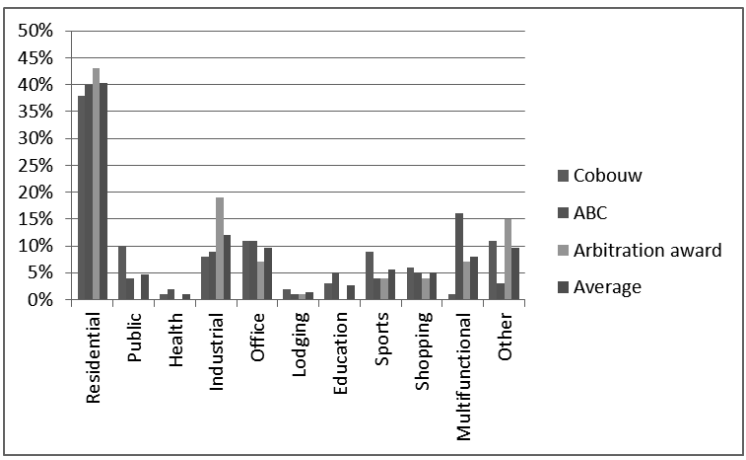


Figure 4: Function of buildings

Materials

Figure 5 shows that the frequency of failure cases with concrete and steel/metal is similar. It seems that steel structures are more prone to errors, because concrete structures are more common than steel structures in The Netherlands. However, reliable data on the exact number of steel and concrete structures is not available.

Unlike the other researches, the ABC registration recorded relatively more problems with reinforcement. It is possible that local building control, who reported nearly 70% of the cases, more often focuses on reinforcement deviations.

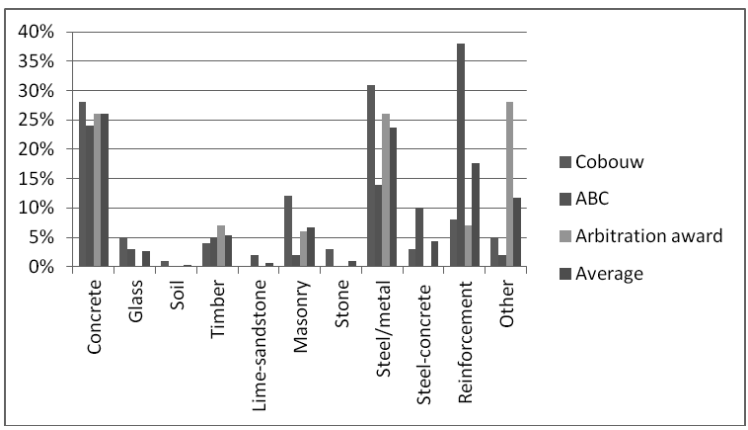


Figure 5: Materials involved

Construction elements

Figure 6 indicates that horizontal elements like floors and beams are more vulnerable to failures than vertical elements like columns. Failures of foundations are also common, which is to be expected because of soft soils and erratic soil profiles in The Netherlands.

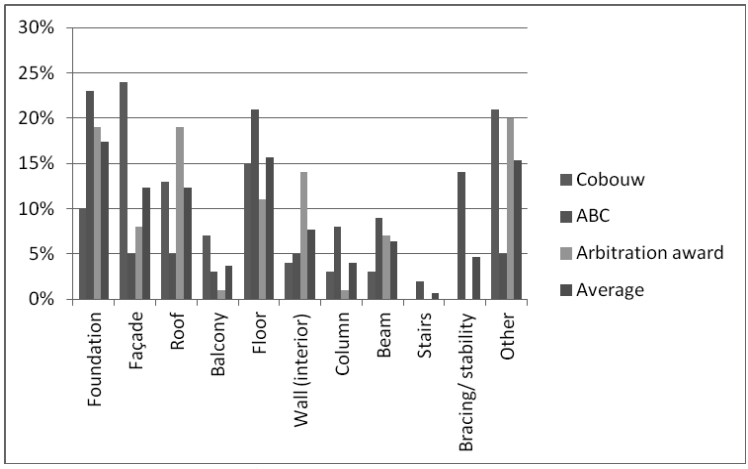


Figure 6: Construction elements

Type of damage

Figure 7 shows that Cobouw more often reports cases with partial/full collapse than the other sources. This can be explained, because these incidents attract the most attention. On the contrary, ABC registration usually reports incidents without damage. This is a consequence of the large number of design error reports (see figure 9).

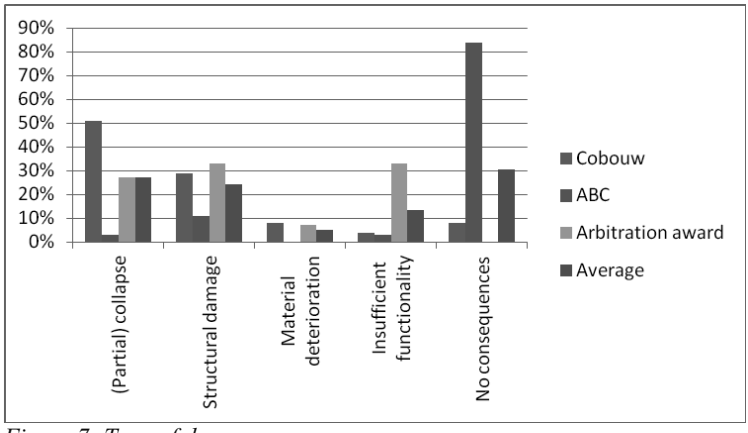


Figure 7: Type of damage

Time of discovery

Figure 8 depicts that incidents from ABC registration are discovered in an earlier phase than cases in Cobouw and Dutch arbitration awards. This follows from the nature of each database; there is little or no sense in media attention or arbitration of errors detected in the design phase, because no damage has occurred yet. Results from Cobouw and arbitration awards show a noticeable resemblance.

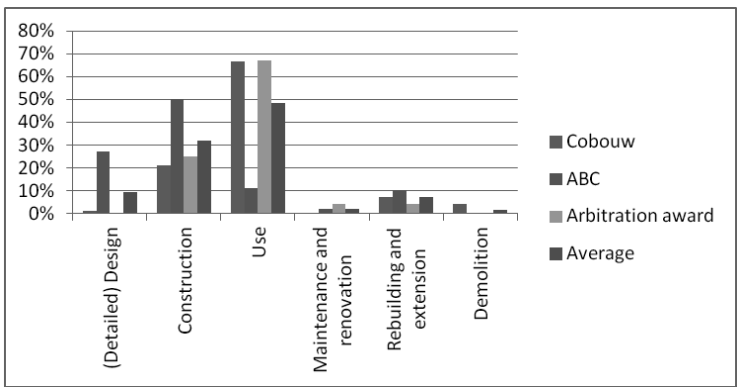


Figure 8: Time of discovery

Causes

Phase with main cause

A remarkably large range of outcomes can be observed in figure 9 between the three researches, especially for the design and use phase. Because most of the mistakes within ABC registration are already discovered in the design phase, it is to be expected that the cause is more frequently found in the design phase.

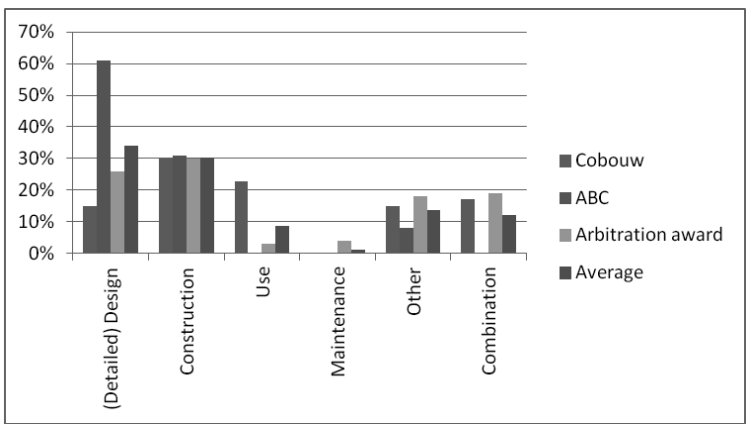


Figure 9: Phase with main causes

Type of errors

Figure 10 highlights that the majority of failures are caused by a human error. Only a minority are due to material deficits or force majeure. Design errors might range from conceptual, modeling, calculation and drawing errors to conflicts between calculation and drawing or even the absence of a drawing or calculation. Construction errors might range from the choice of wrong materials or erroneous assemblage of elements to the application of an insufficient amount of material or erroneous sizing. Use errors usually have to do with overloading or sometimes with lack of inspection and maintenance.

The relative number of errors classified as design errors is similar to the number of cases originating in the design phase. Especially in the Cobouw database a larger number of construction/production errors are found than the number of cases originating in the construction phase, because errors during rebuilding or demolition (category 'other') are more often construction/production errors than design errors. Failures originating in the use phase are relatively often ascribed to force majeure, for instance when the current traffic load is higher than could be expected during design of a bridge.

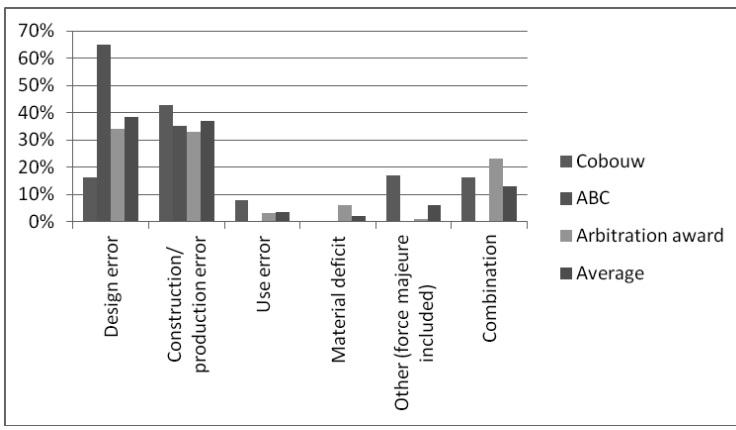


Figure 10: Type of errors

Other contributing factors

All three studies mention underlying factors, like the complexity of the design, number of building participants in a project, absence of warnings, role of changes, time pressure, lack of budget, underdeveloped safety culture, unclear responsibilities, insufficient communication, lack of coordination and control, inadequate codes, the quality of the engineers and workmen, and working conditions. However, comparison of these factors is difficult, because the researches did not focus on the same aspects, and often insufficient information on these aspects was available.

Nevertheless, the authors would like to highlight some significant outcomes. First, from the Cobouw research it was concluded that only about 15% of the cases could be classified as an unusual design. From this it can be concluded that complexity of design is not an essential requirement for failure. Second, from Cobouw and arbitration award research it appeared that in various cases changes were made in design or construction phase. Without these changes, the failure would not have manifested itself. For 19% of the arbitration awards changes influenced the initiation of the failure. Third, from Cobouw and arbitration award research it is known that in many cases prior warnings were given by persons, after control or inspection,

or by the structure itself, resulting in cracks or exceptional deformations. In the Cobouw database at least 168 cases were found, where physical signs could be observed before damage or failure occurred to the full extent. Therefore, appropriate response to warnings needs more attention.

Conclusions

The type of source used for investigating structural failures can bias the results for one or more characteristics of that failure. Arbitration awards seem the least prone to this. ABC registration finds its strength in the thorough analysis. Therefore, it can be concluded that the outcomes of ABC registration and Dutch arbitration awards are considered more reliable than the outcomes of newspaper articles. However, because newspapers provide a significantly larger number of cases, they can be complementary and can give a valuable insight in trends.

Despite these issues results show resemblances and therefore some general conclusions can be drawn. Buildings are involved in approximately 85% of all reported incidents. Horizontal construction elements are more vulnerable to failures than vertical ones. Steel structures tend to be more prone to errors than concrete structures, although this statement should be supported by additional research. It appears that the cause of origin of structural failure is approximately 35% in the design phase, approximately 30% in the construction phase and less often in the use phase (approximately 8%). A reasonable number of cases have a combination of design and construction errors. Other remarkable outcomes include the influence of changes in design or construction on failures and presence of warnings given by people or the structure itself. This indicates an opportunity to reduce failure risks by better procedures when there are changes and proper response to warning indications.

It has been shown that combining results of various sources of structural failures gives a more balanced view of the characteristics of structural incidents than a single source. This comparison study has provided an insight into the causes of failures. Hopefully, this awareness will lead to a safer building industry in The Netherlands.

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SECTION 7:
LEGAL, CONTRACT AND LEARNING

Forensic Engineering: Informing the Future with Lessons from the Past
ISBN 978-0-7277-5822-4

ICE Publishing: All rights reserved
doi: 10.1680/feng.58224.309

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Systemic analysis of risks in project management and safety issues

Denys Breysse, University Bordeaux 1, I2M, CNRS UMR 5295, France
Rasool Mehdizadeh, Franck Taillandier, Esra Tepeli (same affiliation)

Abstract
Structural collapse is the ultimate and most spectacular manifestation of construction project failure. More generally, the project objectives are not only structural stability and safety, but the fulfillment of cost, delays and performances. The authors have developed a systemic approach of construction projects in which risk events are highlighted. The first aim is to support the risk management, by identifying risk events at all stages of a project, in order to define efficient strategies and countermeasures.
The first task was to establish a methodology for building relevant Risk Breakdown Structures (RBS) which are a very efficient way of representing project risks. An in-depth review of existing practices lead to develop a set of three interconnected databases (risk categories, risk events and micro-trees) such as to ensure consistency of knowledge modeling on project risks and efficiency of risk assessment.
It will thus be explained how risk scores (qualitative assessment) or risk values (quantitative assessment) can be estimated at any step of a given project. This assessment can cover all dimensions of a project, namely cost, delay and performances. It can also have a subjective part, since different stakeholders can have a different view on a given situation.
The last part of the paper will explain how the analysis of some structural and catastrophic failures may be used in order to reinforce and iteratively develop the risk knowledge base. Examples of tunneling and temporary structures will be discussed.

Keywords: failure analysis, project risk, risk assessment, risk management

Introduction
A project is usually considered successful if it is completed on time, within budget and to the specified quality standards (Chan and Kumaraswamy, 1997). The major factors causing delays on construction projects have been reviewed and critically appraised in many scientific journals and reports (Luu et al, 2009). The construction project management involves numerous participants: end users, promoters, construction companies, project designers, government/public bodies as well as a large number of sub-contractors, suppliers and other entities (Pérez et al., 2010) whose interests and demands need to be considered in the managerial decision-making, to ensure the success of the project (Aaltonen, 2010).
Construction risks are mainly handled with experience, assumption and human judgment (Baloi and Price, 2003). Lack of an industry accepted model of risk analysis forces every construction company to develop its own risk management models (Hartman, 2000; Baloi and Price, 2003). Other problems relate to possibly biased decision making, when personal background and assumptions inevitably reflect on the person's evaluation (Klemetti, 2006). The improvement of practices is mainly based on the diffusion of guidelines for good practices, which describe what all project stakeholders must do, at each stage of the project, for a better PRM.
Construction risk management has also been a field for research for more than 20 years, either regarding its very general aspects (Tah and Carr, 2001) or pointing at specific issues, like the type of works (Rozek and Loganathan, 2009), the nature of contract (Lam el al, 2007) or a specific country (Wang and Chou, 2003, Zou et al, 2007).
The contrast remains whose purpose is to analyse risks in-depth, trying to qualify/quantify them at any stage of the project. It will be shown in this article how this gap can be filled, by developing modeling tools, adapted to the real project context and to the need of driving factors.

Furthermore, PRM is a scalable activity and should be commensurate with the size and complexity of the project under consideration. As result, the project risk management process has to be tailored for each particular case and project (Mehdizadeh et al, 2010).

Risk Knowledge Database (RKD) and Risk Breakdown Structures (RBS) development

Principles

The Risk Breakdown Structure (RBS) is a hierarchically organized depiction of the identified project risks arranged by risk category and subcategory that identifies the various areas and causes of potential risks (Tah and Carr, 2001; Gidel and Zonghero, 2006; PMI, 2008). Its use has been promoted by (Hillson et al, 2006) who advocated to combine it with a parallel decomposition of works (WBS). RBS offers a synthetic view on risks and it is compatible with evolutionary and dynamic nature of project risks. Furthermore, at a given stage of the project, each stakeholder can choose a different description, i.e. a different RBS, that would be the best adapted to his proper concern, e.g. delays, or to the risks he is in charge of (responsibility). However, there is a lack of consensus on how to develop an RBS for a new project, and there is no clear definition on how to define risk categories or on how transfer qualitative/quantitative information about risks across the tree (Mehdizadeh et al., 2012b).

An innovated method was developed by Mehdizadeh et al. (2012c) in order to design tailored RBS' which are scalable, adaptable to the project development and allow multi-views from each of the stakeholders. The main idea is that each RBS may be viewed as a set of many micro-trees, i.e. 2-level elementary structures which link a single "mother" category to two or several "daughter" categories. Any category may be split into sub-categories in various ways, thus resulting in a quasi-infinite number of possible trees. How a given tree fits the user purposes will be discussed below. We focus first on how the risk knowledge is modeled into risk events, risk categories and micro-trees.

Three interconnected risk knowledge databases (RKD)

Risk is defined, in agreement with 2009), as ISO (the consequence of uncertainty on the objectives of the project. Thus, risk has sources (risk events), which may be grouped into categories and which have possible consequences on the project objectives (namely cost, delay, performances). These consequences may be direct and/or indirect, by generating other risk events. This last point is a fundamental problem, since the interdependency between risks often remains an open issue which may prevent an effective risk assessment and management. Our choice was to decide that PRM must be performed dynamically. Thus, at several well defined steps during the project development, risks must be reviewed and assessed. It is there that, because some risk events have occurred while other have not or have been managed, that the risk review must be updated, and interdependency accounted for.

One main challenge is the consistency of risk knowledge modeling. A specific attention was paid to this issue, based on a wide scientific and technical literature review and a careful analysis of the information contained. Some possible problems are: (a) no clear definition of an event or of a risk category that may induce errors or uncertainties on the "mother-daughter" link, (b) no rigorous split of a category in mutually exclusive subcategories: in a micro-tree, subcategories must cover the same field that the mother category, without any overlap. These problems can only be solved by a careful definition of the events and categories and by checking that the micro-trees are convenient.

Risk event database

Risk Event (RE) is considered as any fact or event whose occurrence can have some impact/consequence on at least one of the project objectives (time, final cost and performance of the project). An event can also consist of something not happening. In the database, each RE is identified by a code (RE#i) which is an integer and unique number, a unique descriptive definition, probability, and three values for its impact on project objectives (cost, schedule, performance).
Exhaustiveness of risk event description cannot be a target, since there is a wide variety of events, making them uncountable. In fact the number of possible risk events increases with the level of detail of the project description.

Risk category database

A Risk Category is defined as a group of several REs. Each RC has a code (RC#i) which is a unique integer number, “name” and “description”. Once the tree will be built, the information attached to the events (probability and impact) will be transferred to the categories that cover this event. The transfer rules will be detailed below.

As for risk events, there is a undefinite number of possibilities for grouping risk events in risk categories. Risk classification into categories may correspond to similarity in origin, in type, in who bears the risk in charge, in the project phase during which it can occur, etc…

Micro-tree database

There is no more theoretical limit in the number of ways a category can be subdivided into subcategories. For instance, at the upper level of “Project Risk”, this global category can be subdivided following various logics: (a) according to project phases, (b) according to project stakeholders, (c) between category of external risks and category of internal risks, etc. A single rule must drive all definitions of Micro Trees in the database: that of a mathematical partition. Any Micro Tree defines a way of splitting a category X seen as a set of risk events) in a series of subcategories (each of them being a subset of risk event) in a way such that they correspond to non-overlapping and non-empty parts that cover all of X. These parts are both collectively exhaustive and mutually exclusive with respect to the set being partitioned.

Consistency and adaptability

Once risk events and risk categories are defined in such a way that their meaning offers no possibility of misunderstanding, the partition rule driving the definition of Micro Trees suffices to ensure the consistency of the database. The database is itself adaptable and can be easily modified, either to extend the topic of interest, or to go further into details in a given field. If one wants to develop more the database (for instance for a more detailed description of tunneling projects), it is necessary to: (a) identify and define specific risk events, (b) specify the belonging relationships between risk categories and these risk events, checking the partition rule for all Micro Trees for which the mother category contains the risk event. A computer program has been developed that provides an initial version of the database and that helps the user for its development, by automatically checking the consistency requirements and providing explicit messages if there is any inconsistency.

Semi-automatic generation of RBS

The utility of RBS is twofold. On one hand, the RBS is a tool for risk representation: it is way to show how the project is concerned with risk. Depending on the choice of the RBS, it will tell where are the sources of risks, who may be in charge of them, at what time they may happen… being thus a very practical tool for discussion and risk management. On the other hand, RBS also makes possible the risk assessment, on the basis of information provided at the basic level of Risk Events. These two functions are complementary. One main challenge is the validation of a process which enables the valuation of RBS, i.e. which is able to select a representation of risk adapted to the context. To do that, a 4-step method is developed:

(a) The first step is the specification and valuation of Risk Events: the user must tell what RE belonging to the database must be pointed at a given stage of his project, and he must indicate the corresponding magnitude of risk;
(b) The second step consists in the automatic building of all possible homogeneous RBS (homogeneous means that all tree branches end at the same level, i.e. at the same distance from the top of the treetop. This can be done easily by the computer once all REs have been specified, since all relationships between RE, RC and MT are in the RKD;
(c) The third step consists in ranking each of these RBS, according to a series of criteria that will be described in the following;
(d) The last step consists in selecting the best one among these many RBS, according to a multi-criteria selection process and to give back the possibility to the user for improving this RBS. This improvement will consist in developing and/or cutting some of the branches in order to be more adapted to the pattern of REs.

The most challenging issue is the definition of a series of criteria through which the “quality” of a RBS will be evaluated. To the best of our knowledge, this had never been done before and our contribution was there to formalize what can be seen as simple common sense. Three criteria have been defined:

(a) A convenience criteria, corresponding to the most adapted level of detail. The basic idea is to favor trees in which each bottom category (i.e. category which is at the end of a tree branch)

contains a number of REs which is more or less identical and close to a target value, provided by the user. By changing this value, the user will develop more or less the RBS into detail, keeping a well-balanced representation between its various branches. This criterion is valuated thanks to a note N_{conv}.

(b) An adequacy criteria, corresponding to the fact that a RBS may be more or less adequate for representing the risk pattern according to general requirements. Three such requirements have been defined. They respectively correspond to the need of highlighting: (aa) project phases, (bb) a given stakeholder (e.g. the user himself!) or (cc) the various dimensions of the project performances (cost, delay, quality). The idea is that some MTs are best fitted for making the part between project phases while others will be best fitted for another requirement. A set of three notes (N_{phases}, $N_{stakeholder}$, $N_{objective}$) can be calculated for each RBS by combining the notes attributed to each MT that it contains. These basis adequacy notes have default values in the RKD, which can be modified by the user. If new MT are added to the database, the user must also enter the corresponding notes.

(c) A contrast criterion, corresponding to the fact that the more the RBS highlights contrast between high risk RCs and low risk RCs at a given level the better. A synthetic note $N_{contrast}$ is calculated by weighting elementary notes calculated at all levels.

The determination of the most adapted RBS is done following a multi-criteria decision process in which the five notes (N_{conv}, N_{phases}, $N_{stakeholder}$, $N_{objective}$, $N_{contrast}$) are weighted according to weighting factors chosen by the user.

RBS and risk assessment

General methodology of qualitative and quantitative assessment

Once a RBS is built, the second main issue is how risk values can be propagated throughout its branches? To take advantage of the tree structure of RBS, the approach is based on aggregation (Taillandier *et al.*, 2011). Risk is first assessed at RE scale and then aggregated so as to calculate risk of RC. One main challenge is to handle in a consistent way quantitative assessment and qualitative assessment. This constraint is due to the fact that when some info may be easily quantified (cost, delays), others are not exactly known or are even very difficult to assess precisely (quality). In order to enable a real multi-criteria analysis of risks, it thus seemed important to keep both possibilities of quantitative and qualitative descriptions at all levels of the tree. The word "*score*" is used for qualitative assessment (it corresponds to a discrete scale) while "*evaluation*" corresponds to quantitative assessment. The final result (global risk) will be delivered on a qualitative scale which makes understanding easier.

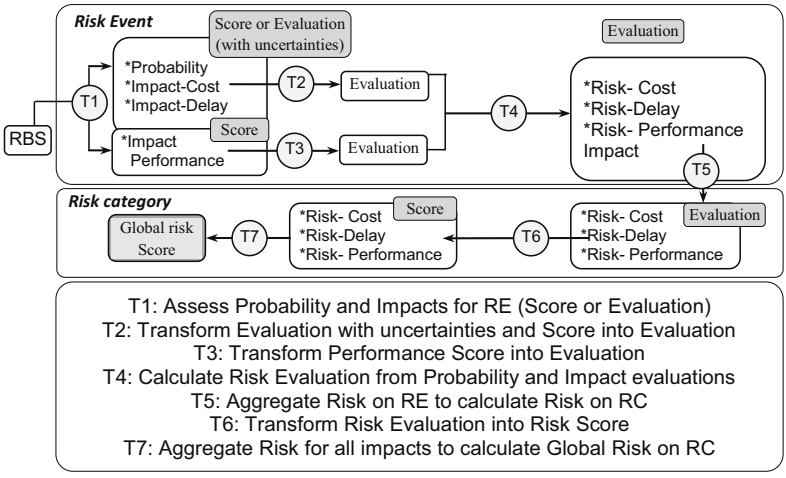


Figure 1. Risk assessment process

Figure 1 illustrates the seven steps of the full evaluation process (T1 to T7), which will be detailed in the following. The interested reader can refer to (Mehdizadeh et al, 2012b) for more comprehensive explanations.

Two scales are used in parallel: a continuous cardinal scale specific to probability, cost and delay impact and a discrete ordinal scale common to the four characteristics. The discrete scale is ranging from 1 to 5 (5 indicating the situation leading to the greatest risk). The impact on performance cannot be expressed on a continuous scale and will be inevitably assessed on a discrete scale. Each scale has its own advantage. Continuous scale is closer to physical reality and has a more concrete meaning. Discrete scale has a strong symbolic value. It allows via a color system a fast and intuitive vision of the situation.

The main idea is that risk is first assessed at Risk Event scale. Risk components (probability and impact) are assessed (T1-T2-T3), by considering the three dimensions of impact on cost, delay and performance. These risk values are all calculated on a quantitative scale (T4) and transferred to categories throughout the branches (T5), keeping the three dimensions independent. The last step (T6) consists in going back to a qualitative scale (score) which makes possible, at the level of each Risk Category, to provide a unique Global Risk Score that merges the three dimensions of impact (T7), notwithstanding their very different units.

In order to define the equivalence between evaluation and score, a qualification grid is built, which expresses scores bounds. Table 1 provides an example of such a grid. It should be noted that the values and ranges in this table are indicative and can be changed regarding the stakes, strategies and size of the project. It is mandatory that all members of the project team spend some effort for its construction. This grid, which puts in relation the two systems of assessment may be used in order to transform score into evaluation (T3) or reversely (T6).

Score	Probability		Impact				
			Cost (k€)		Delay (day)		Performance
		0		0		0	
1	*Very unlikely*		*Very low*		*Very small*		*Very few affected*
		1/100		1		5	
2	*Unlikely*		*Low*		*Small*		*Few affected*
		1/30		10		10	
3	*Few likely*		*Moderate*		*moderate*		*Affected*
		1/5		100		40	
4	*Likely*		*High*		*Large*		*Strongly Affected*
		1/2		1000		100	
5	*Very likely*		*Very high*		*Very large*		*Very strongly affected*
		1		10000		500	

Table 1. Qualification grid with two scales: score and evaluation (indicative values).

Some further refinements have been included in the assessment process in order to account for: (a) uncertainty on quantitative data, (b) aversion to uncertainty (Mehdizadeh et al, 2012b). To compute the risk values for each RC, the choice is to proceed to a simple summation of risk values for all REs that belong to this RC. Other possibilities could be tested, but this simple rule has the great advantage of making the risk value for the RC independent on how the RC is subdivided in the Micro-Tree. From a theoretical point of view, summation is a valid option when the REs considered are uncorrelated. Another problem arises with the summation of scores, which cannot be directly summed. They are first transformed into evaluations thanks to a grid similar to that of Table 1; thus these evaluations are summed and finally transformed back into a score.

Multi-criteria assessment

During the first six steps (T1 to T6) the risk assessment is carried out separately for the three dimensions of consequences. The combination of these three dimensions is only performed at step T7 following a multi-criteria statement. The global risk score is defined by aggregating scores of risk on the different objectives using ELECTRE TRI-b method (Yu, 1992; Almeida-Dias *et al*., 2010). Global risk score is defined on the same scale as for risk scores for each objective (ordinal scale from 1 to 5).

Each RC is represented by a score vector (SV), composed by a set of three risk scores in cost, delay and performance impact. By comparing this vector with bound vectors (BV), a score is assigned. Bound vectors represent the limits between categories (each global score). Figure 2 illustrates the principle of the method. The BV is assigned to the highest category (highest score) for which it is not outranked by any BV. The outrank relation meaning is "at least as good as."

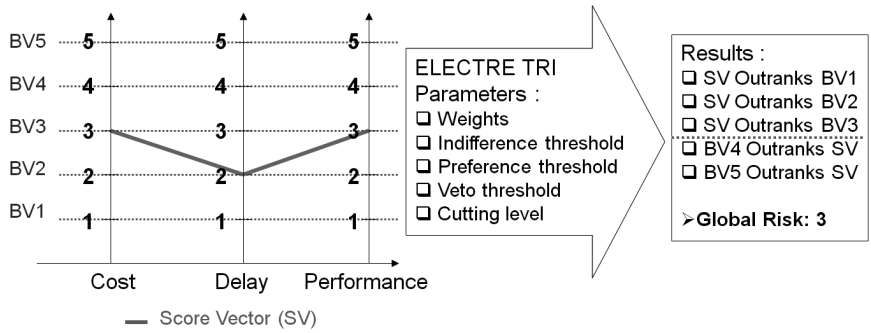


Figure 2. ELECTRE TRI Principle

The outrank relation is determined by the calculation of *concordance* and *discordance* indices. Concordance *cj(A, B)* reflects the level of certainty for the assertion that the score vector A is *at least as good as* the score vector B for criterion j. Discordance *dj(A, B)* reflects the level of certainty for the assertion that the score vector A *is not as good as* the score vector B for criterion j. These two indicators take values between 0 (no concordance or discordance) and 1 (perfect concordance or discordance). They are calculated for each pair (SV, BVi) and (BVi, SV). From these two indicators, credibility degree, that expresses the degree to which the vector of notes A is at least as good as the vector B, is calculated for each pair (SV, BVi) and (BVi, SV). By comparing this degree with the cutting level (parameter to be fixed), a ranking is established between SV and each BVi and then leads to the final assignment (and so the global score). The ELECTRE TRI-b method requires the definition of several parameters which have an effect on the decision process: weight of each criterion, indifference threshold, preference threshold, veto threshold and cutting level. The reader can refer to (Roy and Bouyssou, 1993) for more details on this methodology.

Integrated software for risk management

A specific computer program has been developed which carries out all operations described in this section and at Figure 1 and which is interconnected with the three Risk Knowledge Databases defined at the previous section (Mehdizadeh et al., 2012b). It has a user-friendly interface, so as to enable the user to follow the logic of RBS building and risk assessment. Among the options offered by the computer program, there is the possibility of updating the databases and that of prescribing the values of parameters using a Setting tab (figure 3).

These parameters include all information required in order to generate the RBSs, choose the best one, and calculate risk values. Among the items required, the user must indicate:

- parameters telling if the Micro-trees (thus the RBS) must be assessed from a specific point of view (see definition of adequacy index): that of a given stakeholder (if yes specifying which one) or that of the project itself, that of a specific phase, or that of a specific objective (either cost, or delay or performance, or all of them);
- what is the top level Risk Category (i.e. the main root of the trees). The default value is "Project Risk", i.e. the RC that covers the whole field of risks, but a more restricted field can be chosen if the user wants to focus on a specific domain within the project;
- a parameter (named A_v) which is directly linked to the target level of detail of the RBSs: the smaller A_v, the lower will be the number of REs in bottom RCs and the more detailed will be the RBS,

– weighting parameters for objectives indicating what is the relative importance of each of the three objectives (time, performance, final cost) that are used in the multi-criteria decision process.

Additional parameters are needed: (a) to weight the five notes which enable to assess the value of each RBS and (b) to weight the relative importance of each level when a note is built on the whole tree from notes at each level.

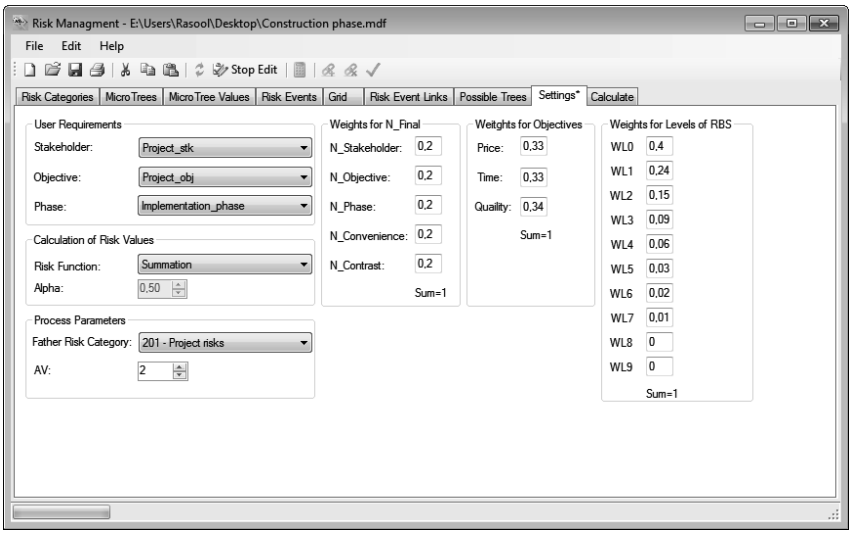


Figure 3. Screen copy of the Setting tab

Once all calculations have been performed, the computer program can display the results in a user-friendly way. Figure 4 illustrates that point by indicating the selected 3-level RBS and assessing risk at all scales. The risk value is visualized for Global Risk Score by a color scale directly on the name of the Risk Category, and the corresponding values have been added (noted a/b/c here for the respective scores corresponding to cost, delay and performance).

All other relevant information, like the value of the RBS itself is easily accessible and the user can refine his analysis by extending or cutting some branches of the RBS. Of course, all these operations can be repeated in order to monitor risk all along the project development. Each time some basic information is modified, for instance that attached to Risk Events (probability or impact), the analysis can be updated. This makes possible to use the developed tool in order to analyze the efficiency of possible risk management strategies on the risk profile of the project.

Another interest of the method and of the tool developed is that they make possible and easy the representation of risks according to different perspectives. Taking into considerations these different perspectives, for instance that of the contractor and that of the client, only requires to specify different point of views in the setting tab and will lead to different pictures for a same set of objective information (probability and impact of Risk Events). Since accounting for the subjective dimension of risk perception has been considered as a key issue in efficient risk management, confronting these two pictures and enforcing the discussion around them is doubtless a source of progress.

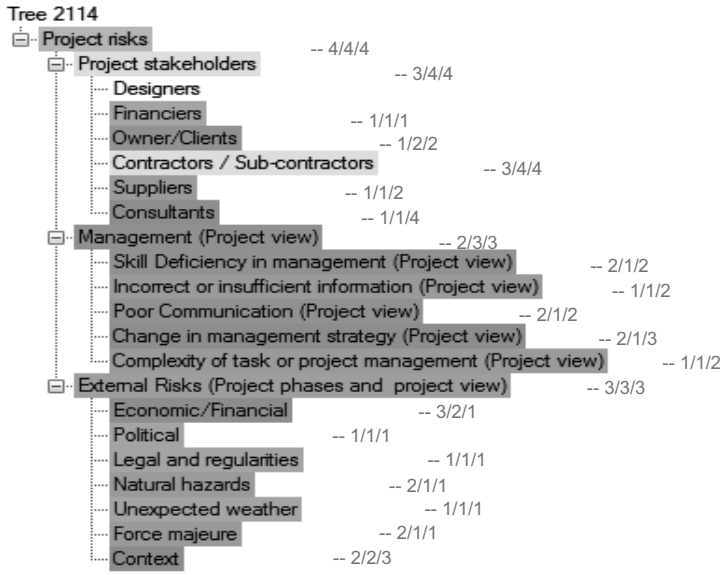


Figure 4. Some of the results displayed for analysis: RBS and risk values.

Failure analysis and RKD improvement

Forensic engineering as an help for risk review and analysis

With the methodology described in previous sections, the reliability of the risk assessment is highly dependent on the reliability of the basic information, i.e. that provided by the user or, in a wider sense, by the project team, on risk events. It is well known that the "weakest link" in risk management is usually not the risk evaluation itself. Even if many methods may be used for assessing qualitatively and quantitatively risks, they all lead to more or less identical results. In fact the "weakest link" in risk assessment is probably that some significant Risk Events may simply be forgotten and not accounted for in the analysis.

One interest of forensic engineering is that it can be used at different scales in different ways (Breysse, 2012). The detailed analysis of a failure/collapse case can reveal some interesting facts and point at risk sources which have to be accounted for in similar contexts. Thus case studies can feed the Risk Event database and improve its quality. Forensic analysis can also provide useful information on the correlation between risks: how a source event can, in combination with a second event, lead to a third one, etc… The main difficulty is to draw general conclusions from singular events.

Failure can also be analyzed at a more global scale, analyzing data sets of failure cases stored in a database. In theory, such analysis could lead to a quantification of risk events (what is the probability of such risk event in such context?), but many difficulties arise, like the degree of representativeness of the set, and the very large variety of events and contexts which makes the statistical analysis difficult. At least, it is probably possible to use failure cases in order to define series of Risk Categories and Risk Events which are more or less relevant in a given context. Providing examples of collapse may also be helpful in reminding the project stakeholders (client, designers, contractor…) that risk sources are there and may have important consequences. Regarding statistics, it would also be theoretically possible to proceed to a statistical analysis of scenarios (source event/consequence) which could have some consequences on the project. Formalizing such scenarios, in order to identify patterns (and later quantifying the possibilities?) was for instance the objective of the Hatamura project (2007). Hatamura built a innovative representation of risk sources (called mandalas) that in fact correspond to a 5-level RBS, where the risk categories correspond to possible individual or collective

attitude / behavior / expertise. To the knowledge of the authors this interesting work would merit further development since it may provide very interesting input for risk analysis and assessment.

Analysis of temporary structures failure and RKD improvement

Breysse et Elachachi (2012) have analyzed two French catastrophic cases of failure of truss structures, that of the Bastia-Furiani temporary stand, which lead to 18 fatalities and 700 injured people on 1992, May 5th and that of the Queen Mary-Saint Nazaire footbridge, that caused the death of 16 people on 2003; Nov. 15th. The comparative analysis revealed that common elements characterize these two events, among which: pressure of the context (either for economic reasons or for prestige), lack of design of the structure, lack of regulations for such structures, lack of control of the built-in structure, lack of experts that would endorse a decision, overloading of the structure, lack of bracings preventing global instability…These elements, as often in failure cases, combine structural and mechanical sources and organizational ones. This shows how simple case studies may help in identifying possible Risk Events (or Risk Factors which could be defined as the precursor of these events). From such an analysis, it is therefore easy to update (if necessary) the Risk Knowledge Database. A further task can be to develop a series of Micro-Trees with the corresponding Risk Categories and Risk Events that could be more specific of temporary structures or of temporary phases in construction works. Without doubt, problems like that of the stability of formworks, the lack of bracings, the absence of design of such structures… etc, would be confirmed as common risk sources in such cases.

Analysis of tunneling projects failure and RKD improvement

Another domain in which many failures have occurred during temporary stages is that of urban tunneling (Breysse, 2010). Even if the number of catastrophic events is probably (and hopefully) too low to enable the establishment of statistics, the in-depth analysis of some well detailed cases on one hand, and the comparative study of a series of cases on another hand are two possible ways of progress. The first one can help in identifying risk sources and failure scenarios, which can be reminded to the project team confronted to a similar context. The second one can provide qualitative (or semi-quantitative) information about the frequency and magnitude of these events and their impacts. Once more, the analysis of a dataset may be helpful in establishing patterns that could be used for future projects. It is on the basis of such cases that we were able to update the Risk Knowledge Database in order to add a series of RE, RC and MT for tunneling projects (Mehdizadeh, 2012d).

Conclusions and perspectives

We have detailed the main characteristics of a methodology whose aim is to develop Risk Breakdown Structures adapted to Project Risk Management. This methodology is based on: (a) the development of a set of three interrelated and consistent databases (Risk Events, Risk Categories and Micro-Trees), (b) the building process of RBS from Micro-Trees, (c) the definition of a series of parameters and criteria which make possible both the quality assessment of the RBSs and the risk assessment. A dedicated computer program has been developed which carries out all operations and, through a user-friendly interface makes possible the risk assessment from its very initial stages.

One main characteristic of the methodology is that it has been designed so as to be user-specific, since the user will provide not only information specific to his project, but also on how he wants to look at risks, and on what he wants to focus on. The methodology has also been designed so as to make easy the risk management throughout the project life, with a dynamic updating of information and a more or less detailed level of description according to the quantity/quality of information available at a given stage. Finally, it makes also possible to compare the different views that different stakeholders may have on the project.

This work opens many perspectives. One way of development is the continuous updating of the Risk Knowledge Databases by a more systemic use of failure cases, which will provide new information, and whose more formal analysis may be fruitful. The identification of reproducible failure patterns, that of correlation between risk sources, or the quantification of risk sources according to the context would be precious. Another way will be a more fundamental work on risk ontology, which will probably lead to a redefinition of the content of the three databases (RE, RC and MT) whose components have been defined and named empirically. This work will also probably require a work in parallel on failure cases since the ontology between the two domains (Project Risk Management and Forensic Engineering) will require a consistent approach.

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Forensic Engineering: Informing the Future with Lessons from the Past
ISBN 978-0-7277-5822-4

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doi: 10.1680/feng.58224.320

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All the World's a Stage - Reflections on a Forensic Engineering Moot Court Experience.

Declan T. Phillips, Department of Civil Engineering & Materials Science, University of Limerick, Limerick, Ireland.

Introduction

Society's intrigue with failures and particularly engineering failures never wanes; perhaps this has provided the motivation for numerous books on the topic, for example; Carper and Feld, 1997; Chiles, 2001; Delatte, 2009; Levy. & Salvadori, 1992; Shepherd and Frost, 1995; Ross, 1984, Wearne, 1999 and many more. Embracing the fascination, this paper tells the story of an initiative between the Department of Civil Engineering and Materials Science and the School of Law at the University of Limerick (UL), Ireland. The collaboration, now in its third year, brings students of law, civil engineering and construction management together to try an engineering failure case in moot court. The forensic cases, drawn from my professional experience, are summarised for the law lecturer and used to prepare legal pleadings. Pleadings at hand, law teams hire technical experts to investigate the failure on behalf of the plaintiff and defence. The experts (engineers & construction managers) compile a technical report and provide expert testimony in moot court. In keeping with professional practice, both teams exchange reports via their lawyers in advance of a court appearance.

In the following paragraphs, the process of preparing for court is outlined for a sample case and reflections on the experts' experience provided. The tangible benefits of the collaborative endeavour are also highlighted and some suggestions for improving the educational experience offered.

Setting the Scene

There is a growing body of opinion that education today must break from the traditional didactic model in preference for a more student centred learning environment (Felder, 2012; Pink, 2009; Robinson, 2011). Solutions to today's challenges are invariably crafted in small collaborative teams involving people from different disciplines and diverse backgrounds working to solve problems on a global stage. Such approaches allow problems to be solved in more efficient and creative ways. So, if current students are to learn problem solving skills and succeed in such environments, then the educational methods employed at universities and institutes of higher education must reflect the reality of modern business. Specifically, education must engage the learner in an experience that prepares them for an unpredictable future; one that will certainly encompass change. It is not however suggested that we abandon the long established practice of tutoring students through lectures. And it is certainly not suggested that working collaboratively in

small groups eliminates the need for individual study or the need to spend quiet time poring over complex challenges in the engineering curricula.

Instead, what's proposed is a model that places students at the heart of the learning process by actively involving them in carefully contextualised problems. This is best done by using well conceived projects or triggers and these require careful planning allied with appropriate technical and expert support. In such learning frameworks, the student must be given the autonomy to organise and plan their work and take responsibility for delivering an acceptable and economic solution within an allotted time. In the civil engineering programme at UL we find this freedom produces student engagement and motivation to learn (Cosgrove et. al., 2010, Phillips, 2010 and Phillips, 2012). We have also observed that a more student centred approach promotes development of many industry desired undergraduate attributes without the necessity of 'adding on' other courses to an already busy programme.

The subject discussed here is an undergraduate course in forensic engineering & ethics (FEE) and involves students from different disciplines collaborating to gather technical data, undertake analyses, interpret the results and draw conclusions based on the evidence. This process is captured in a written report which is then used as the basis for elucidating and defending facts and opinions in a public forum (Photo 1).



Photo 1: Student engineer being cross examined by opposing council

At the heart of this exercise is an evaluation of each stakeholder's ethical behaviour in the dispute. Engineers Ireland's Code of Ethics (Engineers Ireland, 2009)

provides the benchmark for this assessment and the students' attention is drawn to the following guidelines in the code:

- Adherence to an appropriate Code of Ethics is an essential characteristic of the professional practitioner.
- Engineering is a profession requiring a high standard of scientific education together with specialised training, formation and experience.
- Arising from the rights and obligations of the engineering profession, it is essential that Members have a clearly defined Code of Ethics to ensure that they:

    1. Behave at all times with integrity.
    2. Remain aware of their responsibilities to themselves, to society, to their clients, employers, employees and colleagues.
    3. Strive to maintain the highest levels of competence in their respective technical disciplines.
    4. Seek to enhance and promote the standing of the profession and its practitioners.

Discussion on the meaning and implications of these canons is facilitated throughout the FEE course. In particular, two formal in class debates involving well documented forensic cases anchor these discussions. The debates 'open minds' to many challenging ethical dilemmas faced by engineers in their daily work. It is not uncommon to find students staunchly believing in and defending their debate position only to meekly succumb in a calm and reasoned post debate discussion - an intriguing and rewarding experience that certainly would not emerge, at least in my experience, in a didactic educational model. As the cases investigated for moot court are drawn from my experience as a forensic engineer, detailed case records and information are available for examination including pre-trial depositions in some cases (of course, names of all parties involved are removed from the documents). The database of available failures is summarised in table 1 and a summary of one of these cases is now presented.

Example Case: That Sinking Feeling

An extension to an existing steel framed building in a known karst area resulted in the activation of solution activity. The site was excavated into a hill along the eastern side of the property to permit the floor level in the extension to match the floor level in the existing building. The depth of cut varied from ≈1m at the northeast of the site to ≈4m along the south of the site. A void in the overburden soil developed during the earthworks. It was excavated to bedrock, plugged with concrete and the work proceeded as normal.

As a result of this discovery however, a site investigation was commissioned to investigate the bedrock for voids within the build area. Five rotary percussive holes were drilled and rock cores recovered. Shortly after the drilling, a second surface depression appeared in close proximity to one of the borehole locations. Heavy rainfall over a period of days and failure to grout the exploratory holes (normal practice in karst landscapes) are believed to be responsible for the new ground movement (Photo 3). Incomplete surface water drainage made the condition more

acute by discharging concentrate water flows in the vicinity of the open borehole. A legal dispute ensued.

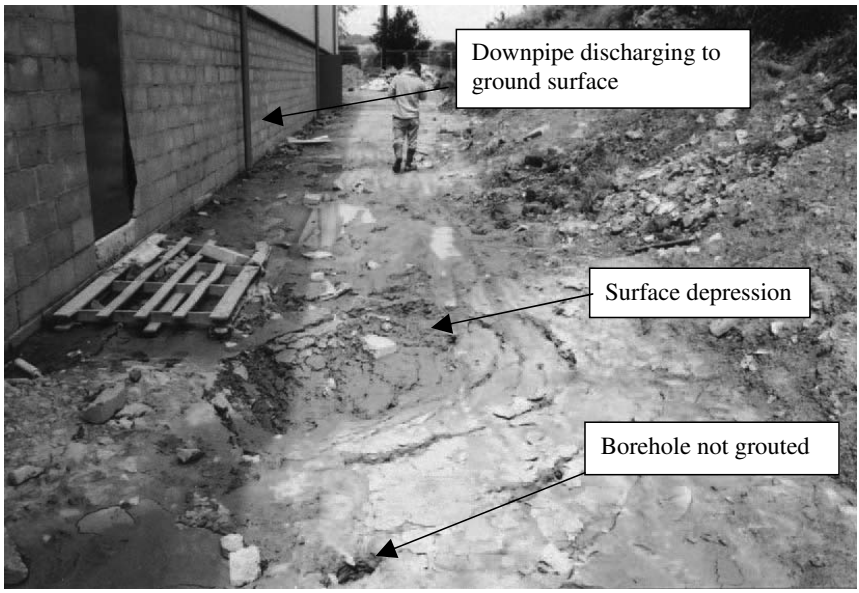


Photo 3: Surface subsidence and open core hole.

How the Collaboration Works

At the start of semester, the FEE lecturer gives a presentation to the law students on 'what engineers do' and how they can assist in providing technical insight and expert evidence in legal disputes. The civil engineers and construction managers also attend the presentation. The law lecturer reciprocates closer to the moot court date by outlining the structure of the legal system in Ireland and providing guidance on the rules of evidence and delivering expert evidence. Useful guidance for court preparation is also provided in Lewis (2003) and Kaminetzky (1991). Briefing notes are provided to guide the technical experts through unfamiliar aspects of the process. In addition, learning seminars or lectures are provided on topics such as; reasons for construction failures, undertaking forensic investigations, writing a technical report, conducting load tests and back analysing well documented engineering failures e.g., Transcona Grain Elevator bearing capacity failure or the Hyatt Regency Walkway collapse.

Interactions with lawyers

A schedule is agreed for the duration of the project and is summarised in table 2. The technical experts and lawyers meet (outside scheduled class time) to discuss the case, brief each other on the case requirements, expectations and duties required for a successful partnership. Unfamiliar terms and procedures are explained by both sides and a schedule of work proposed for their next meeting.

Table 1: Sample forensic engineering cases from the author's personal experience.

Case	Technical Detail
City garage in distress	Salt aggravated corrosion of a below ground multi-storey reinforced concrete flat slab car park
Domestic house basements	The plain concrete basement walls in a row of detached houses failed immediately after backfilling
Underpinning of terraced house to facilitate a deep basement construction on the adjacent development	Poor underpinning practice resulted in the collapse of a 23 foot section of party wall.
Full-scale load test of a steel framed church with eccentric connections	Builders concern for the structural integrity and robustness of the eccentric connections
Collapse of a seven storey steel framed office building over car park	Karstified limestone
No where to go – a split level car park surrounded on three sides	Missing or 'locked' expansion joints in a precast concrete T-beam structure experiences structural distress.
The cracked and popped concrete slab	Numerous issues with quality control of concrete manufacture, slab on grade construction and soil compaction.
A dangerous fraternity	An old red brick building moved laterally due to lateral ground movements during caisson construction on the adjacent site. Ethics of the structural engineer
Tilting tanks and their rescue	Bearing capacity failure during hydro-testing. The cost of not having a site investigation
The day the pier collapsed	A surcharge load on adjacent ground resulted in a slip circle failure that wiped out the adjacent 100 year old timber pier.
Those Pesky Trusses 1	Failure during construction due to wind loading and inadequate bracing
Those Pesky Trusses 2	Failure due to drifting snow.
A lesson in RC Detailing	Emergency changes required when concrete could not flow around a dense array of steel bars from radial beams arriving on a central column.
Pile load test failure	Lack of local knowledge led to incorrect design of driven in-situ piles. Drained conditions assumed when in reality undrained conditions prevailed.
Problems during the construction of a combined bank and apartment building	CFA Piling too close to existing building resulted in structural damage
That Sinking Feeling - The extension construction and ground investigation led to building cracking	The excavation of overburden and non grouting of subsequent SI probes in a known karst area led to cracking in a new building
Mitchell Street Terraced Housing	Building on an old municipal landfill
School fire poses integrity doubts for adjacent classrooms	Load testing roofs of rooms adjacent to the fire – steel frame
Cracking in an old dwelling due to ground vibrations	Driving sheet piles adjacent to an existing dwelling cause structural cracking.

Both teams then advance their research independently and re-convene on an agreed date to assess progress. Engineers work in teams (Photo 2) to brainstorm the problem, develop a plan for investigation including research and testing (if necessary). The brainstorming exercise results in a hypothesis that requires case specific data. The experts make a written request for this data to the FEE lecturer. If the material requested is available, it is provided and the hypothesis tested against these data, validated or modified as required. Relevant material that is not requested but is available will not be offered. So, the team that explores all possible failure modes, including ones less favourable to their client, are most likely to succeed in moot court.



Photo 2: Student engineers brainstorming.

Day in Court

Having tested the veracity of their report findings, expert teams arrive in court prepared to give evidence. As is customary, the courtroom protocol resides with the judiciary who invite opening statements, instruct legal teams, call witnesses etc. The moot court proceedings are video recorded and uploaded to the university's learning management system for subsequent viewing. This has proven to be a valuable resource for students to improve their communication skills, professional appearance and demeanour along with verifying the clarity and exactness of their evidence.

What the students say

The following are just two reflections on the moot court experience. These generally reflect the views of the (63) engineering students taking the course. In fact, the module is very well received and obtains top scores in post module student surveys.

Student 1
I have learned a great deal during this experience and I believe I am better prepared for professional life as a result. I have learnt a lot about teamwork during my time here but this project took it to the next level and was a real eye opener for me. Before, we generally worked in teams with people of the same discipline as me; who had a similar level of knowledge about the project topic and understanding of the material. This time however our team consisted of law students who had no previous experience with building materials such as concrete and the issues associated with it and similarly, I had very little knowledge of the legal issues within the project which had to be addressed. It required a much greater effort than I first anticipated explaining my research and technical findings on the case but I now realise I must take my audience and their backgrounds into consideration when trying to convey technical information and choose appropriate language to facilitate understanding and meaningful communication.

Table 2: Programme schedule

Week Number	**Activity**
Wk 1	○ Law and engineering lecturers agree on legal cases to be tried in moot court. ○ Law and engineering teams of 4 – 5 students assigned to each case
Wk 2	Civil engineering lecturer presents to law students on civil engineering & how lawyers can expect to interact with the engineering profession.
Wk 3	Law teams make initial contact with civil engineering and construction management teams. Legal pleadings discussed and legal instruction issued to the engineers for the defence or plaintiffs.
Wk 4, 5, 6, 7	Engineers undertake research, testing and analyses in support of their case. This also requires regular briefing sessions with their legal team – the essence of the written report is sculpted during this period.
Wk 8	○ Expert report is written, proofread, revised, proofread again and submitted to the lawyers via the online learning management system. ○ Reports exchanged between expert teams via their respective lawyers – both sides familiarise themselves with the content and technical positions of the opposing side. ○ Law lecturer presents to civil engineering and construction management students on the law in Ireland and information relevant for engineers interacting with lawyers e.g. giving expert evidence.
Wk 9	Moot court.
Wk 10	Moot court.
Wk 11	Student reflections & peer and self assessment submitted online.

Also I learned it is much more difficult to organise group meetings between people who have clashing timetables and various other deadlines and projects which they are focused on. Greater communication and collaboration is needed to facilitate everyone's needs and handle their other obligations. To do this you must lead by example and be willing to make concessions as we have to cooperate with each other to ensure the team works efficiently and smoothly. I think this was a major issue at the beginning of the project and possibly a reason why so many of the trials were delayed combined with the fact nobody was stepping up as a leader within the group to direct work, sort out clashes with timetables and also inspire other group members to participate fully in the project. I noticed this and tried to rectify it by standing up and taking a leadership role within the group. The group as a whole worked much more effectively after this.

Looking back on it perhaps the most valuable experience I got was being the witness on the stand during the moot trial itself. It was intimidating at first especially with the courtroom environment and the law students wearing their gowns and microphones set up. It seemed very realistic and daunting but once I got up onto the stand and started to answer the questions my nerves subsided and I grew in confidence. I was very proud of my achievement and was thrilled when I heard the judgment. This experience as a whole was very rewarding for me and although it had its frustrating moments it was an enjoyable exercise which will stand to me in the future. My eyes were also opened to the ethics associated with engineering and how a professional should conduct himself and the responsibilities he must bear in his work whether it be designing, constructing or investigating buildings. This was most definitely a very beneficial experience.

Student 2
Looking back on the last twelve weeks of Forensic Engineering I can honestly say I enjoyed the module and found myself looking forward to the lectures.

I suppose it is hard to teach a person ethics at this stage in our lives as most of us will already have decided what is right from wrong. However from a professional ethics point of view I found my knowledge in the area increased as a result of the different case studies. The case studies were dealt with well in the lectures as we were given the facts of a particular situation and then it was up to us to decide individually what/who we thought was right/wrong. The discussions in class were great as a fellow student might make you think & evaluate your own decision just as much as the lecturer or slide could. I found the case studies very interesting. Sometimes while doing other project work, if I found myself getting bogged down or needing a break I found myself looking up the case studies and having a read.

> The moot court was an interesting exercise and one which was totally different to anything I have experienced in other modules throughout my four years at U.L. It really outlined how as an 'expert witness' we had to tell the facts and not let our emotions interfere with the evidence we presented i.e. no matter what/who we thought was right or wrong we just had to present the facts and answer the questions asked of us honestly. Though it was noted how we could advise the lawyers to put emphasis on a certain fact.

Challenges and Contemplated Improvements

While the normal university scheduling challenges of room size and availability exist, this is not an obstacle when staff and students see the benefits of collaborating. It does however require all parties to do extra work outside the normal timetabled hours. The law students are in the second year of their programme while the engineers and construction managers are in their final year (year 4). Over the past three years there has been a delay in the lawyers contacting the engineers. This is problematic for the engineers given the amount of work required to read the case information, investigate the failure, write and exchange expert reports and prepare for trial. It made meeting the deadline stressful and adjournments had to be facilitated on a number of occasions. Also, when the first moot court were run three years ago, it was evident the judiciary; reading from a prepared script, had predetermined the outcome, without any reference to expert testimony. Their decision was based solely on case law. This has been addressed in the interim by the addition of 'context seminars' by both the law and FEE lecturers. These appear to have helped the lawyers engage meaningfully with the engineers in resolving technical disputes. The cut and thrust of the trials is now much livelier as the expert testimony along with case law are at the heart of the determinations. For this outcome, discipline, teamwork and mutual respect is required on all sides.

Being novices to the trial process, it is obvious that both the lawyers and engineers would benefit from some hands-on tuition in preparing for trial, giving evidence and asking and answering questions - interestingly, student reflections do acknowledge that no amount of preparation can compensate for 'experience in the dock.' A visit to court to observe a civil trial and inviting experienced engineers and lawyers to role play in moot court are under consideration. Finally, an evaluation tool is currently being developed in conjunction with the law lecturer to measure the effectiveness of the moot court as a learning experience for both law and engineering students.

Concluding Comments

There can be little doubt that this experience leads to motivated and enthusiastic students. The autonomy afforded to the students means they are driven to succeed. The added challenge of performing before their peers in law (who they do not know) raises the bar even higher. I believe the exercise gives the students a real experience of being a Professional Engineer and a glimpse into the interesting, challenging and rewarding career that lies ahead. On ethics, the students have developed a greater awareness of their ethical responsibilities to society, their employer and the profession they represent. It is fair to say the ethical dilemmas within the legal cases produced engagement and emotions rarely encountered in the classroom. I guess in matters of the heart, we're all on the same footing and

everyone's opinion is valid. All told, the rewards from this educational experiment far outweigh the effort required in organising and facilitating the experience.

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Forensic Engineering: Informing the Future with Lessons from the Past
ISBN 978-0-7277-5822-4

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doi: 10.1680/feng.58224.330

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The role of engineers in dispute resolution and litigation

Anthony Morgan FICE CEng BSc(Eng) Hons ACGI MCIArb, London, UK

Forensic aspects of construction project management

Introduction

When considering the role of engineers in the course of a dispute, the most obvious issues will be the analysis of the engineering elements in a failed, delayed, cancelled or otherwise distressed project. This is expected to include, for example, the analysis of design, deliverables, construction methods, material selection, environmental factors, the adherence to codes of practice and relevant engineering standards. Such activity is required in order to identify the cause of failure and ultimately the allocation of responsibility following an incident or event. However, engineers can also add considerable value by applying analytical methods to investigate and analyse the causes of delays and disruption and performance of the project management function. This may include the review of management actions arising from technical, commercial or financial issues, or the review of project processes such as change management, procurement, risk management, reporting and governance.

Capital projects have a number of inherent risks and as projects are becoming more complex, with more "mega-projects" emerging all the time, the potential for failure increases. There have been a number of high profile problem projects in recent years, such as Wembley Stadium, the Scottish Parliament building or London Underground network upgrades.

When it comes to identifying why projects have been delayed or have suffered an increase in costs and what was the root cause, engineers with a project management background have valuable insights and skills to offer in helping organisations manage the dispute resolution process and in the field of Expert Witness work.

This paper will discuss:

(a) Common delay analysis techniques;
(b) Establishing the link between cause and effect through a forensic analysis of project records and programmes;
(c) The organisational analysis of project governance and controls and their effect on project performance; and
(d) Lessons learned from disputes experience.

Common delay analysis techniques

Most construction contracts include some form of process for levying damages for late completion. These vary in complexity and severity depending on the type of contract and nature of the project. It is always important that there is an agreed completion date against which performance can be measured. Most contracts will have defined mechanisms for dealing with change and risk events, and provisions for adjusting the completion date, typically through an Extension of Time (EoT) award to the Contractor.

As a general rule, it is considered best practice for an EoT to be issued as soon as practically possible after an event giving rise to entitlement has occurred. This provides all parties with certainty as to their rights and responsibilities under the contract with respect to completion milestones, payment milestones, and entitlement to delay damages. A common approach for dealing with change or delay events in contracts is for the Contractor to assess the impact in terms of time and cost, and make an application for an EoT (and cost adjustment). The contract will define the circumstances in which a Contractor is entitled to an EoT and where they are not.

One of the main causes of dispute is that contracts are often designed to deal with individual changes or delays, and can become difficult to administer where the project involves many changes or where changes either overlap in time or combine in consequence. Where contracts require the Contractor to identify the impact of change in some detail, circumstances can arise where the Contractor finds it difficult to process the changes quickly enough to be awarded EoTs in a reasonable time. This may lead to protracted periods of negotiation which are detrimental to the project and, at worst, can lead to termination of the contract for lack of performance.

One of the main areas of the Programming Expert Witness is in analysing delays and over-spends in order to establish the Contractor's entitlement to an EoT and additional costs. The remainder of this section provides a summary of the more common delay analysis techniques, focussing on critical path programme based analysis. I have used the titles commonly given to these approaches.

Impacted as-planned

The "impacted as-planned" analysis takes the original or "as-planned" programme as the starting point and adds specific identified delays into the original plan in order to show their impact on the completion date.

Such an analysis has the benefit of being relatively simple to produce (and therefore quick and cheap to produce), is easy to understand and can be used where detailed as-built records of progress are not readily available. The main drawbacks are: it produces a theoretical forecast as it does not represent what actually happened; and it does not account for actual delay mitigation measures or re-sequencing of activities as it maintains the original project logic.

In reality, it is likely that the Contractor would have taken measures to mitigate the delay (or should have done). However, such an approach can be a useful in modelling the high level impact of a few specific events, rather than trying to capture all the consequences of the delaying events that occurred.

Collapsed as-built

The objective here is to subtract the delays (or additional work) from the actual performance of the contract (the "as-built programme") to arrive at a date that the project would have finished if the delays had not occurred. This approach is usually used where the Contractor did not follow its plan in carrying out the work and can be used to resolve issues between concurrent Owner and Contractor caused delays. For example, the Contractor may seek to remove all of the Owner caused delays from the as-built programme to show when the

Contractor would have completed the project had the delays that were the responsibility of the Owner not occurred.

The main advantages of this analysis are: it is based on actual performance data; it includes how long particular work activities actually took to complete; and it does not rely on the reasonableness of the planned programme, which may be very different to how the work was actually completed. The other advantage is that by only removing Owner caused delays, the approach is inherently conservative as the Contractor caused delays and disruption consequences are taken into account in the as-built programme.

The main disadvantage is how to adjust the as-built programme when you remove delay effects or additional work activities, as it is possible to end up with an illogical programme. If a delay is removed at the beginning of the programme, then every subsequent activity will show as starting earlier than it actually did. This can appear odd where events occur at specific times, such as a plant shutdown for the Christmas period showing as occurring in the summer.

Windows analysis

This approach uses "after the event" analysis of the contemporaneous programmes to identify the delay to completion which has occurred during certain periods (or windows) in time. The programme at the start of each time window is compared to the programme at the end of each window. Changes to the critical path are identified and the Expert then seeks to explain why and how the changes occurred using contemporaneous evidence, engineering knowledge, logical analysis and experience. The time windows are normally monthly or the actual time period that the Contractor chose to update the programme.

The main advantage of this approach is that if the programmes are updated with physical progress, then critical path, concurrency of delay and pacing delay issues can be analysed. The main disadvantage is that it requires programmes that are reasonable representations, at the start and finish of each window, of both the physical progress achieved and the forecast to complete. The usual complaint is that the programmes are often not accurate in one respect or another (such as not showing procurement periods, temporary works or the actual method of working) and so the forecasts of completion are not reliable.

The other main disadvantage of the windows analysis approach is that it is very resource intensive as all the programme updates and the supporting contemporaneous documents need to be examined.

Time impact analysis

This is the generally accepted as the preferred method (by the Society of Construction Law and also the Engineering and Construction Contract (ECC) of showing the impact of known events on contemporaneous programmes to forecast the cumulative impact on completion. This is often specified in capital project contracts as the required means for a Contractor to demonstrate the overall impact of delaying events when submitting a request for an extension of time. The main advantage is that it produces contemporaneous forecasts and takes the current status of the project as the starting point. The main disadvantage is that, whilst it is usually the preferred method of assessing a single causative event, it becomes increasingly challenging as the number of events increase. This may be a particular problem where the parties are unable to agree the time and cost consequences of delaying events quickly especially if there are numerous design changes.

Forensic time audit

This approach investigates retrospectively what happened and why in order to produce an accurate as-built programme. This is sometimes referred to as the "as-built critical path method" where the objective is to determine the actual path of critical activities that led to completion.

The approach is to start with the completion of the project and to work backwards through the project, to determine (for each as-built activity) what the key predecessor activity was that allowed that work activity to start. When assessing the critical predecessor the Expert has to answer the following questions: why the activity started when it did?; why did it not start earlier?; what had to be completed before it could start?; and what information, resources, access or space was required?

By examining the contemporaneous evidence, the cause and responsibility for delays can be established. The main advantage of this approach is that it is factually based on contemporaneous evidence, and that it can be completed where either or both parties do not want to rely on the reasonableness of the programmes produced.

This approach tends to be preferred when the project's critical path is not clear and programmes are unreliable and it can be difficult to establish in detail what actually happened. Often, various other techniques, such as tracking key performance indicators over time, or a production based analysis can be used in order to support the investigation. In addition to this, the prospect of legal action will drive parties further apart and make each party reluctant to share information and data which inhibits the technical and factual investigation. The investigation can sometimes only be fully verified once the legal process has compelled the parties into disclosing the project facts.

Cause and effect – understanding the programme

In order to successfully and fully allocate responsibility for delay in a significant project dispute, it is generally necessary to seek to prove connections between cause and effect and the associated time and cost impacts of these effects. One of the most challenging steps is the analysis of delays to completion associated with individual causes, isolating the effects of errors, rework and accidents from other causes such as change orders (or variations). Most analysts will seek to use a critical path method (CPM) to represent how the project was planned compared to how it was actually built and identify the cause and responsibility of changes. An engineer's skill set lends itself directly to this task; engineers understand (or can determine) the inter-relationships between activities; distinguish between essential and preferential logic; and understand mitigation options and actions when considering the complex consequences of delay events.

To bring this to life, a case study of a project dispute is described below which explains why a particular method of analysis was preferred. This example comprises an analysis of concurrent delays and highlights the requirement for an engineer's experience and knowledge of the construction process.

Case study – European power station

The matters in dispute relate to the Contractor's entitlement to an EoT for the completion of the construction of a 1200MW power station. This dispute arose after the completion of the project. The Owner submitted an application for liquidated damages which had become due on late completion of the project whilst the Contractor had issued a request for an EoT.

The commissioning had proceeded relatively quickly, but had started late. The dispute focused around what had caused commissioning to start late – Owner responsible delays or Contractor responsible delays (or a combination of both).

The commissioning of a power station relies on the completion of a number of workstreams with various interdependencies. Figure 1 shows the planned key dependencies between the various systems that were needed for the start of the Initial Rolling phase of the Gas Turbine, the first stage of commissioning a power generation unit.

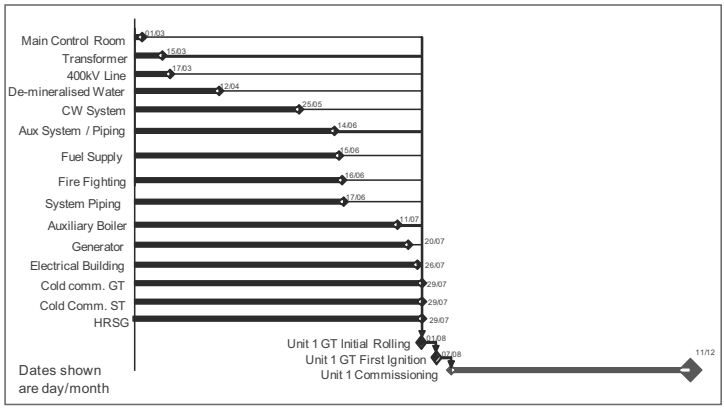


Figure 1: 'As-planned' key dependencies for the start of Initial Rolling stage

The dispute focused on which area of the project delayed the commissioning; the Cooling Water System ("CWS") and associated tunnelling and marine works, or the main plant works being the power plant itself and the associated systems and equipment. The Contractor claimed that the CWS was the main or driving cause of delay, due to a delay in the commencement of the marine works caused by delays in obtaining the necessary environmental permits by the Owner. The Owner claimed that the plant works were the driving delay due to the Contractor's slow progress and poor management of its subcontractors.

A forensic time audit (supported by various specific detailed investigations) was carried out together with a production analysis to ascertain the driving activity of completion. This was based on the contemporaneous programmes and documents (such as the programme updates, management reports, e-mails, and daily progress reports). There were numerous versions of progressed Primavera programmes which had been produced during the course of the project.

It is generally possible to perform certain analyses solely on the basis of programmes, but any such analysis will be subject to limitations of accuracy of the data used and the quality of the programmes themselves. Where engineers can really add insight is in linking the programmes

together or by filling in the gaps where the programmes provide inadequate explanation for the causes of delay.

In this case study, the analysis combined the information in the programmes with a forensic investigation of the facts. The programmes were useful, particularly in relation to logic and interdependencies between work packages, but there were several examples where the programmes did not accurately reflect what actually happened on the project.

The Expert team developed a high level summary of the project, based on the programmes and the witness statements, which identified the key work packages and the inter-dependant logic. A factual chronology of key events was then developed to test and verify the events in the programmes, particularly start and end dates, and to identify the critical path. Where the critical path was not clear, further investigation was undertaken to give a detailed narrative of events at key dates.

This led to a comprehensive explanation of key events and delays, showing changes in the programmes and alterations to dependencies. The outline of what happened was based on programme data extracted from monthly programmes, with the detailed dates and interdependencies supported by primary evidence wherever possible. The as-built programme is shown at a summary level in Figure 2 below.

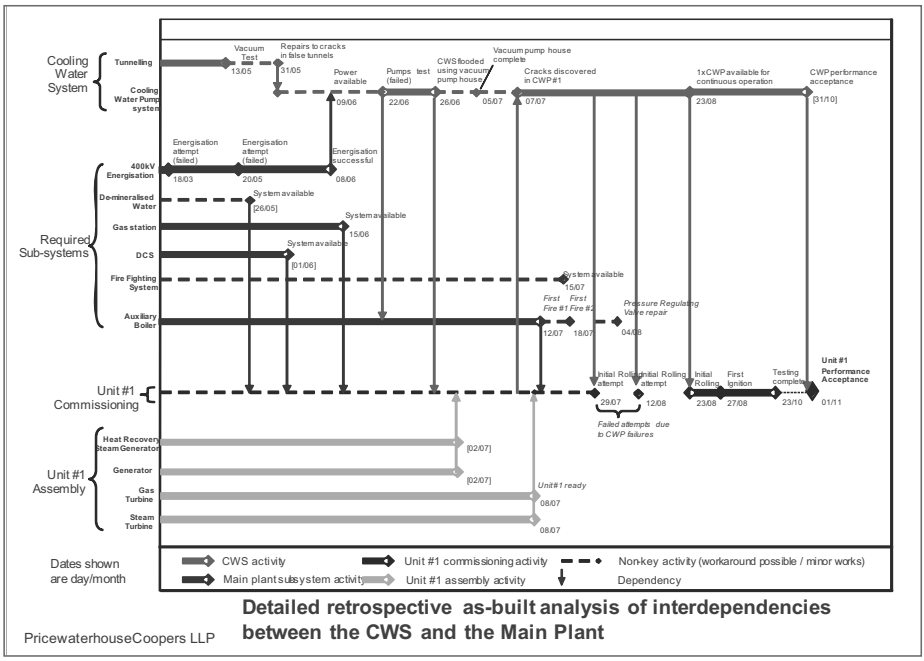


Figure 2: Detailed retrospective as-built analysis of interdependencies between the Cooling Water System and the Main Plant

At first sight, it appears that the CWS (shown in green) was on the critical path prior to commissioning (shown in dark blue), giving rise to an as-built critical path through the CWS. This was the case presented by the Contractor.

A more detailed review of the events showed that a mechanical failure of the CWS pump occurred in the final pump testing, prior to the planned start of commissioning. This led to several weeks of delay to the start of commissioning due to the lack of availability of the CWS works, which makes the CWS appear critical prior to commissioning. However, prior to this mechanical failure, it appeared that the CWS work had been substantially complete for some time.

A competing delay analysis was performed to investigate the status of the key workstreams during this critical period prior to commissioning. A detailed review of the evidence was undertaken to establish how much work remained in each of the key work packages in the period before the pump failure. This is summarised in Table 1 below.

Table 1: Summary of critical and near critical work areas from May to July

	Duration of remaining works to completion by work area				
Date	**CWS**	**HRSG**	**400kV**	**GT/ST/Gen assembly**	**Aux. Boiler**
12-May-08	10-15 days	50 days**	8 days	50 days**	30-40 days*
8-Jun-08	5-10 days	25-30 days*	Available	25-30 days*	30-35 days**
26-Jun-08	Available	5-10 days	Available	10-15 days*	20 days**
12-Jul-08	5-10 days**	Available	Available	Available	Available

Key:

Critical work area	**
Near critical work area	*

The data in Table 1 shows that in May, the CWS was substantially completed, with only a few weeks of work remaining, whereas the main plant had over six weeks of work remaining. By late June, the CWS was available and had passed initial testing, whereas the main plant had three weeks of work to remaining to complete. It was only in early July when the Main Plant was ready that the CWS pump was tested again prior to commissioning and the CWS pump failed, leading to the CWS to become critical to completion.

This analysis shows that it was not correct to surmise that because the CWS was the last activity to delay commissioning, it was responsible for the whole of the delay, as was argued by the Contractor.

The forensic analysis of the project, at a suitably detailed level during the key period of dispute, determined the changing critical path between the main plant and CWS, demonstrating that completion of the main plant was actually critical up to the failure of the CWS pump. The allegation that the project completion was due to the delayed start of the CWS which was due to the Owner was not correct. The CWS was available for commissioning well in advance of the main plant and the Owner was not responsible for the failure of the pump.

By using engineering skills to understand fully the inter-relationships between activities, to interpret project reports and to understand the implications of changes and delays, an accurate assessment of the activities which were truly critical to completion of the project was obtained.

The Expert Report in this case combined a variety of analytical techniques and forensic investigation methods using engineering skills and experience. The dispute was resolved by negotiation prior to a hearing.

Organisational analysis – governance and control

Any dispute or litigation action arising from the delay or failure of a project will inevitably require an analysis of what happened and why. This may include a review of the scope of work or requirements; the project's governance processes and decision-making mechanisms. With respect to decision making, a key area to assess is the effectiveness of the project's governance and reporting processes, to understand who had what information and when. Establishing an appropriate organisation and governance structure during initiation of the project is crucial in avoiding poorly performing projects and disputes.

Project management professionals need to have a rounded understanding and knowledge of how project processes, systems and team skills interact in a complex capital project environment. In the following case study, the lack of an effective governance and organisational structure were inherent in the causes of the project failure.

A key element to a governance regime is the control of management information and how it is produced and the accountability for decision making if below planned performance is measured. This includes a clear delegation of responsibilities in the team delivering the project or service. Project governance is described by the Association for Project Management (APM) as being concerned with:

"*....those areas of corporate governance that are specifically related to the project activities. Effective project governance of project management ensures that an organisation's project portfolio is aligned to the organisation's objectives, is delivered efficiently and is sustainable. Governance of project management also supports the means by which the board, and other major project stakeholders, are provided with timely relevant and reliable information*." (Association for Project Management, 2011 p7)

The APM defines four areas of governance that assist organisations to deliver effective project governance:

- **Portfolio direction** - ensuring that the project portfolio aligns with the organisation's objectives including profitability, customer service, reputation and sustainability;
- **Project sponsorship** - the effective linkage between the senior executive body of the organisation and the management of the project;
- **Project management effectiveness and efficiency** - this relates to the project team and its ability to deliver the project objectives; and
- **Disclosure and reporting** - the component most reliant on the culture of the organisation.

In their 2009 report the National Audit Office identified Metronet's performance and ineffective governance on the London Underground upgrade programme as the underlying cause of failure to deliver a capital programme. Metronet was a consortium formed to deliver the London Underground Ltd (LUL) maintenance and upgrade projects for two-thirds of the London Underground lines in 2003. Metronet won two of the Public Private Partnership (PPP) contracts, while another consortium, Tube Lines, won the remaining third contract. As

Metronet was formed specifically for these projects, the term corporate governance can be considered synonymous with project governance.

The PPP contract set out a 30-year period in which Metronet was responsible for maintaining the track, civil and rolling stock assets, and delivering the station upgrade and modernisation works. In exchange for delivery against four key performance metrics, Metronet received a four-weekly periodic payment called the Infrastructure Service Charge. In addition, Metronet was to deliver some of its capital works programme on a milestone basis.

Metronet's capital programme experienced significant delays, particularly the station upgrade programme. By March 2005, Metronet had not delivered any of the station projects. By March 2006, only 11 out of the 35 planned station projects were delivered, and in the following year only 28 out of the 64 station projects were delivered. Similarly the delivery of track renewal showed that only 44km of the track had been renewed rather than the planned 69km.

Metronet delivered the PPP contracts via a tied supply chain comprising its five shareholders: Bombardier Inc., WS Atkins plc, EDF SA, Thames Water plc and Balfour Beatty plc. A supply vehicle called "Trans4m" was formed by four of the shareholders, to deliver the stations and civil engineering works. Balfour Beatty was appointed to deliver the track upgrade, while Bombardier was to deliver the rolling stock and signalling. The resulting tied supply chain agreements were not necessarily back-to-back with the provisions made in the main PPP contract between Metronet and LUL, which made for a very complicated governance and contractual structure (shown in Figure 3 below).

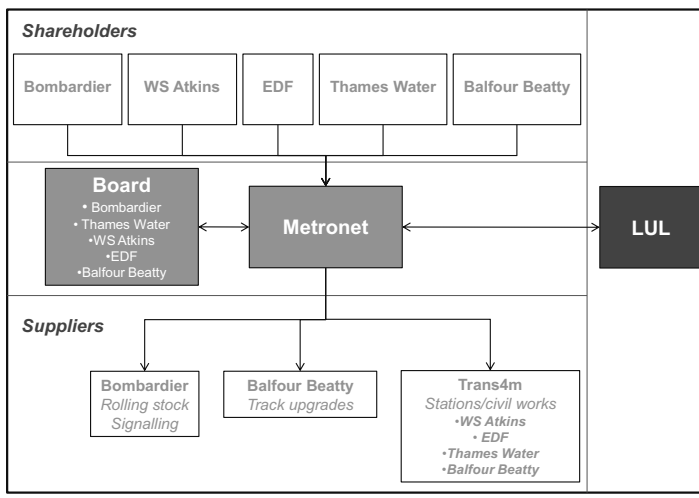


Figure 3: Basic structure for Metronet

Portfolio direction

The NAO concluded that Metronet's governance structure and tied supply chain agreements contributed to the delay to the programme. The shareholders were also the suppliers, resulting in a supply chain that had an element of control over the scope of works and expected to be paid for it under its supply contract. The direction of each of the individual project portfolios

(civil works, track, rolling stock and signalling) were not aligned with that of Metronet itself, which was to deliver the PPP.

Project sponsorship

All five shareholders had to agree unanimously on any decisions made on the project. This presented challenges to the leadership of the project and the decision making process as the shareholders often had different commercial views based on their respective positions regarding the particular asset under discussion.

The NAO found that there was no clear incentive for Metronet's lenders, shareholders or its boards to implement a strong governance regime. The lenders had no incentive as only 5% of their investment was at risk. The remaining 95% was guaranteed by LUL and Transport for London. The shareholders, although having a £350 million equity stake in Metronet, were not incentivised as they were also the beneficiaries of the tied supply chain agreements. The board itself had no incentive to set up a strong corporate structure because there was no independent chairman of the board to set policy and direction.

Project management effectiveness and efficiency

The NAO reported that in Metronet's case, the appropriate authority was not delegated to an appropriate level within the organisation, with the supply chain having significant power over decisions regarding scope. The PPP arbiter also reported that Metronet did not appear to have the resources necessary to deliver the full scope of the track portfolio.

Disclosure and reporting

The shareholders forming the supply chain failed to keep the Metronet board adequately informed about progress and costs measured against delivery. This led to a weak project governance environment as the reporting components were rendered ineffective. Metronet was unable to monitor its costs and act accordingly until it was often too late to mitigate any extra costs that may have arisen.

Conclusion

The NAO noted that although complicated, tied supplier relationships had been successful in the past (for example, by London & Continental Railways to deliver the high speed railway between St Pancras Station in London and the Channel Tunnel). The success of this tied relationship was attributed to two key differences: (i) additional shareholders; and (ii) close partnership working.

The presence of additional shareholders to those in the tied supply chain would have provided greater accountability and scrutiny to the project. This would have ensured that the project portfolio direction aligned with those of the corporate objectives. Close partnership working, also described as "a policy of total openness", ensured a reporting regime provided the information required for timely decision-making so that it was readily available and reliable.

Lessons learned

Commercial organisations can face reputational and financial loss if the delivery of their capital projects prevents them from achieving their corporate objectives. With the experience gained from over 100 dispute analysis and forensic assignments, I have selected five common themes that contribute to project failure. Some of these may appear obvious but are often overlooked or can lose the management focus they deserve.

Business case/ key success factors

The business case and clear definition of success for the business is often overlooked or taken for granted. Vision, objectives and business benefits should be fully integrated and supportive of each other in achieving the overall business change or need. The business case should be regularly monitored to report on objective and benefit realisation. Where business case realisation is not being achieved, or the business strategy has changed, there should be a realistic consideration to amend or even abandon a project that no longer meets its objectives. Too often, projects are pursued by stakeholders even where no clear business benefit remains.

Scope/change control

Robust and appropriate change control processes must be adhered to and regularly challenged in order to maintain control and effectively manage risk. This should be integrated with contract management and administration processes so that the mechanisms in the contract are applied correctly and appropriately to manage change. Attempting to manage change outside of the contract, as often occurs, is a common cause of loss later on.

Governance

Governance processes should be sufficiently mature to manage the risks involved in complex capital projects. The governance of a project should have clear lines of accountability and responsibility. The project team should be clear on what they are individually responsible for and who they are accountable to for the delivery of the project. A diagnostic can be used to indicate if the existing support frameworks are appropriate. An example of such a governance diagnostic is shown in Figure 4 below. On the diagnostic tool a project that aligns more to the right, for example a project that was business critical, in multiple geographies and had a high number of third parties, would require a greater depth of experience in the project team and a higher level of independent challenge.

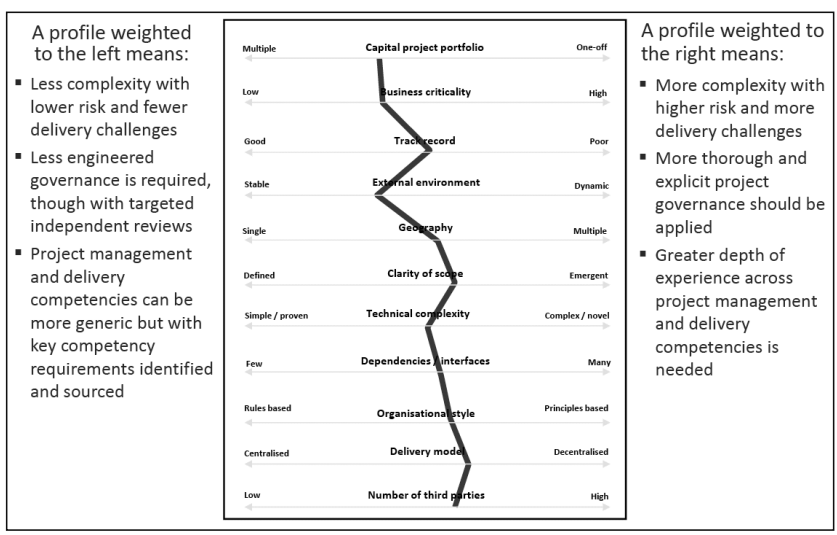


Figure 4: Governance Maturity Diagnostic

Risk and contingency

To mitigate delivery risks, the maturity of project management systems and people competency in the delivery team must both be strong (as shown in Figure 5 below).

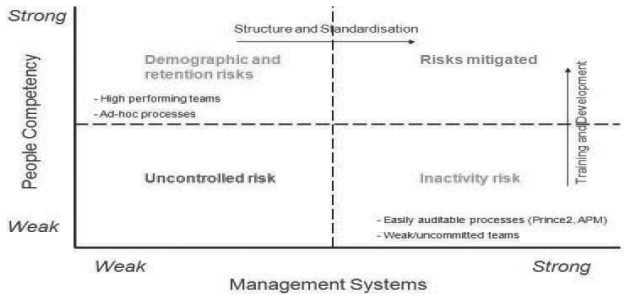


Figure 5: People and Process Matrix

Reporting (timely information) and decision making

The production of management information relies on the ability to produce data from the core project management processes. It is difficult for management to make good quality decisions or interventions without timely and accurate information together with defined authority and issue escalation processes.

Overall conclusion

Civil engineers gain valuable skills and experiences through both project work and forensic investigations of projects that can be effectively applied during dispute resolution. Disputes relating to delays, disruptions, contract issues, governance and the management of projects require engineering and project management experts to perform analyses and provide opinions. The experience and knowledge gained in dealing with a dispute process provides insights into how to avoid disputes on future projects.

The governance and project management of projects can have a significant bearing on project performance, particularly with respect to change control, progress reporting and decision making processes. Engineers must be alive to these issues and acknowledge the time and resources required to deal with and mitigate risks early.

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Forensic Engineering: Informing the Future with Lessons from the Past
ISBN 978-0-7277-5822-4

ICE Publishing: All rights reserved
doi: 10.1680/feng.58224.342

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The "Standard of Care" and Negligence of the Forensic Engineer

Joshua B. Kardon, PhD, SE, F.ASCE, 1930 Shattuck Avenue, Suite C, Berkeley, California 94704, 001-510-548-1892, Fax: 001-510-548-1895, jbkse@jbkse.com.

Introduction

In civil litigation in the United States, forensic engineers are occasionally asked to present opinions regarding a practitioner's professional negligence. This question hinges on the concept of the "standard of care," which can be thought of as the boundary between negligent and non-negligent error.

In providing forensic engineering services, including assessing another engineer's performance relative to the standard of care, forensic engineers themselves are held to that same "standard." This paper presents the concept of the standard of care, and describes how a forensic engineer's services are measured relative to the standard of care. It also discusses the concept of the immunity from negligence afforded to expert witnesses, and the current trend of erosion of that immunity.

The Standard of Care

Case law (e.g.: *Alfred Gagne et al., v. Benjamin E. Bertran* 1954, *City of Eveleth v. Ruble* 1974) in the US allows professionals some leeway with regard to error: there is some level, degree or type of error which a professional may commit without incurring liability, or responsibility to pay for damages arising from that error. However, some errors are of the level, degree or type that the professional may be liable. Those errors are called negligent errors, and the boundary between negligent and non-negligent errors is called the "standard of care."

Definition of the Standard of Care

The standard of care has been defined in case law and has been presented in what are known as standard or pattern jury instructions. After a civil lawsuit is argued in court, and the jury is preparing to deliberate on the evidence presented and answer the ultimate question of fact raised by the lawsuit, the court gives the jury a set of instructions they should follow to answer the questions.

Pattern Jury Instructions:

Sets of pattern jury instructions have been compiled into reference documents. One such compilation of pattern instruction, the Book of Approved Jury Instructions (BAJI 1986), included the following:

> *6.37 The Duty of a Professional*
>
> *In performing professional services for a client, a professional has the duty to have that degree of learning and skill ordinarily possessed by reputable professionals, practicing in the same or similar locality and under similar circumstances. It is his or her further duty to use the care and skill ordinarily used*

> *in like cases by reputable members of his or her profession practicing in the same or similar locality under similar circumstances, and to use reasonable diligence and his or her best judgment in the exercise of professional skill and in the application of learning, in an effort to accomplish the purpose for which he or she was employed. A failure to fulfill any such duty is negligence.*

The Book of Approved Jury Instructions was revised since that 1986 publication, and in fact is no longer typically used in California. In its place, The Judicial Council of California has published Judicial Council of California Civil Jury Instructions, (CACI 2011) which includes the following:

> *600. Standard of Care*
>
> *[A/An] [insert type of professional] is negligent if [he/she] fails to use the skill and care that a reasonably careful [insert type of professional] would have used in similar circumstances. This level of skill, knowledge, and care is sometimes referred to as "the standard of care."*

The wording of a jury instruction is not limited to the pattern instructions, and if, based on fact and opinion evidence admitted in court during the trial, a judge finds that a different instruction would more accurately state the law and be understood by jurors, the pattern instruction need not be used.

Elements of the Standard of Care

BAJI 6.37 identifies attributes of learning, skill, care, diligence and judgment as duties of a professional, and states that those attributes are to be applied by the professional in an effort to accomplish the purpose for which the professional was hired. The CACI instruction, on the other hand, lists skill, knowledge and care as attributes contributing to the duty of a professional, but does not include diligence or judgment, nor does it require consideration of the purpose for which the professional was hired.

CACI also instructs the jury to compare the professional whose performance is being examined with that of what "a reasonably careful professional would have used in similar circumstances." BAJI doesn't require comparison with what someone "would" have done – it requires comparison with what reasonable practitioners actually did in similar locales and circumstances. Testimony concerning what a reasonable professional would have done allows an amount of conjecture because the measuring of what one "would have done" may be more subject to bias than would measuring what someone actually did. For these reasons, BAJI's instruction concerning the duty of a professional is a better definition of the standard of care than is the CACI instruction.

The jury can determine the facts concerning the standard of care, and concerning the performance of the defendant engineer, by answering the following questions:

- Does the defendant have the degree of learning and skill ordinarily possessed by reputable professionals, practicing in the same or similar locality, and under similar circumstances?
- Did the defendant use the care and skill ordinarily used in like cases by reputable members of his or her profession practicing in the same or similar locality and under similar circumstances?
- Did the defendant use reasonable diligence?
- Did the defendant use his or her best judgment?

- Did the defendant do all that in an effort to accomplish the purpose for which he or she was employed?

To answer those questions, the following also have to be determined:

- Who are "reputable professionals"?
- What are "same or similar" localities and circumstances?
- What is "skill"?
- What is "care"?
- What are "like cases"?
- What learning and skill are "ordinarily possessed"?
- What care and skill is "ordinarily used"?
- What is "reasonable diligence"?
- What is "judgment"?
- What is the defendant's "best judgment?"
- What was the purpose for which the defendant was employed?

Expert Testimony Concerning the Standard of Care

According to U. S. Federal Rule of Evidence 702,the purpose of expert witness testimony is to "assist the trier of fact to understand the evidence or to determine a fact in issue" (FRE 2011). Under that rule, expert testimony is admissible in Federal Court if: 1) the testimony is based upon sufficient facts or data, 2) the testimony is the product of reliable principles and methods, and 3) the witness has applied the principles and methods reliably to the facts of the case. The wording of 702 has been influenced by case law such as *Daubert* (1993) and *Kumho* (1999) which established criteria for courts to rule on the admissibility of scientific and non-scientific technical evidence. Some state jurisdictions have not embraced FRE 702, or the Daubert and Kuhmo standard for admissibility, but instead follow the standard defined in *Frye* (1923), which requires that the expert's testimony "must be deduced from a well-recognized scientific principle or discovery, and the thing from which the deduction is made must be sufficiently established to have gained general acceptance in the particular field in which it belongs." One application of the "general acceptance" test requires the methods used by an expert to support an opinion be those methods that are generally accepted within the relevant scientific community.

To find that a defendant engineer is negligent using the definition of the duty of a professional as presented in BAJI 6.37, a jury would have to understand the learning, skill, diligence, judgment and care of the defendant and of other professionals in similar locales and circumstances, and understand the purpose for which the defendant engineer was hired. These are facts the jury typically requires the assistance of expert testimony to understand in order to answer the above bulleted questions. Expert witnesses may testify for the purpose of assisting the jury in their determining the answers to those questions.

Forensic engineers can be retained by attorneys or parties to the lawsuit (or in some situations by the court itself) to investigate the technical aspects of the dispute and, when admitted by the court, to offer expert opinion testimony for the purpose of assisting the jury. The forensic engineer expert is ethically obliged to offer truthful evidence, developed through valid and reliable means, in the best possible light for the party that retained the expert (Kardon, Schroeder & Ferrari 2003). Ideally, competent forensic engineers testify for each side, allowing the jury to understand the technical evidence from all sides. With that understanding, the jury can answer the ultimate questions of fact concerning the defendant's negligence.

Professional Negligence of an Expert Witness

Licensed professional engineers providing forensic and expert services are held to the same standard of care as the defendant in a professional negligence suit. That is, they have the duty to possess and use the level of learning, skill and care of engineers providing forensic and expert services in similar locales and circumstances, and to use reasonable diligence and their own best judgment in an effort to accomplish the purpose for which they were hired. If forensic engineers fail to fulfill that duty, they can be found negligent and therefore might be liable for damages caused by their negligence.

Four general areas where an expert witness may face claims of professional negligence, or failing to meet the standard of care for expert witnesses, are: failing to accurately estimate damages, failing to perform an adequately extensive investigation, failing to persuade the trier of fact, and changing the expert's opinion (Erikson 2011).

There are not many cases in the published record where a forensic engineer expert has been sued for negligence in providing those forensic or expert services. One, *Forensis Group, Inc. v. Frantz, Townsend & Foldenauer* (2005) arose out of a wrongful death lawsuit filed after a worker died in an accident involving a forklift. In a deposition of an expert mechanical engineer hired by the attorney for the worker, the expert did not identify any safety standards applicable to forklifts. However, later when the forklift manufacturer moved for summary judgment, the expert stated in a declaration that the vehicle failed several safety standards. The court granted summary judgment, noting that the expert had contradicted himself. The decedent's family filed a malpractice (professional negligence) suit against their forensic expert.

There are cases where experts other than forensic engineers have been sued for professional negligence arising out of their testimony. One such case (*Pace* 2008) involved a medical expert (Swerdlow) retained by a family who was suing a doctor for damages arising from the death of their daughter, a surgery patient. The expert testified in his deposition that the patient should not have been released from the hospital after her surgery because of the amount of pain he understood she was experiencing at that time. He testified that it was his opinion the doctors who released the patient were negligent in that a patient experiencing that amount of pain should have been kept in the hospital for observation and treatment. After reading the opposing party's expert subsequent deposition testimony, however, the plaintiff's expert changed his opinion. His opinion after reading the deposition testimony of the opposing expert was that the doctors did not know of the high level of pain, and instead knew only of the patient's reports of a much lower level of pain. Under those circumstances, the expert was of the opinion that the doctors who released the patient were not negligent. At that point, just days before the trial was scheduled to start, the plaintiff's expert wrote an addendum to his own deposition transcript stating his revised opinion that the original doctor was not negligent. The expert sent that addendum document, without notifying the plaintiff's attorney beforehand, to both the plaintiff's attorney and the defendant's attorney. The defendant filed and was granted a motion for summary judgment based on the plaintiff's expert's addendum to his deposition wherein he stated his opinion that the doctors were not negligent. The family sued the expert for professional negligence in his providing expert witness services alleging that the way he abruptly and at such a late date changed his opinion was the cause of the courts dismissal of their case against the doctor. The expert challenged the suit against him, claiming that his changing his opinion was not the cause of the plaintiff losing their case. That question of causation was adjudicated by the Federal Circuit Court and the negligence case against the expert was allowed to proceed.

Immunity of the Expert Witness

United States Supreme Court has ruled that witnesses cannot be sued for the content of their testimony. In *Briscoe v. LaHue (*1983), the Court of Appeals for the Seventh Circuit held that witnesses are absolutely immune from liability damages based on their testimony. The concept of witness immunity is generally understood to be in the public interest because without it, witnesses might be unwilling to testify because of a fear of legal retribution. Cases that have been brought against expert witnesses point out the inherent conflict between a need to encourage testimony by expert witnesses and the need to assure that expert witnesses testify in a professional manner.

In a dissenting opinion in *Pace*, the case cited above concerning the medical expert, Appellate Court Judge Gorsuch wrote:

> *Here, we have cause for even greater concern. Allowing this claim to march along sends the message to would-be expert witnesses: Be wary – very wary – of changing your mind, even when doing so might be consistent with, or compelled by, the standards of your profession. Neither can there be any doubt this is exactly the message plaintiffs wish to send, candidly explaining, as they do, that their real beef with Dr. Swerdlow was his failure to "deliver[] the expert liability opinion he had promised the Paces all along." In our legal system, demanding that experts "deliver" a specified opinion, as opposed to their honest judgment, is supposed to be ethically out-of-bounds –not the basis for a cause of action.*
>
> *Parties already exert substantial influence over expert witnesses, often paying them handsomely for their time, and expert witnesses are, unfortunately and all too frequently, already regarded in some quarters as little more than hired guns. When expert witnesses can be forced to defend themselves in federal court beyond the pleading stage simply for changing their opinions – with no factual allegation to suggest anything other than an honest change in view based on a review of new information – we add fuel to this fire. We make candor an expensive option and risk incenting experts to dissemble rather than change their views in the face of compelling new information. The loser in all this is, of course, the truth-finding function and cause of justice our legal system is designed to serve.*

Erosion of the Immunity of Expert Witnesses

The immunity of expert witnesses has been eroding in recent years due to such cases as *Jones v. Kaney* in England (2011), and *Pollock v. Panjabi* (2000) among others in the United States.

In *Jones,* a medical expert for Mr. Jones, an injured party in a suit against an insurance company concerning injuries caused by a traffic accident, signed a joint experts' statement concerning the injuries despite the fact that she held an opinion contrary to that presented in the joint statement. Jones' case was settled for an amount with which Jones was unsatisfied, so he sued his expert, Kaney, for professional negligence. Kaney argued that she was protected against Jones' negligence claim by case law establishing witness immunity. The lower court accepted that argument, but Jones appealed. The UK Supreme Court, in a 5 to 2 split decision, reversed that decision and wrote in the majority decision that an expert should "perform his function as an expert with the reasonable skill and care of an expert drawn from the relevant discipline. . . ."

In *Pollock,* Dr. Panjabi, an expert in spinal biomechanics, performed laboratory tests intended to show whether a restraint hold performed by a policeman during an arrest had caused debilitating spinal injury to attorney Pollock's client, a Mr. Green. Dr. Panjabi's testimony was shown to be based on invalid and unreliable testing methods, and therefore was not admissible at trial. Without valid and reliable technical evidence to the contrary, Mr. Green's injuries were adjudicated to have not been caused by the restraint hold, and the court awarded costs against Green. Pollack and Green filed suit against Panjabi claiming, among other causes of action, they were damaged by the professional negligence of Panjabi in his use of invalid and unreliable methods to come to his expert opinion. Panjabi filed a motion to strike because Panjabi was protected by the doctrine of witness immunity. The court recognized that witness immunity was an important and valid doctrine, but that it did not apply in this suit because Dr. Panjabi failed to use valid and reliable means to come to his opinion. The decision to deny Panjabi's motion to strike included this quote from another case, *LLMD v. Jackson* (1999):

> *The goal [of witness immunity] of ensuring that the path to truth is unobstructed and the judicial process is protected, by fostering an atmosphere where the expert witness will be forthright and candid in stating his or her opinion, is not advanced by immunizing an expert witness from his or her negligence in formulating that opinion. The judicial process will be enhanced only by requiring that an expert witness render services to the degree of care, skill and proficiency commonly exercised by the ordinarily skillful, careful and prudent members of their profession.*

The court in *Pollock* found that the LLMD argument was appropriate and relevant to its case, and denied the motion to strike.

Conclusions

Professional engineers providing forensic engineering and expert witness services are held to the same standard of care as professional engineers providing other services. That is, they have the duty to possess and use the level of learning, skill and care of engineers providing forensic and expert services in similar locales and circumstances, and to use reasonable diligence and their own best judgment in an effort to accomplish the purpose for which they were hired. The doctrine of witness immunity does not give the forensic engineer serving as an expert witness a free pass to violate that duty.

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Forensic Engineering: Informing the Future with Lessons from the Past
ISBN 978-0-7277-5822-4

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doi: 10.1680/feng.58224.349

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Expert Witness Training - The Practitioner's Viewpoint

Daphne Wassermann, Technical Director, Cadogans, Glasgow, UK

Abstract

This paper describes the advantages and disadvantages of different kinds of expert witness training for an engineering consultancy employing about 20 people. This training is in addition to the normal CPD[1] which an engineer undertakes in his or her own specialist field. The Introduction explains what kind of work we do and later sections describe how we meet our needs with in-house and external training. This training addresses matters of report writing, meetings of experts and presentation of evidence at hearings. The requirements of different types of staff including consultants are considered. Differing rules and procedures in various jurisdictions are also covered.

Introduction

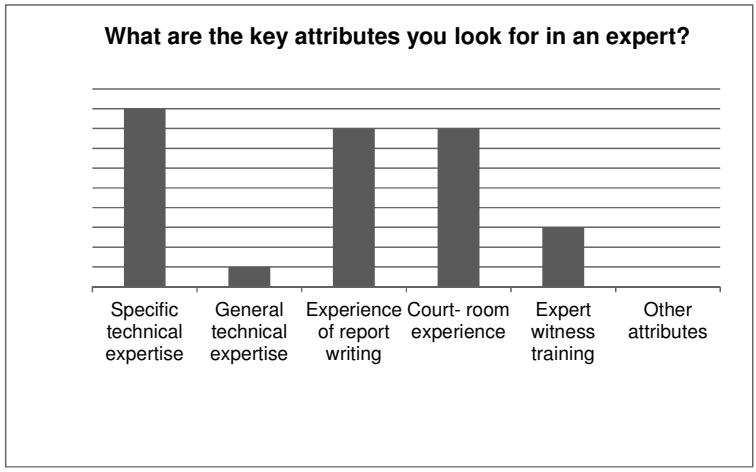
Back in the 1970s two engineers founded a consultancy, specialising in chimneys, boilers, furnaces and steelworks, industrial and fuel technology. From the start, clients asked for investigations of problems, the first being the chimney at Ninewells hospital in Dundee[2]. The work expanded and by the early 1980s clients were bringing all sorts of problems to the company. One large hospital had leaks in its medium pressure hot water system and was suing the installer. Thus began Cadogans' involvement in expert witness work.

Over the years the project work of the company expanded from combustion equipment into general building services and civil and structural engineering. In parallel, the expert witness work grew in all these disciplines. We now have a permanent staff of 15 engineers supplemented by consultants who work with us on specific matters. Although the expert witness side of the business predominates, we are still involved in project work as well.

What do our clients want?

We recently issued a survey to a small selection of our clients. We asked 'what are the key attributes you look for in an expert?' Not surprisingly, all respondents ticked 'Specific technical expertise'. Only one was interested in General technical expertise but almost all were also looking for 'Experience of report writing' and 'Court-room experience'.

[1] Continuous Professional Development
[2] Scotland



We hope that all our engineers can provide the technical expertise in their specific disciplines. They are members of professional bodies and keep up with CPD in the usual ways.

What about the report-writing and court-room experience?

In-house training

Training can be carried out in many ways, for example by coaching, more formal in-house training or by sending people on external courses[3]. We have experience of all these methods and use them in different ways at different times.

Less experienced staff

A graduate joining the company may have little practical engineering experience and no experience of expert witness work. However, they will usually be adept at ferreting information out of the internet and libraries and carrying out other kinds of research. They can be of great assistance to an experienced engineer in this way. Such a graduate will read the report that the expert writes and be coached on the requirements for such reports.

Gradually as they gain experience graduates will be able to draft report sections for the appointed expert and thus learn how to prepare their own reports.

Some of our appointments are large and complex. While a single engineer may be the Expert they will be supported by a team of staff investigating and addressing different aspects. This will include going through documents, correspondence, emails, reports, pleadings etc to discover key facts. Preparation of a timeline or chronology of events or a Scott schedule[4] is of great assistance to the legal team and can be done by someone who is not the appointed expert. It will be of assistance to the expert as well! Needless to say, the Expert must direct this work and be completely familiar with it.

Experienced engineers

The needs are different for experienced engineers who join the company but are new to expert witness work. They are experts in their fields and will be appointed as the expert fairly quickly. They therefore need rapid training in the specific requirements for an expert witness.

[3] For example those run by Bond Solon, The Academy of Experts, City University London, Professional Solutions.
[4] A table setting out information about the claim and defence.

The report format is perhaps the most obvious area. We have a standard format based on the recommendations of The Academy of Experts which complies with the requirements of CPR35 (the English and Welsh civil procedure rules for experts)[5]. It includes the basic background information then clearly separates the facts from the opinion.

Separation of fact from opinion is something that many new experts find difficult. Part of our company practice/quality assurance is to peer review reports and through this we are able to provide intensive coaching to help the new expert to marshall his or her thoughts in a logical fashion and express this clearly on paper. This process will ensure that we provide a report which explains the technical issues to an intelligent but non-technical lawyer, arbitrator or judge.

Because we prepare dozens of reports a year and get feedback on these from our clients, we are able to provide most of the required report writing training ourselves, in-house.

The new expert is also likely to have to undertake site inspections. While this might appear to be a familiar task, where litigation is anticipated it is essential that nothing is missed and everything is meticulously recorded. You are unlikely to get a second chance. You may also have to contend with a hostile site manager who did not want to let you on the site in the first place and refuses to show you the very bits that you need to see.

A novel experience will be meetings of experts. These are an opportunity to find out what information the other party to a dispute has, which may be very different from what your instructing solicitor has supplied to you. In extreme cases you may wonder whether you are looking at the same case as the other expert. Again, these meetings require diplomacy. You do not wish to be obstructive and it is important to share information. However, you do not wish to compromise your client. Further discussion of expert meetings is included below. When preparing a new expert for these meetings discussions both before and after the meeting will be helpful.

External training providers

There comes a time when input from external trainers is of advantage. They bring a broader knowledge of the world of courts and arbitrations and specific knowledge about requirements for report formats, independence of the expert, how to behave in a hearing and deal with awkward cross-examination. At this point a company needs to make the decision: do we get the trainer to come to us and provide in-house sessions; or do we send our engineers to courses run externally, probably by the same organisations.

In-house

During the course of our company history we were at one point part of the High-Point Group and worked closely on claims work with the parent company. At that point, in around 1990, High-Point arranged for an in-house session for all our engineers. This included cross-examination and for those of us (the majority) who had not been in court it was an invaluable experience.

When we had several staff who all needed training we organised subsequent courses with external trainers, both in report writing and in court-room skills.

The advantage of in-house training is that it is tailored to the company's specific requirements. The subject matter is engineering rather medicine or psychology. The course can address specific areas of difficulty and particular questions.

One question that has exercised us relates to the form of CV in the report, Should you include a generic CV in your report or one concentrating on your experience in the subject matter you are addressing? Clearly, you have to demonstrate your expertise in the particular field. But if you use a

[5] The Civil Procedure Rules Part 35, Experts and Assessors, October 2009. See http://www.justice.gov.uk/courts/procedure-rules/civil/rules/part35

different CV each time, a perspicacious barrister or advocate may latch on to this and give you a hard time. So far we haven't had a definitive answer to this question.

External courses

In some cases it may be more valuable to attend a course externally. Interaction with experts in other disciplines and sharing of experiences over a wider field will bring new insights. This may also be the most cost-effective option if a new member of staff requires training that others have already had.

I have also found the annual review run by The Academy of Experts[6] interesting and stimulating.

The benefits of these courses are disseminated by the trainee running in-house lunchtime seminars for colleagues. As well as sharing the course content, the trainee gains experience of (semi-)public speaking and informed discussion on the topic.

Web learning

A third training method is via the internet. We have used this means to provide training on CPR35 to our staff. At a fraction of the cost of attending a course or hiring a trainer to come into the office staff can gain competency in specific aspects of being an expert. They can also do it at a time that suits them.

Specialist training

The variety of problems that crosses the desk of an expert witness may well be greater than that of a designer. My background is in metallurgy and mechanical engineering. I have provided expert evidence on problems ranging from failed plumbing fittings to broken bicycles, from oil spills during demolition to personal injury from woodworking equipment or the hinge on a wardrobe door in a care home.

During the course of the investigations the expert will be learning all the time as his or her research uncovers the relevant regulations or standards and explores industry standards and the mechanics of the component. However, there may come a time when further technical training is needed.

Personal injury

As an engineering consultancy we deal with many cases of personal injury where an engineering component is faulty. Part of the investigation is pure engineering. But the case will also involve health and safety legislation. As well as finding out how the accident happened we will be asked to give our opinion on the responsibilities of the employer and employee for health and safety or regulations covering maintenance and inspection for the type of equipment. For example, we need to know about working at heights, ladders and scaffolding, confined spaces, provision and use of work equipment, manual handling. We are also asked to comment on more general aspects of health and safety such as the preparation of risk assessments. For this reason we have supported some of our engineers to do training for NEBOSH[7] certificates and diplomas, to complement their engineering qualifications.

Site work

We are often called on to inspect sites and equipment where there may be significant hazards. These might include climbing up the outside of a chimney, inspecting complex machinery or the inside of a furnace or other confined space.

Staff are therefore trained in carrying out risk assessments and for each site job the job-specific risk assessment will be checked by a colleague with health and safety qualifications. As well as ensuring the safety of staff this is a further opportunity for learning on the job.

[6] www.academy-experts.org/
[7] National Examination Board in Occupational Safety and Health

Practical skills

More specific training has also been provided, for example in welding examination in relation to the hospital medium pressure hot water system failures. We are considering future training in fork-lift truck, crane and excavator operation as many of our personal injury cases result from use of construction equipment.

Jurisdictions and processes

An expert may have to work in a variety of countries and in a variety of settings: courts, arbitration, adjudication, mediation, inquests/fatal accident inquiries. He or she may be asked to do Expert Determination.

Some aspects of the work are universal. The duty of the expert is to provide independent advice[8] and the test is always 'would I have written the same report/given the same evidence if I had been employed by the other party to the dispute?' This is not as easy as it sounds. You may be under pressure from your client to be a little more definite than you feel comfortable with. This is where the support of peers within the company can assist.

While the over-arching philosophy of being an expert may cross jurisdictions, the devil is in the detail. In Scotland we have a different legal system from in England. CPR35 does not apply. Meetings of experts are less common and you may be ambushed in court with evidence you have never seen before.

The procedures in court are different from those in arbitration. Arbitration itself varies from one country to another. Many arbitration systems are based on the UNCITRAL Model Law on International Commercial Arbitration but most are transposed into local law with some variations. The 2010 Arbitration (Scotland) Act is different from the 1996 Act covering England, Wales and Northern Ireland. If you are involved in international arbitration this could be in countries as different as Switzerland or Singapore.

An inquest in England or Fatal Accident Inquiry in Scotland will be different again.

Perhaps the most different type of dispute resolution process is mediation. As this is a collaborative and constructive way of addressing disputes the expert must also be open to the arguments of the experts appointed by other parties and seek jointly to find a solution.

Discussions with colleagues who have experience in these fora may be invaluable but you should also seek information from your instructing solicitor. It is a case of life-long learning.

A further possibility is that you may be asked to act as Expert Determiner. *'Expert Determination is a dispute resolution process in which an independent expert in the subject matter of the dispute, is appointed by the parties to resolve the matter. The expert's decision is - by prior agreement of the parties - legally binding on the parties. Like all ADR processes it is entirely confidential*[9]. Here you are the 'judge', a completely different role from the expert yet requiring your specialist knowledge.

The independence of the expert

Since the early 1980s it has been clear that the duty of the expert is to present impartial evidence to the court. Lord Wilberforce stated[10]

> *"While some degree of consultation between experts and legal advisers is entirely proper, it is necessary that expert evidence presented to the court should be, and should be seen to be, the independent product of the expert, uninfluenced as to form or content by the exigencies of litigation."*

[8] See below
[9] http://www.academy-experts.org/edrules.asp
[10] Whitehouse v Jordan [1981] 1 All ER 267; [1981] 1 WLR 246.

This was confirmed by Lord Woolf[11] and formalised in the Civil Procedure Rules part 35.3 as follows.

> **35.3 Experts – overriding duty to the court**
>
> (1) It is the duty of experts to help the court on matters within their expertise.
>
> (2) This duty overrides any obligation to the person from whom experts have received instructions or by whom they are paid.

Meetings of experts

There is an increasing trend in the UK for Courts and Arbitrators to manage a case more pro-actively and require experts to meet. The purpose of these meetings is to agree as much of the evidence as possible and thus narrow the issues between the experts. This can be an opportunity for experts to share and agree basic facts and evidence or to carry out or commission joint testing. In many cases it will become apparent that other experts have evidence that you do not know about and which might alter your opinion.

The Civil Procedure Rules in England and Wales ask for a joint statement of points of agreement and disagreement to be signed at the end of the meeting, if possible, and at a maximum within 7 days of the meeting. However, Clients and instructing solicitors like to know what is going on and are understandably nervous that such an agreement might undermine their case. Our advice to our experts, therefore, is to delay signing a joint statement until any new evidence or change of opinion has been discussed with the solicitor and client. It may be, for instance, that they are in possession of additional information which might counter the evidence provided by the other expert. It may also be necessary to ask officially for evidence that has been referred to by other experts but not handed over at the meeting.

Thus, while such meetings can be very useful, it is necessary to tread a careful line between openness and search for the truth and keeping your client in the picture. In 2011 the Supreme Court ruled that experts were not immune from prosecution if they were negligent in signing a joint statement.[12] For this reason we are particularly careful about the training that we give to our experts before they meet other experts. We would also advise them after such meetings and before the joint statement is agreed, if this seems to be necessary.

External consultants

Although our 15 engineers are flexible and versatile we cannot cover every engineering problem and discipline. We therefore work with a variety of consultants who are expert in their own fields. For these engineers, who may not be acting as expert witnesses with any regularity, training and assistance is even more important. To this end we have prepared an expert witness manual for the use of consultants and staff. Some of this covers our internal housekeeping matters but it also includes sections on carrying out the investigation, site visits, report format, meetings with lawyers and other experts and the hearing itself.

Fewer and fewer cases come to court these days: most settle before the hearing. This is not the case with arbitrations or other dispute resolution processes. Nevertheless, it is increasingly difficult to gain 'live' courtroom experience. For this reason we support both staff and consultants in providing external training in courtroom skills and cross-examination.

Spreading the word

[11] Lord Woolf, "Access to Justice": Interim Report to the Lord Chancellor on the Civil Justice System in England and Wales (June 1995) Chapter 23; and Final Report to the Lord Chancellor on the Civil Justice System in England and Wales (July 1996) Chapter 13.
[12] Jones v Kaney, 2011.

Most legal firms provide in-house training for their staff. We have assisted with this by giving seminars on engineering or health and safety topic. Subjects have ranged from slips and trips through concrete technology to nuclear power stations. This type of contact also gives us feedback from the clients as to what they require of an engineer. So it is a learning process for both parties.

Conclusion

We know from our clients that as well as technical expertise they are also looking for experience in various aspects of being an expert witness. Some of this can be acquired from colleagues and via in-house coaching and training. Some aspects require input from external training providers with specialist skills and knowledge.

A company or a sole expert witness will need to evaluate the needs of their clients and adopt a training regime that meets those needs.

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Forensic Engineering: Informing the Future with Lessons from the Past
ISBN 978-0-7277-5822-4

ICE Publishing: All rights reserved
doi: 10.1680/feng.58224.356

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Forensic engineering applied to contract management in the construction industry

SERGIO PALAZZO - Civil Engineer, MSc in Forensic Engineering
- Responsible for Business Department of Forensic Engineering in Construction - C.M.C. Cooperativa Muratori & Cementisti di Ravenna s.c.p.a.
- Via Trieste 76, 48122 Ravenna, Italy
- Phone +39-0544-428581, Fax +39-0544-428430
- E-mail sergio.palazzo@cmcra.com

ABSTRACT
This paper deals with Forensic Engineering (FE) practice in the field of Contract Management with special emphasis on infrastructure construction. The following failure case studies are discussed along with their contractual and post-contractual analyses: (1) FE applied during contract performance and prior to possible contract resolution; (2) FE applied after contract resolution or conclusion, aimed at reaching a transaction before the judge's decision; and (3) FE applied after contract resolution or conclusion, aimed at clarifying unexpected technical issues of the contract to the judge and supporting his/her decision. It is shown that forensic investigations played a key role in all case studies, by reducing costs and ensuring profitss for both the construction companies and their clients.

INTRODUCTION

Forensic Engineering investigates the most likely causes of discrepancies between actual and expected performances, as well as the associated responsibilities. To this end, inverse methods are applied to identify the causes of such discrepancies, starting from their effects. In other words, to identify the initial conditions of the problem, by looking and studying its final conditions (Augenti, 2008). Assessing an expected performance must inevitably be done within a legal context which includes standards, contract rules and codes which regulate relationships between people, government and local authorities, institutions, etc. Such a legal environment calls for both technical and legal skills. General infrastructure construction companies operate within a framework of specific contracts signed with their clients, which are subject to detailed codes at both national and international levels. It is essential that both the general contractor and its client should carry out their respective activities by following a suitable management plan (namely, Contract Management), which is mainly designed to optimize business. If management performance does not meet expectations, the reasons for the failure caused by the company, its client or third parties, should be investigated by means of Forensic Engineering methods based on technical, economic and law principles. It is well known that forensic engineers work in either a civil or criminal procedure as technical consultants for parties or judicial authorities. Therefore, Forensic Engineering can be connected to Contract Management through a contractual performance.

This paper is aimed at analyzing how Forensic Engineering applies to Contract Management by means of three failure case studies in the field of infrastructure construction contracts:

1) Forensic Engineering applied during contract performance and prior to possible contract resolution;
2) Forensic Engineering applied contract resolution or conclusion, aimed at reaching a transaction before the judge's decision; and
3) Forensic Engineering applied after contract resolution or conclusion, aimed at clarifying unexpected technical issues of the contract to the judge and supporting his/her decision.

For each case study, contractual and post-contractual analyses are discussed.

CASE STUDY #1: FORENSIC ENGINEERING APPLIED DURING CONTRACT PERFORMANCE

Contract performance is very important because in the case of litigation the parties are exposed to the risks of the contract resolution process and compensation claims. Anyone who signs a contract wants to obtain a profits by delivering the full performances required in the contract. From an industrial point of view, it is advisable to invest resources in the contractual phase, even if a great amount of money is needed.

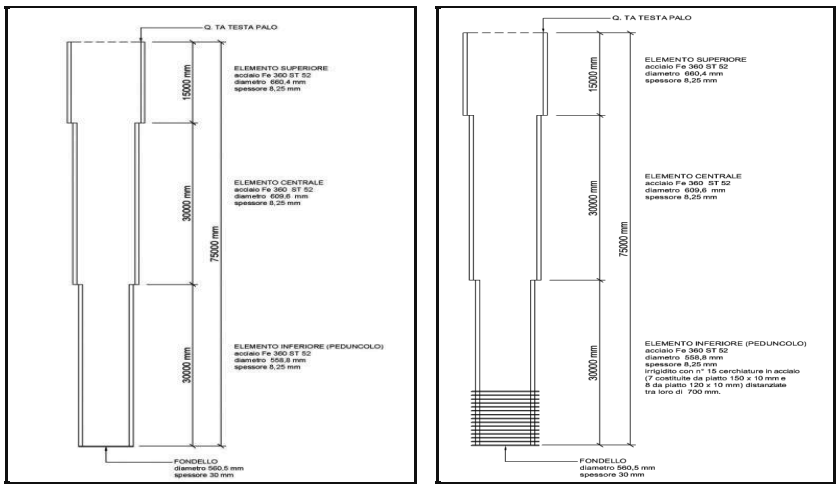
This can reduce the risk of negative effects on production (e.g., longer production time, disorganization of construction site, increased company workload, uncertainties about the future progress of the contract) resulting from a civil litigation. In the following, a case study where forensic engineering was applied to a contract litigation case between an infrastructure construction company and its client during construction is discussed. Forensic Engineering applied to Contract Management allowed for the completion of construction, thereby ensuring the expected industrial benefits.

This case study resulted from a contract relating to the construction of a cargo building in an Italian airport. A deep foundation system composed of impact driven, hollow steel piles filled with concrete, was designed.

The piles were 75 m long, had three varying diameters along the depth, and were without reinforcement. According to the contract design solution, only the steel cross-section contributed to the pile load-bearing capacity. Nevertheless, the general contractor verified the structural design documents produced by the client, and highlighted the fact that the piles designed were not adequate to support the building. Therefore, the client redesigned the building foundation by increasing the number and diameter of steel piles and by adding a structural rebar (see Figures 1a and 1b).

Technical building codes included in the contract provide for pile penetration with multiple impacts produced by a Delmag type pile driver down to a depth of 70 m. The piling had to be stopped after rejection, i.e., 20 strokes of a Delmag A36 type pile driver for a 100 mm (refusal) penetration measured over a distance not deeper than 500 mm. When carrying out piling operations in accordance with contract rules, significant difficulties were found in the soil crossing; this was caused by the excessive strength of a sand layer at a depth of between 6 and 16 m. The presence of sand caused piling inability due to premature rejection. The general contractor informed the client of these problems, and proposed the following new solution:

disintegration of intermediate sand layers by high-pressure water jets and stiffening of the pile base using steel hoops with continuous welding.



(a) **(b)**

Figure 1. Comparison between pile foundation design solutions.

The client did not modify the original contract documents to allow for new and more expensive works, without any respect for the contract documents, because the construction issues were caused by unexpected geological conditions. The client believed that the new construction method should be included in the general contractor's costs, without consideration of the increased working costs (i.e., water-based disintegration). The general contractor was then forced to make a contract option and to apply for a preventive judicial procedure in order to demonstrate that the original contract piling method was not feasible. Based on the Italian Civil Procedure Code, parties can apply for a preventive judicial procedure in order to seek a technical opinion on issues that could lead to litigation. In this case study the technical consultant of the law court reached his conclusions after *in situ* realization and testing of both the formerly-designed and subsequently proposed piles. In the light of experimental results, the technical consultant concluded that the design piling method, without pile base stiffening and water-based disintegration, did not allow the expected depth to be reached, given that the result was either a premature rejection, or pile collapse. The client did not cancel the contract, so as to avoid the beginning of litigation and acceptance of additional costs resulting from the new construction method. This situation was resolved thanks to the intervention of the forensic engineering.

This case study shows that the technical adviser of the court, a forensic engineer, has a good knowledge of geotechnics, foundation engineering, as well as the ability to carry out inspections of the facilities. Furthermore, the technical advisor showed an understanding of the content of the contract, and was therefore able to explain to the judge the reasons why the work had to be carried out.

CASE STUDY #2: FORENSIC ENGINEERING APPLIED AFTER CONTRACT RESOLUTION OR CONCLUSION AND PRIOR TO THE JUDGE'S DECISION

This case study falls within the 10 years guarantee phase which is mandatory for general contractors to ensure the quality of their work after construction, in accordance with Italian Civil Code. A transaction was accepted by both parties after construction, so litigation was avoided. A negative decision could reduce or even cancel economic profits after construction for both the general contractor and its client (e.g., compensation, legal costs, image damage to general contractor in its own market).

This case study deals with a contract for the construction of an agri-food market consisting of 12 buildings with specific water-proofing systems. Some of those buildings were composed of multiple structures and had thermal insulating systems. In particular, the project included the realization of a thermally-insulated roof as follows: "*Realization of a roof with bituminous thermal insulating system composed of: (1) non-woven fabric polyester layer with 200 g/m^2 unit weight; (2) vapor barrier made of 0.25 mm thick polyethylene sheath cast in dry conditions with 100 mm overlapping and double-sided sealing tape; (3) extruded polystyrene, thermal insulating panels with 50 mm thickness and 38 kg/m^3 unit weight; (4) non-woven fabric polyester layer with 200 g/m^2 unit weight; (5) lightweight concrete slab with 800 g/m^3 unit weight, 60 mm thickness, and squared rebar having 4 mm diameter and 200 mm spacing; (6) coating of high-penetration bituminous primer; (7) first polymeric bituminous water-proofing membrane reinforced with 3 mm thick polyester; (8) first polymeric bituminous water-proofing membrane reinforced with 4 mm thick non-woven fabric polyester and laid with misaligned joints over the first membrane; and (9) protective spray gun.*".

As he was opposed to the contract documents, the client changed a part of the thermal insulating and water-proofing system, inducing some risks during construction and service life. That change consisted of the insertion of a lightweight concrete slab between the thermal insulating layer and the water-proofing sheath, which could result in detachment due to vapor pressure connected to the mixing water, as well as out-of-control cracking. Given the low unit weight of the layer (48 kg/m^2), its weighting function was unsafe in boundary zones. The client redesigned the water-proofing system by eliminating the lightweight concrete slab and also replaced the polystyrene thermal insulating layer by using expanded perlite panels, in order to preserve walkability. A slate-based sheath was used instead of protective coating in order to reduce maintenance costs. The thermal insulating and water-proofing system also ensured a weighting function through a total adherence connection between layers. Ten years later that design solution was found to be inadequate for those large roofs (see Figures 2 a, b, c).

This led to litigation between the general contractor and its client. The latter accused the general contractor of bad construction. However, the general contractor stated that the construction was carried out in accordance with project documents, but some errors were identified in the new design documents prepared by the client. A preventive judicial procedure was also activated by the law court in that case, and the technical consultant of the judge decided to perform some tests. Thermal conductivity and compressive tests were carried out on expanded perlite panels, whereas tensile tests were carried out on bituminous sheath specimens.



(a) (b) (c)

Figure 2. Failure of waterproof system.

Such tests demonstrated that the materials used by the general contractor were in accordance with the project documents, but a negative trend was found in test results for degraded specimens.

The technical consultant concluded that seepage and degradation of roofs were caused by a number of factors associated with both the construction methods used by the general contractor, and the incorrect design solution of the client. The use of expanded perlite as the thermal insulating layer was not suitable for large roofs because high temperature variations and the moisture absorption capacity of perlite rapidly reduced the water-proofing capability.

Based on the technical consultant's conclusions, both parties reached an agreement and thereby avoided legal costs and other effects. It is important to stress that the general contractor activated a 10 year insurance policy after construction to cover its own quality construction liability. Such costs were then avoided by the general contractor.

The study described above shows that the forensic engineer appointed by the court chose to carry out the correct tests on the waterproofing and insulation of the roofs of the buildings examined. The knowledge of the materials and the correct reading of the draft contract enabled the judge's consultant to understand that the perlite was the real problem with the waterproofing. The consultant's expertise enabled the judge to understand what the responsibilities of both the company and its client were.

CASE STUDY #3: FORENSIC ENGINEERING APPLIED AFTER CONTRACT RESOLUTION OR CONCLUSION TO CLARIFY TECHNICAL ISSUES TO JUDGE

This case study deals with post-construction problems of a highway tunnel which led to litigation. Some failures during excavation occurred in the initial part of the tunnel. A partial cross-section excavation method was used, but it was found to be inadequate for soil layers to be crossed (see Figure 3).

After those failures, the client made a new design based on full cross-section excavation method and signed a new contract with the general contractor. That change consisted of (1) a direct excavation of the tunnel without piers, (2) soil strengthening beyond the excavation front, and (3) a rigid pre-covering close to the excavation front.

The effectiveness of such a construction method was checked in a number of underground structures which had been built in the past in very complex

geotechnical conditions. A further failure took place during construction: the collapse of the excavation front, vault and part of soil embankment back to the front.
As a result, a 20 m long crater formed on the ground surface and a 10 m long soil distress failure was detected. This produced a total 30 m long ground failure with heavy damage and relevant geotechnical problems (see Figures 4a and 4b)

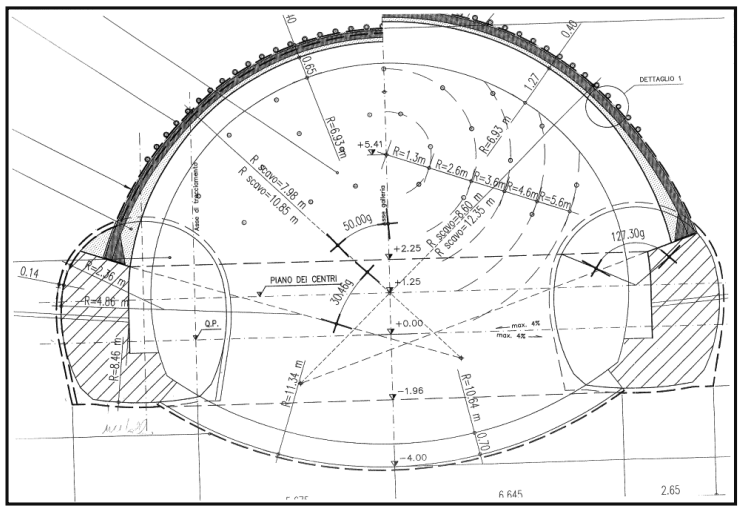


Figure 3. Partial cross section excavation method.



(a) **(b)**

Figure 4. Soil debris and distress failure.

Complex remedial interventions were carried out by the construction companies to support the tunnel roof. The general contractor claimed reimbursement from its insurance company, in accordance with the insurance policy contract against any construction risk, but it was rejected, so a private arbitration was initiated. The insurance company pointed out that the failure occurred after front excavation, and hence there could be no reimbursement. In addition, no permanent covering of the excavation was carried out. The technical consultant of the parties and arbiter decided to use the ADECO-RS *"Controlled Deformation Analysis – Rocks and Soils"* (Lunardi, 2000) analysis procedure which is based on finite difference method

(see Figures 5a-c). In fact, the main design principle of this construction method is played by the soil core in progress, which is located beyond the excavation front and provides both short- and long-term control and stability and so affects the structural behavior of the whole tunnel. Therefore, the soil core in progress is an active part of the tunnel. All remedial interventions are carried out in this context; especially those aimed at upgrading materials as the excavation continues (see Figure 6). Given that the general contractor carried out remedial works, its activity was aimed at reducing damage and further potential failures. Forensic engineers were able to define reasons which conflicted with those of the insurance company, showing that tunnel excavation is work which is entirely covered by the insurance policy. Finally, forensic investigations made it clear to the arbiters that the general contractor was in the right. In fact, the general contractor managed the insurance policy correctly.
It is important to point out that the Board of Arbitration's technical consultant, a forensic engineer, showed considerable knowledge of tunnel geomechanics and of insurance contracts. It is for these reasons that the consultant was able to provide the judges with scientific explanations as to the company's actions.



(a) (b) (c)

Figure 5. Remedial works and numerical model of gallery.

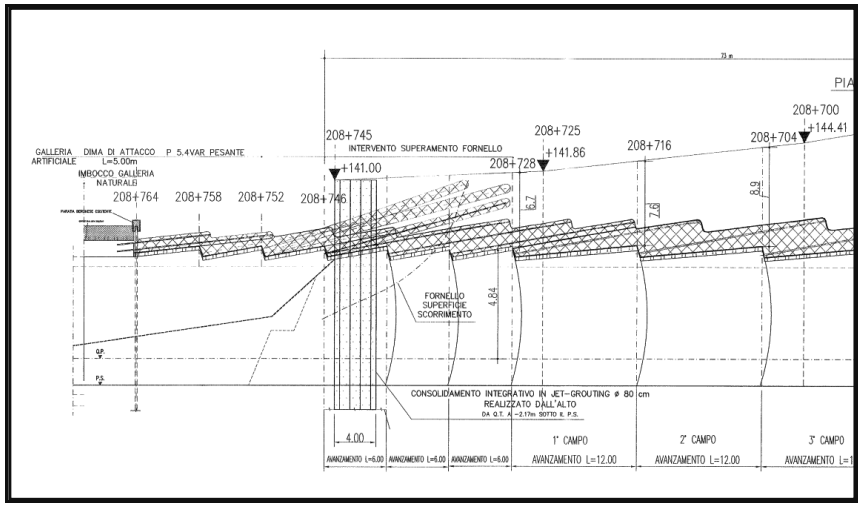


Figure 6. Remedial intervention scheme.

CONCLUSIONS

The analyses which resulted from the 3 reported cases of forensic engineering are due to the close relationship between legal and engineering expertise. Indeed, as we have seen, the complexity of the causes which characterize litigation, and the identification of these causes, can not be separated from their legal or contractual effects on the contract management of a company. Therefore, the study of litigation concerning the contract management of infrastructural works involves both legal and engineering disciplines.

Furthermore, *"an interdisciplinary knowledge of both subjects is required for the correct application of forensic engineering in legal contexts"* (Fiorani, 2008).

We know that the reconstruction of the legal/contractual consultancy, and the expert/technical content, are among the most complicated aspects in the area of procedural studies. This is further true due to the fact that there are those who claim, according to classical conceptions, that these consultations and expertise are a means by which the judge can integrate the scientific aspects. Less traditional thought, however, considers them to be evidence.

The truth is most likely to be found in how the consultant or expert acts in each specific case. *"If the truth remains the ultimate goal of the trial, then this must be ascertained using methods, procedures and interpretations which are both technical and legal, and which must be confirmed or rejected in order to arrive at a 'single' truth"* (Pierbon, 2008).

Any error evaluation by a court consultant during litigation involving company contract management results in extensive damage to the economic management of the company, as well as to the customer that commissioned the work.

With this paper we have shown how expert engineering knowledge, applied to contractual analyses, has led to the identification of rights of the parties and to equitable solutions on their behalf (transaction, just sentence, proper compensation etc).

We can therefore say that nowadays, considering the Italian legal system (and even more so, the European legal system), that technical consultants used by public and private institutions are required to have precise knowledge of the basic principles of the law, as well as trial and procedural rules. These rules are essential in guaranteeing the validity of the work of these experts and consequently the protection of rights, and of companies which are both components of a civilized society.

We think it appropriate to finish this paper by referring to a passage of Aristotle's "Etica Nicomachea" (Book 5), according to which a *corrective justice* similar to an arithmetical proposition exists, "*whose responsibility it is to balance the advantages and disadvantages in the contractual relationships between men, be they intentional or not*".

This last sentence illustrates how, in Ancient Greek philosophy, equity was more than a mere secondary source of law: it was an important virtue central to the moral development of a good man. Aristotle, Pythagoras, Leonardo da Vinci and many others, are figures who have contributed to establishing the fundamentals of law and engineering over the course of human history.

As engineers we have a lot to offer and it is our duty to use our expertise and experience to make the world a better and fairer place to live.

The cases of Forensic Engineering presented above, together with all the other cases in the Conference, form the basis of the spirit of this ICE Congress "*Informing the Future with Lessons from the Past".* Reviewing cases from the past is the way to understand today's cases, and to prepare for those of tomorrow.

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Forensic Engineering: Informing the Future with Lessons from the Past
ISBN 978-0-7277-5822-4

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doi: 10.1680/feng.58224.365

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Prevention is better ……..how insurers try to reduce failures

Eur Ing **Richard Radevsky** BSc, CEng, CSci, CEnv, IntPE, FICE, FCIWEM, MEI, MIFireE, FCIArb - Technical Director- Charles Taylor Consulting plc, UK.

Insurers and influence

Most major engineering projects are the subject of insurance covering the construction, the liabilities of the parties involved, the professional liabilities of designers and sometimes the profits that the project is anticipated to generate soon after completion. Completed works carry property insurance and cover for the liabilities of those who own and operate them. Clients, contractors, consultants and operators of projects and property have to spend considerable sums to obtain insurance so risk managers usually appreciate the need to create and maintain a good impression with insurers. A bad relationship with insurers or a poor loss record can result in higher than normal premiums or restrictions on the availability of cover. In extreme circumstances non-availability of cover can kill a project or restrict what a contractor or consultant is able to do. Day-to-day the influence of insurers takes many forms. The primary opportunity for this is at the underwriting stage when terms are being negotiated but increasingly insurers want to be proactive during the life of a policy to improve risk management post-underwriting.

Rarely is the influence of insurers on safety or risk management publicised. When a highly publicised failure occurs senior managers will be looking to reassure investors that the company is fully insured. Insurers have to tread a fine line in exerting a positive influence whilst steering clear of becoming an extra checking authority, a designer or construction advisor. It is important that contractor, consultant, client or owner makes their own decisions and takes full responsibility. Insurers are keen to be seen to add value by offering advice and bringing to a project a new perspective which differs from the perspective of those whose

main aim is to complete it on time and on budget. Some project decisions have major cost implications and on occasion, provided a disaster is avoided; significant savings can be made by consciously adopting a high risk approach. Contractors can win competitive tenders by identifying shortcuts which others may have missed. Insurers are keen to avoid being used to cover the downside of such situations whilst others benefit if a shortcut succeeds. They are however conscious that construction involves taking risks and that they cannot impose such tight restrictions that normal construction activity is impossible. A reasonable balance is required. Since insurers cannot dictate how a project is managed day to day they have to use more subtle methods to ensure that sound risk management practices are deployed.

Insurance is not essential for a major engineering project. Its absence does not physically prevent works from being undertaken but today in the western world it is almost impossible to attract the finance necessary to allow a project to start unless appropriate insurance policies are in place. Those providing the finance often specify what types of cover are required and may also place restrictions on the credit rating of the insurers that can be used. Providing finance for a major engineering project exposes the lender to risks and the lender will want reassurance that if certain undesirable events take place the resulting cost will not affect them or only affect them to a limited extent.

Obtaining insurance for a major project often involves multiple complex negotiations between the insurer and the purchaser mostly represented by a broker with expertise in insurance. For large projects cover is usually provided by a number of insurers each taking a percentage of the risk often supported behind the scenes by reinsurers. None of the insurers or reinsurers is forced to provide cover. They do so in the belief that they can make profits not only from paying less in claims than the premium but also from investing the premium. In fact the state of the financial markets and returns on invested premium often have more of an impact on insurers' profitability than the skill of their underwriters. Successful insurers carefully select what they underwrite and decline to insure projects they judge are unacceptable. There are limits upon how fussy an insurer can be. If an insurer wishes to remain in the business of underwriting major construction projects, repeatedly refusing to provide cover or only quoting very high premiums holds risks for their business. An insurer can earn a poor reputation which might result in brokers not presenting potential business to them.

Obviously the situation is more complex than just saying yes to some risks and no to others. Insurers can sometimes negotiate the price for insurance cover, the terms under which the cover is given and even the nature of the risk that is to be covered. If a proposed project creates a high level of risk, it may be difficult to find any insurers who are prepared to underwrite it or the premium quoted may be prohibitively high. In this case, the choice for the

promoter of the project is to abandon the project, to modify it to make it less risky or to proceed without insurance.

Underwriting

Before an insurer is prepared to give cover they want to know a great deal about who is involved, how works are to be undertaken, the risks to which the works will be exposed and the methods being used to manage them. Policy terms and the price of insurance are adjusted depending on the information they receive. Risks to be insured are often the subject of a presentation (frequently prepared by brokers) and declarations in a proposal form. To obtain the most favourable insurance terms emphasis is often placed in presentations upon the experience, competence and capability of the parties involved and the procedures that are to be used to manage risks. Underwriters rely on these statements in making underwriting decisions and therefore expect that what has been promised will be delivered. If it is not, then questions of non-disclosure or misrepresentation may arise when claims are made since insurance contracts are based upon utmost good faith which requires that risks are accurately and thoroughly explained. This applies not only when the policy is taken out but also during the life of the policy. If a material change in risk occurs, the insured must notify the underwriter to give them the chance to alter the terms of the policy mid-way through its life if they consider it necessary.

Coverage is subject to compliance with policy conditions. These can place many types of obligation upon the parties insured including the need to take specific precautions to avoid loss or limits on the extent of particular types of works insured or on their value (for example how goods should be stored in relation to flood risk or what length of trench can be left open at any point). Insured parties can choose whether or not to comply with these conditions but, if they breach them, they may find coverage is denied when a claim is made. Insurers are likely to expect surveys to be undertaken from time to time to check how matters are progressing and to see if improvements need to be recommended. Claims are subject to close scrutiny and when policies come to be renewed, the claims history is taken into account before determining renewal premiums and policy terms. Insurers operate in a competitive market which places limits on how much they can charge and how wide or narrow coverage can be. The ability of insurers to influence risk management depends on how much competition there is between insurers to underwrite policies which in turn depends upon the cyclical insurance market. The situation is therefore constantly changing.

Design codes and construction methods

In deciding whether to insure a project, and if so at what price, insurers will look at the design standards that are to be used (the appropriate building code and design standard requirements

related to earthquake, hurricane, fire and explosion protection measures appropriate for a particular process). Promoters of projects often have the option for their project to comply only with the minimum legal requirements or to use better standards which will result in a higher construction cost. In many countries local legislation is not well developed for specialist projects. If the promoter of a project can see the extra premiums they would have to pay for a construction policy and subsequent operational insurance or they are told they will be subject to restrictions on cover, they may be motivated to accept the extra cost of complying with better design standards. Insurers can have some influence on decisions associated with selecting different levels of risk which may otherwise be difficult to quantify. Occasionally promoters will appoint risk engineers to be involved at the design stage of a project to look at what design changes may reduce the level of risk that will in future affect the cost of operational insurance once a project is complete. Some insurers, with an interest in providing both construction and post-completion operational insurance, will set their own standards for a plant that must be followed during the design and construction stages. Factory Mutual, for example, produces its own standards for a number of industries which are focussed on loss prevention during operation. Whilst complying with such standards sometimes increases construction costs, these may be offset by fewer or less severe losses and possibly correspondingly lower insurance premiums in future. The level of risk posed by a project and thus the price and availability of insurance can be substantially influenced by the method of construction to be used. This applies particularly in certain types of project such tunnelling or heavy earthworks, especially under or near water.

Sometimes insurers will be consulted before policy coverage is taken out or they will be asked to provide cover before a contractor has finalised the method to be used. Sometimes contractors may tender significantly different prices for the same project or sub-project because they have selected different methods of construction. If an insurer provides cover before a subcontractor and a method of construction has been finalised the policy may be made subject to specific conditions to limit insurers' exposure. Conditions can be financial, such as sub-limits of cover for particular types of loss or restrictions on what a contractor can do (such as a limit on how great a length of partially completed road or open pipeline trench can be exposed at any one time). Sometimes contractors will approach insurers for cover for works they are about to commence and the reaction of insurers may persuade promoters to accept a variation to works which lowers the level of risk. A typical example of this is where a cofferdam has to be built and a decision is required about its height which is linked to the return period of the flow which it is designed to resist. If a contractor finds it difficult to obtain insurance cover for the height which a promoter has approved, the promoter can sometimes be persuaded to reconsider the design parameters resulting in a cofferdam able to resist an event with a significantly greater return period than that original intended.

Case history

Between 1998 and 2002 the losses experienced on tunnel projects were so great that insurers came close to withdrawing cover altogether from this type of project. Such an action would have brought new tunnelling activity to a standstill. Techniques used to manage tunnelling risks were inadequate across the tunnelling industry and it had to change its approach. Changing an entire industry is never easy but insurers were able to apply concerted pressure. Insurers and tunnelling engineers worked together to produce a risk management standard[1]. Cover was then made available on condition that the standard was applied. This had the effect of reducing the number and size of tunnelling losses bringing tunnelling risk down to a sustainable level.

Professional Indemnity cover and quality control

For complex projects, insurers will be looking to see that an experienced and competent design team and an experienced contractor are to lead the project, with sufficient resources to cope with problems that may arise during construction. Where insurers are being asked to quote for Construction All Risks insurance, the underwriters often take a great deal of interest in the Professional Indemnity insurance arrangements of the designers and contractors. This is not only to ensure that the costs of a design problem would be insured but also because the presence of adequate Professional Indemnity insurance is a test of the designer's/contractor's quality. If designers/contractors are able to obtain Professional Indemnity insurance with a significant sum insured, this indicates that they have been scrutinised by a Professional Indemnity insurance underwriter who is unlikely to provide cover unless the designer's/contractor's methods, procedures and claims history are of an acceptable standard. Insurers therefore provide direct quality control feedback.

Reality check

Where underwriters refuse to provide cover or quote a very high price, this can set off alarm bells within the promoter's organisation where, up to that point, the level of risk may not have been properly appreciated. If a promoter finds that what they propose cannot be insured, questions will be raised at senior management level. This can lead to major changes driven not only by the problem of arranging insurance, but also by the realisation that what was proposed was unacceptably risky. Insurers therefore sometimes provide a risk-reality-check before a design is complete or construction has started. Since experienced insurers and reinsurers have a wealth of knowledge of projects and the losses they can create, they are often better placed to make risk assessments than project promoters. Before an underwriter is approached, an experienced and knowledgeable broker can often predict whether a project in a particular form will be uninsurable or attract an unusually large premium.

Case History

An oil refinery being upgraded in Africa included a number of process units including one that had suffered significant corrosion. Underwriters were happy to provide cover for the bulk of the refinery but excluded cover from the unit suffering from corrosion problems. This simple approach of excluding part of a plant from coverage allowed underwriters to deliver a stark message to the refinery owner. All units in the refinery did not have the same risk profile. The problems in one unit were so severe that it was uninsurable. This crystallised the refinery owner's options: solve the problems with the corroded unit, shut it down and dismantle it, or operate it without insurance.

Risk surveys and risk engineering during a project

Risk surveys[ii] provide an opportunity for an expert appointed by insurers to review the project in detail from a risk perspective. Risk engineering[iii] is a similar process but involves regular visits during the life of the project. In both cases an opportunity is provided for insurers to be updated on details of the project, for their expert to check how risks are being managed and to make recommendations for improvements. Project managers and staff generally welcome an external review.

Case History

On a major European railway project, a risk engineering survey identified that different checking procedures were applied to different types of temporary works (such as scaffolding and erection gantries). The criteria for selecting which checking method was to be used were not well defined. Procedures were complex because of the number of checking bodies. Over a series of risk engineering surveys, procedures were clarified and defined. Between surveys meetings took place between the lead checking authority and a number of contractors. Uncertainties and ambiguity in checking procedures were progressively eliminated thereby reducing the risk that an error could adversely affect the stability of surrounding construction, safety of third parties and rail traffic.

The primary purpose of the process is the prevention of losses by examining the performance and progress of the construction works, identifying key areas of risk, providing recommendations, analysing losses and sharing lessons learnt with the operational teams. This is achieved through regular visits to site, and the discussion of recommendations with the Insured. Sometimes those involved in the construction are already aware that improvements need to be made but may find it hard to attract the funding or corporate will to make changes.

Case History

At a power plant in the Middle East a large diameter gas main was located beside a busy highway with no crash barrier to prevent a vehicle impact puncturing the pipe. Although the project was aware of the danger, funding for a barrier was not made available and no action was taken to install it until it became the subject of a risk engineering recommendation. Risk engineering recommendations are generally circulated at senior management level and come from an independent external source. They are therefore difficult to ignore particularly as they explain why and recommend how quickly action should be taken. In this instance an internal logjam was broken and action taken faster than would have otherwise been the case.

Risk engineering recommendations can cover a wide range of subjects including:

- general site safety
- fire safety including fire systems design, construction and operation
- risk registers and risk management administration procedures
- design checking procedures for permanent and temporary work
- preparedness for adverse weather and other natural perils
- business continuity, contingency planning and emergency pre-planning
- project organisation and management
- co-ordination with other projects, agencies, local authorities, emergency responders and nearby sites
- compliance with insurance policy conditions
- management of interfaces with third parties
- procedures for dealing with multiple languages
- stability monitoring and control of the site and its surroundings
- testing and commissioning procedures
- security measures and resources
- management of storage and laydown areas
- work permitting procedures
- configuration of installations
- handling and control of hazardous substances
- quality management
- management of subcontractors and suppliers
- site accessibility
- movement procedures for unusual loads to site and on-site
- heavy lifting
- bonus systems and incentives
- incident and near miss reporting systems

Case History

On a tunnelling project in North America tunnelling crews were found to operate in 4 different languages creating communication problems since instructions were prepared only in English. Following a risk survey recommendation, instructions and safety orientations were translated so that all members of the crew could properly understand them. The risk survey had identified a problem which had been overlooked by senior management and tolerated by working crews. On a day to day basis it was possible to overcome the communications problems but in a crisis a delayed or mistaken communication can make the difference between a near miss and a disaster. The risk engineer, with experience of investigating previous disasters, was able to explain why improvements were necessary and thus stimulate managers to make changes.

Occasionally risk reviews may be conducted during the design stage of a project looking not only at the way that the project is to be constructed but also at features that may be incorporated into the project which will have an impact on the insurability of the works once complete. Designers and contractors sometimes look at risks only as far as their involvement extends. Owners of projects are concerned about managing and insuring the project once it is complete and is operational.

Questions are often raised about the status of risk engineering recommendations. Is the project required to follow them and what happens if they are ignored? Risk engineering reports with recommendations normally make clear that recommendations are for the project to consider since they are not instructions. Responsibility for deciding about changes remains with the project. Unless something is identified with immediate safety implications (such as a release of gas into a hot work area which may lead an underwriter to warn that should a subsequent loss happen it would not be sudden and unforeseen and thus not covered by the policy) ignoring the recommendations will have no immediate consequences. Insurance coverage is defined by the insurance policy and not the risk survey report. However most projects will not want to ignore an opportunity to improve their risk management but they may find an alternative solution to that suggested in a recommendation.

Risk analysis information

Many insurers make available, free of charge, publications on technical and engineering risks. These are usually generated from research commissioned by insurers and reinsurers and insurance trade bodies which will often include analysis of past failures. Major providers of such information include Munich Re, Swiss Re, Factory Mutual and the International Engineering Insurance Association (IMIA). Such information is frequently used by insurance underwriters from different companies to help them understand the risks they may be asked to underwrite. It also helps those involved in construction to understand risks associated with different types of construction and explains what has gone wrong in the past. Information on failures can be extremely useful if lessons are to be learned but specific information on

individual failures is rarely freely available since those who suffer losses usually do not want adverse publicity. Insurers however are able to extract the valuable technical details of the cause of losses reported to them and also compare experience from numerous projects to see if there are common failings.

Being realistic about claims

Anyone who has been involved with insurance claims will be aware that insurers are not an easy source of money. If there is doubt about the cause of a loss, the value of the claim or the application of a condition or exclusion, then insurers will undertake a very thorough investigation. Inevitably there are claims where an insurer believes there is no cover and an insured believes there is. This will usually result in negotiations and discussions with insurers during which positions will be clarified but agreement is by no means guaranteed. Mediation, arbitration or litigation may follow until the matter is settled. Since insurers are generally commercial companies with shareholders, they cannot readily pay claims that are not covered since they must look after their shareholders' money. They will also often have purchased reinsurance and if they pay claims that are not covered they will face difficulties recovering from reinsurers. Occasionally there are commercial reasons why an insurer will pay a claim that is not covered or will give the benefit of the doubt to the insured, but it would be wrong for an insured to rely on this. Insurers have a reputation for being tough. They are, however, conscious of the need to avoid a reputation for being unfair. They operate in a commercial marketplace and if they do not pay legitimate claims, it will not be long before customers look elsewhere for coverage.

Those experienced in dealing with insurers develop realistic expectations about how insurance claims will be dealt with. By playing down unrealistic expectations insurers are trying to avoid situations where a company will take risks in the knowledge that they are insured that they would not take if uninsured. Insurers expect their clients to behave as a "prudent uninsured". The consequence of this approach is to force companies involved in taking risks to exercise proper care with risk management and to put in place all reasonable measures to avoid losses. If a loss happens despite the exercise of proper care, insurers are there to meet the cost but they are not there as a substitute for proper care.

The role of the broker

Buyers of policies want to be sure they get value for money but will also be supported properly if they have a loss. Inevitably each insurer behaves differently and since claims arise well after a policy has been purchased, without experience it is difficult to tell whether an insurer offering a low premium should be taken up on their offer or might pose problems in future. Judging whether an offer of cover is value for money is not easy. The terms of cover

on offer may vary between insurers. Insurers with different credit ratings also charge different premiums. A buyer needs to know whether it is worth paying more to have higher rated insurers. Buying insurance for a major project requires knowledge and experience.

Brokers, who are often remunerated through brokerage fees, have a duty to look after their clients' interests and can be very useful for insurers since they can explain why it is in the buyer's interest to spend extra money on a particular type of cover or a policy from a particular insurer. Although insurers can develop very sophisticated insurance products or devote great efforts to building up a reputation for fairness and reliability, there is little point in doing this if insurance buyers do not appreciate why these should influence their buying decisions. Brokers can also influence an insured's approach to a project before it is presented to the insurance market. By using their experience, they can identify, at an early stage, what is likely to make a risk unacceptable to underwriters or cause them to charge high premiums.

Case History

For a tunnelling project, the financiers were keen for the contract to be on a fixed price basis thereby putting all the tunnel risk on the contractor. Insurers are often wary of fixed price contracts for tunnelling work since this can encourage contractors to look for short cuts if they run into problems. Knowing this, the broker persuaded the client that they should contractually accept the risk of increased tunnelling costs caused by unexpected ground conditions. The broker anticipated problems in placing with Underwriters the project insurance if a fixed price contract was used and was able to use this concern to counter the financier's desire to push as much risk as possible towards the contractor.

Brokers can advise how to modify conditions and structure of coverage to reduce the cost. For major projects they will also prepare the presentation given to prospective underwriters which will be used by the underwriter to decide whether to offer to insure the project and if so upon what terms. Considerable care is needed in this exercise since if an insurer, facing a claim later, feels he/she was misled by the information provided at the underwriting stage, he/she may deny the claim. Brokers are therefore important in ensuring that any temptation to conceal the riskier aspects of a project or mislead an underwriter is resisted. Brokers will also know which insurers are likely to respond positively to particular types of project since insurers have their own business strategies. Brokers therefore have an important role to play in helping many insurance markets work effectively. By being knowledgeable about the inter-relationship of risk and the cost and availability of insurance, they can advise their clients at an early stage about the likely consequences of the choices clients have to make.

Conclusion

By putting a price on risk, insurers assist companies to make commercial decisions on a sound basis. In the relatively recent past, insurers used only the price of insurance or restrictions placed on the cover to signal that a project was perceived to be highly risky or poorly risk managed. After a policy was issued, insurers would only become visible to a project once a claim was made. Today, insurers use a variety of different ways to influence those involved in construction projects to improve the way they manage risks. They engage specialists to review how a project is being managed and make specific recommendations for improvement. They are a valuable source of information about what can go wrong and most try to disseminate this through publications so that all can learn from past mistakes. Some define their own technical standards or are instrumental in promoting risk management standards and codes prepared by industry bodies.

Whenever a promoter or contractor asks "how will this proposal affect our insurance premium?" or "does our subcontractor or consultant have proper insurance cover?" or "does our insurance cover us for this activity?" insurers are exerting their influence. Having to ask these questions may be annoying but the process that the questions generate is a useful one. Insurers are known to be knowledgeable, thoughtful and careful with the risks they will accept and this makes those they insure think carefully about risks before they act.

When losses do occur, by dealing with claims in a firm but fair way, insurers dispel the notion that insurance can be used as a substitute for proper risk management. If risk management were ever to reach to such a sophisticated level that losses could be eliminated, no one would need insurance. We are some way away from that situation but it is possible to reduce the severity and frequency of losses through ever improving risk management. Insurers have a valuable and increasingly active role to play in bringing this about. Those undertaking construction projects can sometimes take some convincing that their insurers are enthusiastic to help improve the risk situation. As relationships develop, however, the mutual interests of insurers and those they insure in avoiding losses should foster a spirit of co-operation. Where losses are avoided altogether and a project finishes ahead of time, there are often bonuses and other financial rewards available for contractors and others. Not only that, but a good record from past projects encourages insurers to want to participate in the next project and thus impact the premiums charged.

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Forensic Engineering: Informing the Future with Lessons from the Past
ISBN 978-0-7277-5822-4

ICE Publishing: All rights reserved
doi: 10.1680/feng.58224.376

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Appropriateness of Current Regulatory Requirements for Ensuring the Robustness of Precast Building Typologies

G.P.Robinson Centre for Innovative and Collaborative Engineering, Loughborough University, Loughborough, UK
A.Palmeri Department of Civil and Building Engineering, Loughborough University, Loughborough, UK
S.A.Austin Department of Civil and Building Engineering, Loughborough University, Loughborough, UK

Abstract

The phenomenon of progressive collapse can be likened to the failure of a house of cards where structural damage propagates beyond the locality of the initial damage and to an extent disproportional to the original cause. Insufficient consideration of the structure's potential for progressive collapse has widely been seen as responsible for some of the most high profile structural collapses of the last 60 years. The need to rigorously consider and mitigate for the risk of such collapse occurring is often seen to be more imperative within the design and detailing of pre-cast concrete structures, which is mainly due to their segmental nature of and associated inherent lack of structural continuity. Aimed at highlighting the need for a more quantitative design methodology, the paper evaluates the suitability of commonly advocated measures for ensuring structural robustness in pre-cast building typologies.

Using a non-linear 'push-down' simulation the suitability of existing tying and anchorage force provisions are evaluated, with such prescriptive detailing rules often adopted by design engineers to justify a suitable level of structural robustness. This computational assessment enabled a quantitative assessment of the performance of pre-cast framed buildings subjected to a sudden column loss event. The findings highlight a need for current design and detailing practice to take more appropriate account of the nonlinear response of components and joints incorporated within multi-storey buildings.

Introduction

Progressive collapse refers to the phenomenon where localised damage brings about wider and even total structural collapse. The failure of the Ronan Point flats in London (1968), the collapse of the Alfred P Murrah Building in Oklahama (1995) and the destruction of the World Trade Centre towers (2001) are perhaps the seminal incidents that highlighted the potential vulnerability of structures to such events. In response, the major international design codes incorporated a number of regulations designed to assure that adequate attention is provided within the design, detailing and construction of buildings so that the resulting structures can be considered to have an adequate 'robustness' or insensitivity to local failure events (Starrosek, 2008). However, despite the existence and application of such rules, a number of studies and publications (Beeby, 1999) have suggested that although practising engineers inherently understand the need to prevent against disproportionate collapse they still lack: the analytical tools, assessment methodologies, appropriate metrics and explicit design guidance that would enable them to ensure the risk of disproportionate collapse is adequately

considered and suitably mitigated. Starrosek (2007) actually suggests that the lack of a widely accepted quantitative method for assessing a structure's ability to resist collapse following an event means that the scope for efficient and repeatable structural design, optimisation and effective review is currently limited.

One such potential method is through the use of a non-linear 'push-down' computational assessment procedure. This method is directly analogous to 'push-over' analysis, which is often used as part of contemporary seismic design to determine the ultimate performance of a structural system. This is achieved by increasing the load multiplier in step increments until the plastic failure of the assembly can be identified and has previously been successfully adapted for use within the robustness assessment of both steel and RC building typologies (Kim 2009, Lee 2011). However, no such similar analysis appears to have been conducted for common precast building types. This is despite the fact that such buildings are often intuitively considered to be less robust than similar, alternative steel or in-situ reinforced concrete structures. Such analysis is perhaps then more imperative for buildings constructed in this way, because of the need to demonstrate that the individual structural components used will be effectively and robustly 'stitched' together.

This paper conducts a series of non-linear push down assessments to better understand the response of typical precast framed structures to a column loss event, quantifying their performance relative to existing robustness requirements. Such analysis will also allow for an assessment of the widely adopted prescriptive 'tying' and 'anchorage' rules, currently deemed sufficient for the avoidance of disproportionate collapse within many of the major international codes. Such assessment is necessary following concerns (Izzuddin 2008) that such rules and details are (in reality) unrelated to the actual actions imposed and structural performance necessary following such a damage event. Specifically, the existing provisions currently exclude any consideration of the necessary ductility demands placed on the connections and structural members (Izzuddin 2008).

Limitations of Current Design Methods

The UK, US and European (ODPM 2004, ACI 2008, CEN 2006) design regulations all contain specific provisions addressing the need to design against disproportionate collapse. All adopt similar procedures in which the buildings are classified based upon their intended use, size and the level of risk that any potential structural collapse may present to the public. This process of building classification defines the appropriate level of structural robustness that must be achieved following the design, detailing and construction processes. However, the subsequent and necessary definition of the required structural performance always appears to be: highly qualitative, aspirational and subjective in nature for each of the regulatory guidance documents.

Starrosek (2007) highlighted this fact, also suggesting that the lack of a more quantitative performance requirement limits the engineer's ability to assess, evaluate or optimise one design method or structural solution against another. Consequentially an engineer is not currently able to quantifiably demonstrate that his/her building will adequately perform in a manner deemed appropriate to its risk classification. For example the designer cannot currently assess how much closer or further away from an acceptable robustness the design will become by adopting one structural form, transfer structure or connection detail over another.

Such limitations within the existing design codes effectively force the design engineer to instead achieve regulatory approval by demonstrating that their adopted design and detailing philosophy is in line with one of the ‘approved’ design strategies available. Through employing such design methods, the resulting building is adjudged (by the pertinent design codes (ODPM, ACI, CEN)) to be sufficiently robust without any need to: assess, measure or justify the resulting structural performance of the construction. This paper however, aims to assess the suitability of two of these strategies, each of which is discussed below.

Prescriptive Tying

Perhaps the most commonly adopted of the available methods is where the engineer ensures that the structural elements and any resulting joints detailed are in line with the prescriptive 'tying force' provisions provided by the codes. The philosophy is based on the assumption that through the use of such details, the designer will consequentially improve the indeterminacy of the structure, localising any damage that may occur, by taking advantage of the alternative load paths established. This is accepted however, without a subsequent need to demonstrate or justify these mechanisms by explicit calculation or computational assessment. Recent studies (Izzudin 2008) have questioned this approach, querying whether the tying provisions defined suitably allow for the true structural actions and effect that such elements and joints will be required to resist following a partial building collapse. Given the period during which they were developed (following the Ronan point collapse in 1968) and the simplicity of the resulting equations, it is unlikely the expressions developed were intended to account for the complex dynamic and non-linear effects induced in reality. The lack of any compulsory regulation requiring the engineer to demonstrate that the adopted construction details are suitably ductile to allow for the resulting large deformations induced, is perhaps the starkest indication that these design expressions do not rigorously consider the true performance requirements for buildings exposed to accidental load conditions.

Alexander (2004) also introduced that for certain structural typologies the philosophy of ensuring structural redundancy via the provision of adequate joint continuity may actually contribute to a progressive collapse event. This work argues that in the event of the loss of structural stability, excessive tying may actually have the effect of 'dragging' out or down elements above or below the region in which the member has been removed or destroyed, questioning the blanket insistence on the use of continuous vertical ties. This 'pull down' phenomenon was actually observed on an experimental concrete panel high rise block constructed and tested by the Building Research Establishment (HMSO 1968). However, no further detailed experimentation, modelling or quantification of this effect appears to have been subsequently conducted. As such, there is little understanding of which building types, layouts or details might be most susceptible to its realisation.

This paper asserts that the suitability of stipulating tying provisions without having first demonstrated that such detailing rules provide a suitable performance in relation to the likely structural actions and ductility demands for which they are included should be questioned. An assessment to check that the final building is not susceptible to any secondary ‘pull-down’ effects is also prudent, with the paper therefore aiming to demonstrate the suitability, or otherwise, of common 'fully tied' precast concrete framed structures in meeting these additional design requirements.

Effective Anchorage

A further, code compliant (ODPM, CEN), robustness design strategy is that of demonstrating that 'effective anchorage' exists at the structural connections between elements, with such a design approach restricted to buildings classified as being of a lower risk of disproportionate collapse (i.e. 2A, 2Lower etc). This provision is again prescriptive in nature, with similar concerns to those expressed in relation to the existing tying requirements again applicable. That is the suitability of the current guidance in relation to calculating and stipulating acceptable loads that the effective anchorage details must resist. For example, the overly simplistic requirement for the connections to resist a (presumably factored) force equivalent to the dead weight of the member it supports (BSI 2010) is again unlikely to correctly allow for the true structural action, ductility requirements and dynamic effects that will be experienced during a collapse or damage event. In a manner similar to that which currently exists for the contemporary tying expressions, little information is at this time provided into how such guidelines have been derived and more significantly in regards to their validity.

Additionally, engineers often fail to appreciate that the use of effective anchorage and prescriptive tying rules represent two distinct design approaches to ensuring a building's robustness. A prominent

example of such a misunderstanding appears to be present within the latest European national guidance document for precast structures (BSI 2010). The specific clause requires all precast floor, roof and stair members to be 'effectively anchored' regardless of the building's robustness class, stating that such anchorages must be designed so that they are capable of transmitting the *"dead weight of the member to that part of the structure that contains ties"*. Such a requirement however, is incongruous and contradictory to the currently accepted approach to robustness design. For example an engineer is entitled to design a lower class structure without the inclusion of vertical ties, with the engineers also possibly having adopted one of the two alternative design strategies available to them (see Research Methodology). If either of these philosophies is instead adopted, it would then be possible that no part of the structure would have to contain vertical ties. How then could the necessary anchorage regulation be met?

Research Methodology

The major international design codes allow the adequate robustness of buildings to be demonstrated through the use of any one of four potential design approaches. These include meeting the prescriptive 'tying force' or alternative 'anchorage' provisions discussed, with the anchorage provisions only relevant to the UK and European regulations for class 2A and 2Lower buildings respectively. Alternatively the engineer may also achieve compliance by ensuring that either the 'Notional Member Removal' or 'Key Element' provisions have instead been met.

Most pertinent to this study are the assessment methods that can be adopted as part of a notional member design. This requires the engineer to demonstrate that following the loss of any vertical load bearing member, the remaining structural components will have sufficient 'capacity' to transfer any resulting actions, through the establishment of suitable alternative load paths. The provision however is currently commonly, with some suggesting unsuitably (Izzudin 2008), applied using conventional design checks. Adopting such a simplistic approach will again fail to account for the complex non-linear geometric and material effects induced by the occurrence of partial structural collapse. Despite this current practice however, the notional column design provision is essentially performance based and as such has been found to allow for the more appropriate consideration of the progressive collapse phenomenon (Kim 2009, Lee 2011). This is because if the correct assessment methodology is used, the engineer becomes able to assess the actual capacity of the structural system. The term 'capacity' is taken to refer to the critical property preventing structural collapse and may therefore relate to element strength, deformability, ductility, stability or stiffness.

This study asserts that because no robustness performance metric is currently defined within the design regulations, and because a building designed using any of the available design approaches can be considered to be adequately robust, it must therefore also currently be the case that a building designed using one of the possible design strategies should be equally robust to a similar building designed using an alternative strategy. As such, it should be that a building designed using the prescriptive tie or anchorage rules will be able to sustain the actions imposed on it under an assessment conducted to meet the notional column removal provisions. This therefore presents an opportunity to assess the adequacy of current tying and effective anchorage rules for ensuring the insensitivity of typical precast concrete building to a progressive collapse. This will be done through the use of a non-linear push down computational study, with such assessments having historically been shown to be suitable for robustness assessment and design against disproportional collapse (Kim 2009, Lee 2011). This analysis will also be capable of investigating both the suitability of the joint details and the building's susceptibility to any secondary effects that may compound the advancement of a progressive collapse. It will thus allow the key concerns associated with tied and anchored buildings to be addressed and evaluated in a quantifiable manner.

Nonlinear Pushdown Analysis of Model Structures

Guidance does exist (GSA 2010) with regards to conducting this type of computational progressive collapse analysis and assessment. Such analysis should be performed by instantly removing vertical load bearing structural elements and then by assessing the structure's residual ability to accommodate such damage. However, software packages that are capable of carrying out such analysis, in order to correctly account for instantaneous changes in the stiffness matrix and building geometry are rarely commercially viable, and thus available to a practising design engineer. As such the GSA (2010) guidelines also allow for alternative (yet 'equivalent'): linear static non-linear static (pseudo-dynamic) and simplified dynamic assessment procedures to be adopted within the progressive collapse assessment of buildings. These analysis methodologies are then more easily carried out using more widely available software packages.

The non-linear static analysis method defined within the GSA regulations requires a stepwise increase in regards to the amplitude of applied vertical loads, until the maximum amplified loads are reached or a collapse is observed (Marjanishvili 2006), with the computational analysis essentially becoming a vertical derivative of a seismic 'push-over' analysis. This assessment technique is utilised as part of this study, as it is capable of allowing for the non-linear material and geometrical effects currently believed to be absent from the contemporary prescriptive design techniques. Although the 'push-down' method cannot capture the instantaneous dynamic effects associated with aspects such as column loss events or debris loading for example, studies (Izzuddin 2008, Marjanishvili 2006) have shown that the application of factored ‘equivalent’ static load cases are capable of accounting for such actions and effects.

Interestingly, the GSA guidelines (GSA 2003) allow for and define a load controlled, non-linear static analysis, in which the load is applied to the structure in at least ten incremental steps from zero up until the total specified loading. The resistance of the structure against such loading is assessed, with the output forces, moments, shears and deformations each then compared against the relevant acceptance criteria. However, it has been shown that such load controlled push-down generally involves numerous analysis re-runs, is sensitive to the chosen load steps and tolerances and is also unable to converge to a solution when the ‘load factor’ begins fall, i.e. when building collapse is progressing (Marjanishvili 2006). The load factor refers to a measure of performance utilised as part of similar studies considering the non-linear push-down assessment of multi-storey buildings (Kim 2009, Lee 2011, Marjanishvili 2006). The metric essentially quantifies what proportion of the load case the plastically deformed or ‘collapse-arrested’ structure can transmit to the foundations through the alternative load paths it can establish and is defined as:

$$\text{Load Factor} = \frac{\text{Equivalent Applied Load}}{\text{Total 'Linear Static' Load}} \tag{1}$$

In this way a load factor >1.0 represents a building that would not collapse before the required design load conditions have been exceeded and as such can be considered to be suitably robust.

Design and Analysis of Structural Models

The adopted analysis models have been designed to represent a precast framed structure with a $7.5 \times 7.5\text{m}$ structural grid and a floor to floor height of 3.8m. Models consisting of two, four and ten storeys were analysed for a 'tied' frame design. Alternative models adopting effectively anchored connections were also considered for the two and four storey structures, with the elevations for the analysed structures illustrated within Figures 1 ((a)-(c)). The precast column and beam elements are designed in accordance with the EC2 design code (CEN, 2006) for $C40/50$ and $f_y = 500 N/mm^2$ grade concrete and reinforcement respectively. The resulting beam and column sections are depicted as part of Figure 2 ((a)-(b)).

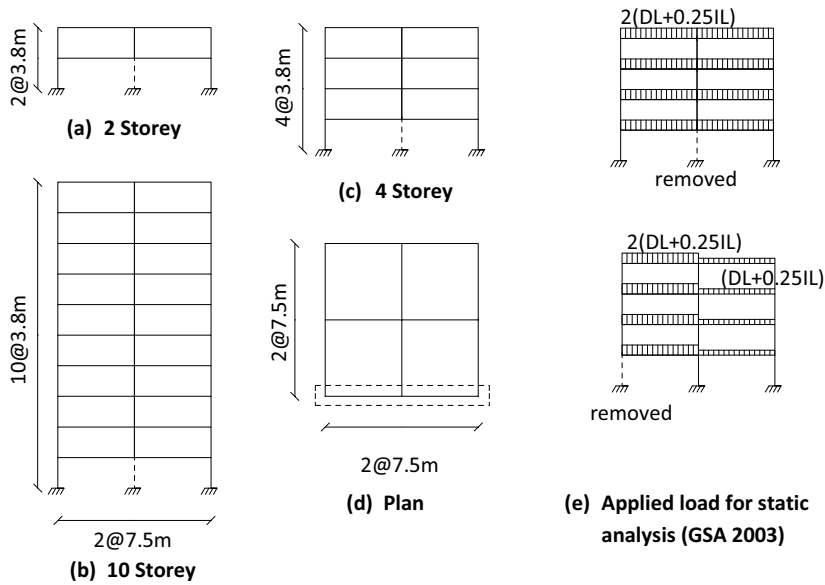


Figure 1 Elevation, Plan View and Loading Conditions for the Analysis of Computational Structures

A superimposed dead load of 6kN/m^2 and an imposed load of 2.5kN/m^2 were applied to each of the models analysed. In addition, a notional horizontal lateral load, equivalent to 1.5% of the characteristic weight of the structure, was also applied in order to represent the non-verticality of the precast column members. The lack of application of such a minimum level of horizontal load appears to be a significant limitation of similar previously undertaken studies (Kim 2009, Lee 2011). However, this load is important in correctly accounting for and identifying any potentially detrimental secondary geometrical effects that may occur.

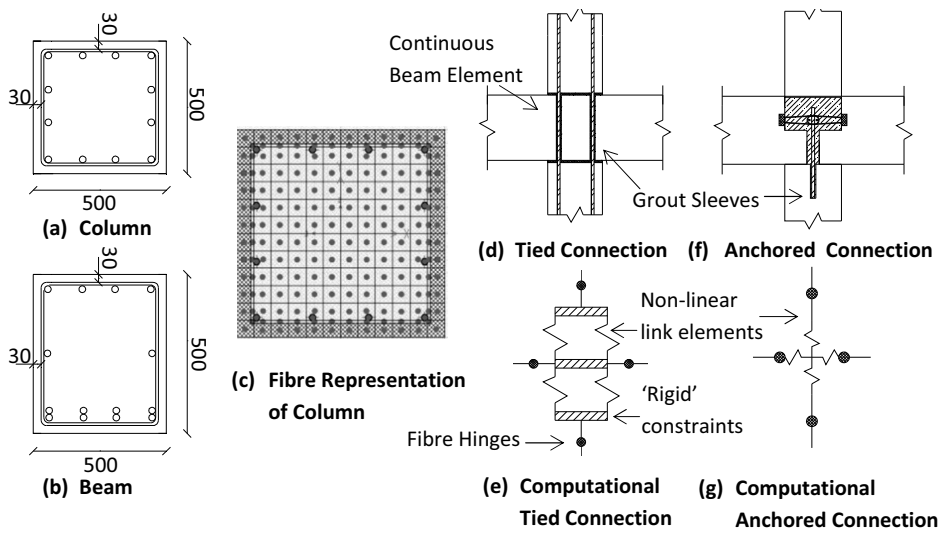


Figure 2 Structural Sections, Connection Designs and Computational Equivalents

Further, in order to account for the dynamic effects associated with a sudden column removal event, the recommendations given within the GSA (2010) guidelines require a dynamic amplification factor of 2 to be applied to the spans for which the column is removed. An un-amplified dead load is also applied to the remaining spans, with an imposed load reduction factor of 0.25 applied in both cases. The resulting load combinations are illustrated as part of Figure 1 (e). Because the chosen building types have a simple and repetitive layout, only two critical structural damage scenarios are considered necessary for investigation. These were the removal of central and corner column elements at the critical lower level of the structure (Figure 1 (e)).

Because of the identified limitations associated with the load-controlled push-down method currently prescribed within the GSA recommendations, the vertical push-down analysis was instead carried out through the adoption of a displacement controlled assessment. That is, the vertical displacement at the position (or node) where the column is removed was incrementally increased with the corresponding vertical load to this displacement then calculated. This allows the load factor to be similarly evaluated, although the analysis can be more expediently run, as well as being significantly less likely to diverge, when compared to the load controlled alternative (Kim 2009).

Effective use of the proposed non-linear, static robustness assessment procedure is of course highly dependent on the adopted representation of the plastic properties of each component, as well as their connections, as part of the computational model (Inel 2006). That is, our understanding of the ultimate inelastic deformation capacities of the components detailed in terms of their geometric and mechanical characteristics should be captured as part of the assessment. The required non-linear load-deformation relationships have, in previous studies (Kim 2009, Lee 2011), been based on those values published within seismic design guidance, such as ASCE 41-06 (2007). However, these values do not account for the effect of significant variations in the axial forces applied to the components. Such forces and variation though, will be much more prominent and critical within a progressive collapse simulation than for the seismic assessments for which the values were derived. This is because such forces will significantly affect (in potentially both a beneficial and detrimental manner) the rotational behaviours and thus capacities of the elements and connections. Therefore, a much more effective method of capturing the structural behaviour of the RC elements was considered to be through the use of 'fibre-hinge' analytical elements. In this representation, the element's cross-section is subdivided into a number of elementary layers or 'fibres' to which the appropriate material models

can then be assigned (Figure 2 (c)). By dividing the structural cross section in this way it is possible to determine an effective representation of the non-linear moment-curvature relationship for the structural component in a manner that suitably accounts for the proportion of axial load applied. The non-linear load deformation characteristics derived in this way were then also validated against relevant experimentally derived values (Panagiotakis 2001). The associated structural behaviour was then incorporated within the computational models as 'hinge' elements that are specified at the locations where the applied lateral and gravity loads are considered to produce maximum effects. That is the plasticity of the structural components (modelled as a $P-M_2-M_3$ hinge) is assumed to be lumped at the centre and ends of the beam and column elements.

The load deformation characteristics relating to the precast connection details were determined by consideration of the behaviour of the details illustrated within Figures 2(d) and 2(f). Such details are commonly adopted in UK structural design in order to meet tied and effective anchorage conditions respectively. For the vertical continuity tying requirements, the load that the connection is required to resist (as a tensile force), is determined by consideration of the equivalent axial compressive load that the column removed resists prior to its loss. However, this load only relates to that action which results from the application of the accidental load case and only for the load that is from the storey which would have been directly supported by the removed column. The resulting detail (Figure 2(d)) incorporates H25 reinforcing bars which are equally spaced about the centre point of the column, with the bars also fully anchored and lapped with the reinforcement within the precast column. The connection was modelled using non-linear 'link' elements and constraints as illustrated within Figure 2(e) in order to assess the suitability of the connection with respect to its rotational capacity.

To demonstrate the suitability of an effective anchorage connection (Figure 2(f)), it was only necessary to demonstrate that a lateral force equal to the dead weight of the horizontal member it supports can be resisted, with all the applied loads again factored as required, under accidental conditions by the European code (CEN 2006). For the detail considered, the reinforcing bar grouted into position is designed to act as a type of cast in steel billet, i.e. it acts in shear. However, because of the insufficient lap/anchorage of the bar it cannot be considered to have any rotational capacity. Therefore this connection type was modelled as non-linear link element, which was specified to lose load bearing capacity once the code stipulated axial limit had been reached (Figure 2(g)).

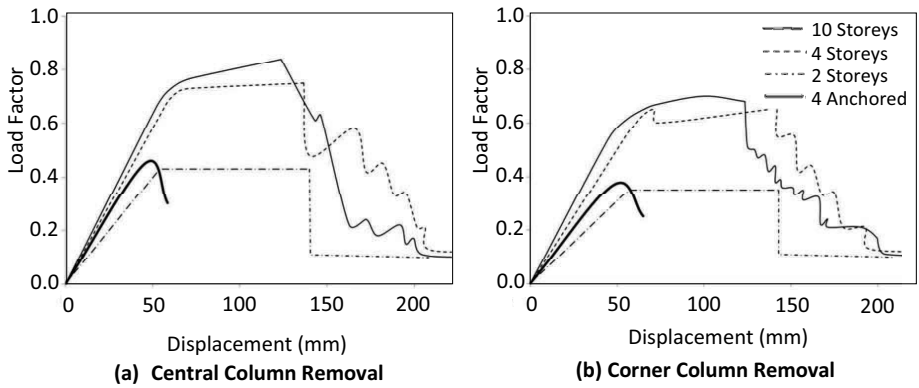
The Performance of Tied and Anchored Precast Framed Buildings

The response of the chosen precast building typologies to the nonlinear static push-down analyses conducted is presented within Figures 3(a) and 3(b) for the structures subjected to a column loss event at the centre and corner of the building's end bay respectively. The plots show the load factor (Eq. (1)) against the imposed deflection at the location at which the column has been removed. Because the maximum strength of structures in each case does not exceed a load factor of 1.0 none of the structural typologies considered would satisfy the recommendations of the GSA (2010) guidelines.

It was observed that for buildings of 10 storeys adopting a tied design, and for cases in which a corner column was removed, the precast framed structure 'yielded' at a load factor of around 0.58 with plastic hinge failures observed to occur initially, and as would be expected, at the point of maximum moment due to the induced cantilever. A much higher initial yield (0.75) and increased maximum strength (0.81-0.84) was observed for the structural models in which the central column was removed. Such a response should be expected because in the cases where a corner column has been removed, the push-down load is only being resisted by one, rather the two bays that act for the central column case. Interestingly, an improved performance was seen for both the central and corner column load cases as the number of storeys was increased for the tied buildings. This appears to be because of a combination of effects. Firstly, the increased axial load appears to act so as to improve the moment rotation capacity of the plastic hinges. In addition, the taller buildings also have more structural

members and the presence of more components in the building/model appears to inherently increase the number of alternative load paths which are available to resist and redistribute the induced loads.

As can be seen from Figure 3(a) a much more suitable overall building response to the column loss events was seen for the 'tied' rather than the 'anchored' building types. This is because after reaching ultimate strength a much more gradual saw toothed falling branch is seen until failure, with every instantaneous drop in strength relating directly to a plastic hinge reaching its ultimate strain limit and the loss of residual plastic strength. In contrast, the load factor plot observed for the anchored low rise buildings was observed to be almost 'elastic' and 'brittle' in its nature. This is because framed structures resist progressive collapse essentially through the action of the vertical ties in tension and the rotational capacity/ductility of the beam to column connections. Because the anchored connections modelled only offer restraint in one constrained axial direction, they are consequentially ineffective in arresting the building collapse for the low rise structures considered.



(a) Central Column Removal **(b) Corner Column Removal**

Figure 3 Load-Displacement Relationships of Model Structures

Conclusions and Proposals for Future Work

For the simplistic precast framed structures considered none were found to meet the GSA (2010) robustness performance regulations. All of the two and four storey structures investigated could also be classified as 'susceptible' to progressive collapse, according to the performance metric proposed by Marjanishvili and Agnew (2006). However, none of the buildings considered showed any indication that a secondary, detrimental 'pull down' effect due to the use of ties would induce or hasten the collapse sequence. The resulting behaviour and therefore ‘performance’ of the tied structures though is considered to be directly related to and significantly affected by the chosen tying detail Figure 2(d). However, a larger amount of investigation into the sensitivity of building performance to the nature of the precast tied connections to be used is required before any firm conclusions in regards to the suitability of the current prescriptive tie design methodology and detailing rules, as they apply to precast framed structures, can be drawn.

Further, this study also provides no indication of in what manner the measured robustness of the structure will change in response to variations in: span length, storey height or plan shape. It is proposed therefore that such variables should be considered and incorporated as part of any future, similar studies, so as to further inform any necessary corrections to the existing robustness design regulations and guidance. In addition, analogous investigation of the performance of alternative precast cross wall construction typologies and the effect of utilising and suitably modelling for segmental and flexible floor diaphragms (e.g. Prestressed Hollowcore floor units) would also be of great significance to ensuring the suitable design of robust precast building typologies in the future.

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ISBN 978-0-7277-5822-4

ICE Publishing: All rights reserved
doi: 10.1680/feng.58224.389

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Lessons learned from structures damaged by delayed ettringite formation and the French prevention strategy

Bruno Godart, Université Paris-Est, IFSTTAR, Paris, France
Loïc Divet, Université Paris-Est, IFSTTAR, Paris, France

The Development of DEF

The expansive sulphate internal reaction due to Delayed Ettringite Formation (DEF) can damage concrete structures severely. The primary ettringite (a hydrous calcium trisulphoaluminate) is a normal reaction product formed from the reaction of C_3A and C_4AF with gypsum during the plastic stage of the hydration of Portland cement. However, when peak temperatures in concrete are over about 65°C, the sulphates may be incorporated in other cement phases. After concrete hardening, the very slow formation of higher volume secondary ettringite may occur as water is taken into the crystal structure which can lead to potentially disruptive expansion. DEF is defined as the formation of ettringite in a concrete after setting, and without any external sulphate supply, but with a water supply. DEF appears in concretes exposed to frequent humidity or contact to water, and subjected to a relatively high thermal treatment (> 65°C) or having reached equivalent temperatures for other reasons (massive cast-in-place concrete, concrete casting during summer, etc).

The first reported cases of DEF occurred in some precast concrete elements subjected to a heat treatment unsuited to the composition and the environment of the concrete. International examples of DEF include railway sleepers (Tepponen 1987, Heinz 1989, Vitouva 1991, Shayan 1992, Oberholster 1992, Mielenz 1995, Sahu 2004, Santos Silva 2008), and massive cast-in-place concrete components (Collepardi 1999, Hobbs 2001, Thomas 2008, Ingham 2012). DEF was first observed in France, in 1997 (Divet 1998), on bridges whose concrete had been cast on site. The bridge parts damaged by DEF were primarily massive structural elements (piers, crossbeams on piers or abutments, etc.) in contact with water or subjected to high moisture. DEF is distinct from the more traditional sulphate reactions where the sulphates attack the concrete from outside and create a progressive degradation from the surface towards the interior of the element. DEF affects the interior concrete without any ingress of external sulphates. It leads to a concrete swelling and the cracking of the structure.

Diagnosis of DEF

Laboratory studies were conducted using petrographic and mineralogical analysis (Deloye, 1977, AFPC-AFREM 1997). They included the following investigations: porosity, chemical analysis, X-ray diffraction, thermal analysis and scanning electron microscopy (on polished sections and broken surfaces). Several core samples were taken from the structures, with varying depths. The specimens were taken from both degraded zones and un-degraded zones to allow comparison between the two.

It is important to note that no alkali-silica reaction product was observed in the following investigated structures cases. The microscopic analyses showed that the studied concretes have been subjected to a major expansive internal sulphate reaction as ettringite formed after hardening. This check was conducted with the objective of avoiding any confusion with damages caused by the alkali-silica reaction. This is important as when both causes of deterioration are present, it is difficult to know which reaction is mainly responsible for the damage.

The internal sulphate reaction results in the presence of ettringite. This latter was present throughout the material and on all the examined surfaces. The ettringite that is generally observed in the concrete has a wide range of textures, but it is mainly poorly crystallized and massive. Furthermore this form is principally present at the cement paste/aggregate interfaces. Figure 1 is a back-scattered electron (BSE) image showing ettringite-filled gap around aggregate particles.

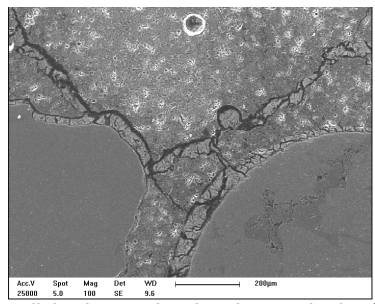


Figure 1: BSE image of a polished sample showing ettringite-filled gap around aggregate particles.

Temperature is a fundamental parameter for the formation of delayed ettringite. For this reason it is necessary to evaluate the temperature within concrete during setting. Our starting hypothesis was that the temperature rise in a massive concrete element can be estimated on the basis of the temperature rise under adiabatic conditions. The temperature field T(X,t) was predicted at all points of coordinates X in the structure and at each time t. This prediction is based on a finite element calculation which requires the knowledge of the exothermic properties of the concrete, the geometry and the parameters that regulate exchanges between the environment and the concrete element. Modelling was conducted as a diagnostic aid to understand the damage observed in the structure after several years. Numerical simulation of the thermal effects was undertaken with the TEXO module which is part of the CESAR-LCPC finite element program of IFSTTAR.

Lessons learned from investigations of structures

Most of the following cases are presented in detail in the report of L. Divet (Divet 2001).

The Ondes bridge

This bridge, located near Toulouse, was completed and opened to traffic in 1955. It consists of four pre-tensioned beams which are joined together by a top slab and crossbeams (Fig.2). It has a total length of 202 m and consists of five independent spans. The average height of the piers is 6.8 m. They are constructed on mass concrete which is set in the molassic substratum and they consist of two circular columns, 2 m in diameter, which are connected at the top by a

cap beam (see Fig.2). These latter are either hollow, at piers P1 and P3 supporting the moving bridge bearings, or solid, at piers P2 and P4 to which the fixed bridge bearings are fitted.

The damage only affects the solid cap beam of pier P2 (Fig. 3), the concrete of which was cast in August 1954. This is a solid parallelepiped reinforced concrete cap beam, measuring 1.5 m high, 8.2m long and 2.7 m wide. The first visible defects were observed during a detailed inspection conducted in 1982. The inspection report mentions the presence of cracking and the growth of vegetation at the ends of the pier cap beam. The previous inspection reports did not mention these defects. We can consequently assume that the damage first occurred in the early 1980s.



Figure 2: Overall view of the Ondes bridge



Figure 3: View of the end of the cap beam in 2005, after the treatment applied in 1995

The damage appeared as a network of widely-spaced multidirectional cracks. Cracking sometimes has a preferential orientation that depends on how the reinforcement is distributed. The crack widths vary from a few tenths of millimetres to a few millimetres according to the zones. The cracks are frequently emphasized by moisture and, occasionally, a whitish or greyish substance has been observed to exude from the cracks. The horizontal cracks appear to be more severe and are occasionally as much as 7 mm wide. Lastly, the damage is mainly present at the cap beam ends, on the facings that are directly exposed to bad weather and water dripping from the footpaths. No damage was found on the central part of the cap beam.

The concrete of the cap beams has a specified cement content of 350 kg/m^3 of Ordinary Portland Cement (CEM I type), with a water/cement ratio of 0.5 in mass. The aggregates are alluvial. The investigations conducted in the laboratory showed that the deteriorated concrete of pier P2 did not conform to the specifications. Analysis showed that the concrete contains a cement overdose of about 100 kg/m^3 (cement content of 430 kg/m^3). The mix is well composed of sand and gravels 5/15, but it incorporates gravels 25/50 that were not specified initially (see table 1).

The heating cycle of the concrete during its setting was then estimated by a numerical simulation using the TEXO module. For this, information was required on: meteorological data during the setting period of the concrete, geometry of the structural element, type and duration of use of the formwork, construction phases, exothermic properties of the concrete. A three-dimensional thermal modelling was performed using this data (fig. 4). Figure 5 shows a plot of temperature versus time. The temperature increase was important for the cap beam of pier P2: the maximum temperature reached in the centre of the material was close to 80 °C, while for the cap beam of pier P4, the temperature did not exceed 70 °C. Moreover, the temperature in the concrete of pier P2 remained over 70°C during about 70 hours.

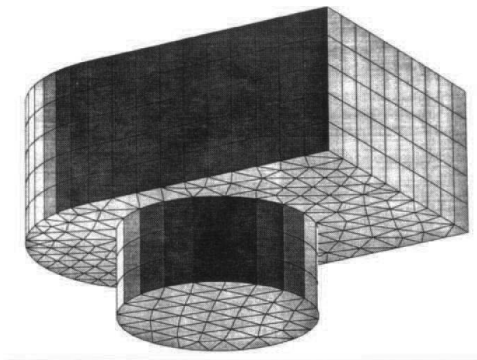


Figure 4: 3D modeling of the cap beam of pier P2 with CESAR-LCPC.

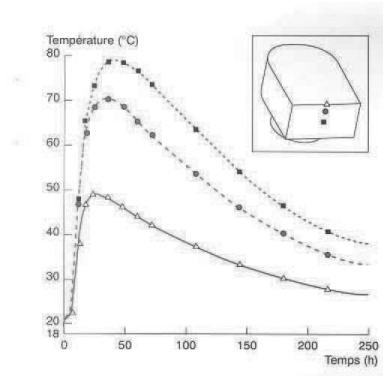


Figure 5: Computed temperatures in the cap beam of pier P2

In consequence the deterioration due to the DEF is limited to that in the cap beam of pier P2. This is the consequence of the initial heating of the concrete from the combination of several causes: use of an exothermic OPC for a massive concrete part, pouring of the concrete during summer time without any particular precautions and an overdose of the cement relative to the initial specified concrete composition.

As part of the remedial programme on the Ondes bridge, the waterproofing membrane was repaired and the expansion joints of the roadway and sidewalk were reformed in June 1995. On pier P2, the extremities of the cap beam were treated by an injection of the cracks and then protected by a surface coating. In July 1996, a discontinuous horizontal crack of width 0.2 mm was observed. Water inflows are also observed on the extremities, despite the repair made in 1995. The evolution of the phenomenon is confirmed in 1998 with the occurrence of new cracks and by the results of the monitoring of the global deformations that has been installed. This attests of the difficulty to implement durable repair solutions for this type of pathology, and justifies the interest in enforcing preventive measures during construction to avoid DEF.

Table 1 : Specified concrete composition of the different bridges (in kg/m^3).

Material	Cement	Water	Filler	Sand	Gravel	Gravel	Gravel
Ondes	350	175		750 (0/5)	1100 (5/15)		
Bourgogne	400	180		730 (0/4)	376 (4/12)	700 (12/20)	
Lodeve	400	188	140* (0/3)	555 (0/4)	395 (4/10)	310 (10/20)	
Bellevue	380	205		800	290 (6/10)	130 (10/14)	640 (14/20)
Beynost	350	170		750 (0/5)	250 (5/10)	930 (10/20)	
Cheviré	385	185	75	730	285 (6/10)	130 (10/14)	630 (14/20)

* calcareous sand

The Bourgogne bridge

The Bourgogne bridge is a cable-stayed bridge built during the years 1989-1992 over the Saône River in Chalon/Saône (Fig. 6). It has a total length of 351 m and comprises 8 spans. The main span is 151.80 m long, and the pylons are 46 m high. The deck is made of a prestressed concrete slab. The lower part of each pylon is composed of a footing, a base and two legs. The dimensions (in metres) with respect to length, width and height are respectively for the footing: 15.20 x 6 x 2, for the base: 11.90 x 3 x 6.5, and for the legs: 2.20 x 3 x 6.

The footing is always immersed, the base is partially immersed in water but may be sometimes fully immersed during floods, and the legs are always above the water level of the river. The immersed parts of the base of the pylons P3 and P4 are affected by a multidirectional cracking like in the aerial parts (fig. 7). The width of the cracks reaches several millimetres on P3, and is lower than one millimetre on P4. A high cracking density is observed on the faces of the base, with a maximum density in the middle of the faces. The most important crack is observed on the top face of the base with a width of 1 cm. Some cracks also show an offset of the crack lips which is typical of a differential expansion inside the concrete; the most important offset is 2 to 3 mm on the top face of the P3 base. The lateral faces of the legs of pylons P3 and P4 present some cracks that are oriented horizontal, with widths varying between 0.2 and 0.4 mm.



Figure 6: Overall view of the Bourgogne bridge



Figure 7: Cracking of the aerial part of the pylon base

For the footing, the base and the legs, the cement content is 400 kg/m3, the water/cement ratio is 0.45 and the cement used is a CPA 55 R (CEMI 52.5 N type).

A finite element calculation of the temperature reached in the base and in the legs was conducted with the module TEXO, and with the help of adiabatic test results obtained on a reconstituted concrete. The results show that the temperature in the footing, the base and the two legs exceeds the 65 °C threshold, with respectively 79 °C for the footing and the base, 71°C for the lower part of the legs and 61 °C for the upper part of the legs. They show that the temperature of the base remains over 70 °C during 6 days, while those of the footing remains over 70 °C during 3 days. The laboratory investigations (particularly with the Scanning Electron Microscope) confirmed that the concrete of the basement was affected by DEF and that the concrete of the footing and legs was unaffected.

The Lodève Bridge

This bridge was constructed during the years 1980-1981. It is a statically determinate pre-stressed beam structure with a three span deck which is 120 m long and 13 m wide. The deck is supported by two abutments and two piers constructed on footings. The damage is confined to the cap beams of the two piers. Each pier (Fig. 8) consists of a pillar whose cross-section fits within a rectangle measuring 3.1 x 7.5 m, with a height of 14.5 m in the case of the northern pier and 9.3 m in the case of the southern pier. Each pillar is topped by a cap beam which has a length of 14 m, a width of 3.5 m, and whose height varies between 1.1 m and 2.0 m. The extremities of the cap beams are covered with architectural facings consisting of six prefabricated reinforced concrete units. The two cap beams were built in August and September 1980.

The cap beam on the North pier has closely-spaced vertical cracks on both sides with a maximum crack width varying between 0.2 mm and 1.2 mm (fig. 9). The extremities of the cap beam are covered by the architectural facings which prevent their observation. Some cracks contain streaks of calcite, which indicates a circulation of water inside the concrete. Several horizontal cracks are also present and appear to be located at construction joints. The results of the 1997 crack survey for the cap beam of the north pier shows a considerable increase in cracking as compared with the 1989 inspection.



Figure 8: Overall view of a Lodève Bridge pier

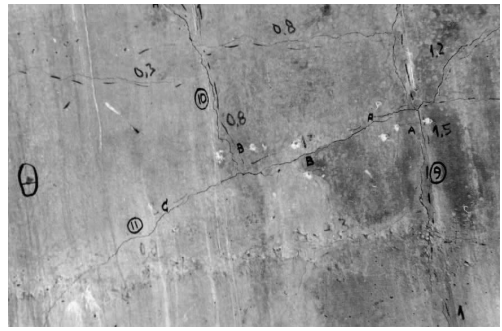


Figure 9: Network of cracks on the cap beam of the North pier

The cap beam of the South pier suffers from the same type of damage, but seems less severely affected. The maximum crack width is only 0.3 mm, except on the east extremity where widths attain 1.7 mm. Furthermore, the network of cracks is denser and efflorescence can be observed in this zone. Lastly, the detailed inspection conducted in 1997 showed a lack of drainage from the supports (abutments and piers). Therefore, some parts of the supports are abnormally exposed to a humid environment.

The damage rating of a concrete structure may be determined with the help of a cracking index which corresponds to the summation of cracks intercepted on axes of variable lengths and normalized to a length of one metre (LCPC 1997). One of the cap beam of the Lodève bridge has been monitored with the help of cracking indexes and measurements of overall movement (LCPC 2009) since 2000. The cracking index measured in the central part of the cap beam is 0.55 mm/m and is stabilised. According to the classification (LCPC 1997), this value corresponds to a low cracking. For the extremity of the cap beam, the cracking index

reached 1.64 in 2002 and 2.0 in 2007; it indicated an evolution of the phenomenon and the cracking is rated moderate to high. This was confirmed by the overall movement monitoring which gives a global deformation rate for the cap beam extremity of 0.2 mm/m/year.

The concrete of the piles and cap beams was made with an OPC cement (CEM I type) having a rapid hardening; the cement content was 400 kg/m^3 and the W/C ratio was 0.47. The aggregates are silico-calcareous.

A finite element calculation of the temperature reached in the cap beam and the pillar (module TEXO) was carried out to explain the difference of behaviour between the damaged cap beam and the non damaged pillar. Laboratory investigations (particularly with the Scanning Electron Microscope) confirmed that the concrete of the pillar was unaffected by DEF. In the cap beam, the calculated temperature increase was close to 80 °C in the centre and the temperature remained over 70 °C for nearly 5 days. In the pillar, which is less massive and has a particular geometry which favours the heat losses because of a larger surface area, the maximum calculated temperature reached 70 °C in a relatively small zone in the centre. Most of the concrete of the pillar reached a temperature below 65 °C. This was the main reason for the absence of DEF in the pillar but it is also less exposed to water ingress and humidity..

The Bellevue Bridge

This bridge was constructed in Nantes in 1988–1989 and opened in 1990 (Fig. 10). It is a continuous box girder bridge with 6 spans and a total length of 385 m, and has 7 supports (2 abutments and 5 piers). The 5 reinforced concrete piers are massive and are 6.3 m long by 3.0m wide. The damage is mostly confined to pier P6 on which multidirectional cracking typical of that caused by an internal swelling reaction is apparent (Fig. 11). In addition the map cracking is mainly localised within the tidal zone, where the average tidal range is between 3 and 4 metres. The four sides of the pier are affected and cracking seems to be progressing in the dry zone. The concrete of this pier was cast in August and September 1989. Pier P4 also exhibits map cracking, but this is less severe. Lastly, piers P3 and P5 have vertical cracks with widths reaching 0.8 mm. However, this cracking was observed immediately after construction and was attributed to thermal shrinkage of the concrete.

The cement content is 380 kg/m^3, the W/C ratio is 0.54, and the aggregates are predominately from siliceous and silicate rocks. The cement is a Portland cement (CEM II/A type) composed of about 10 % of calcareous fillers (see table 1).

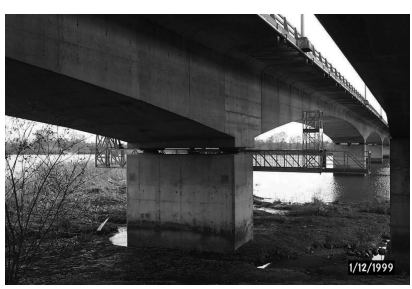


Figure 10: Overall view of The Bellevue Bridge



Figure 11: Pier 6 of the Bellevue Bridge with map cracking in the tidal zone.

The evolution of the temperature inside the concrete has not been estimated by a finite element calculation. The maximum temperature was estimated by the simplified method of calculation presented in the annex 4 of the LCPC guide (LCPC 2007). With the hypothesis that the temperature of the fresh concrete at the moment of pouring was equal to 20 °C, the maximum temperature at the centre of the pier was estimated to have reached 75 °C.

The Saint-Maurice de Beynost bridge

This bridge, built in 1992, has a deck made of reinforced concrete beams and is composed of three independent spans made continuous on piers by the slab over the beams. The spans are skewed and have different length (14.92 m – 17.44 m – 9.47 m). The deck is resting on two abutments and two intermediary piers (fig. 12). Each abutment is composed of 4 rectangular columns topped by a cap beam. Each pier is composed of 5 columns of 5.4 m height and also topped by a cap beam.

The damage has occurred only on the extremity of the cap beam of the pier P2 which is exposed to the elements (fig. 13). A map cracking was observed during the first detailed inspection of the bridge in 1992. This cracking is evolving since the width of the cracks was below 0.3 mm in 1992 and reached up to 2 mm in the detailed inspection of 1997.



Figure 12: Partial view of the Saint Maurice de Beynost bridge



Figure 13: Map cracking at the end of the cap beam exposed to rain

From the documentation of the bridge, it was possible to find the dates of casting of the concrete in the different parts of the bridge. The cap beam of pier P2 was built in August 1982. The cement is an OPC (CEM I type), the W/C ratio is 0.49, and the concrete composition is close to the specified value of 350kg/m^3 given in table 1.

The calculation of the temperature increase with time in the bridge parts (pier and cap beam) was conducted with the TEXO. It was based on the meteorological data for the construction period and data obtained from adiabatic tests carried on the concrete composition reconstituted in laboratory. No damage has occurred in the piers, and the calculation shows that the maximum temperature reached at the centre of the piers was 61 °C. For the cap beam of the pier P2, laboratory investigations identified characteristic features of DEF, and the maximum temperature reached at the centre of this cap beam was 69 °C. The calculation shows also that the cooling of the cap beam was slow and that its concrete temperature remains greater than 60 °C for 44 hours.

The Cheviré Bridge

The Cheviré bridge was opened to traffic in May 1991 and is classified as an exceptional bridge (Fig. 14). It comprises two access viaducts composed of continuous box girders made

of prestressed concrete having respectively a length of 603 m for the north viaduct and 798 m for the south. The central span has a length of 242 m and includes an independent steel box girder 164 m long, supported by cantilever prestressed concrete box girder. The total length of the bridge is 1562 m. The North viaduct is supporting by one abutment and nine piers, and the South one is supporting by one abutment and twelve piers. The hollow piers have a variable height (from 5.2 to 43.5 m) and comprise concrete bases at the bottom. The dimensions of these bases are: 3.15 x 3.90 x 4.00 m.

In 1998, map cracking was observed on the facings of some bases of the South access viaduct (Fig. 15). DEF was identified by laboratory investigations. The rate of deterioration of the bases is variable: some bases presented only some vertical cracks with a width lower than 0.3mm while others exhibited more or less developed map cracking (up to a width of 0.6 mm). The phenomenon is globally slow and the rate of opening of the cracks was 0.1 mm/m/year, from 2003 to 2006.



Figure 14: General view of the Cheviré Bridge. (Courtesy G. Forquet)



Figure 15: Cracking is more important in the bottom of the base because of the capillary suction (Cheviré bridge)

The searches in the records showed that the damaged bases were cast during the summer 1988. The cracked faces are exposed to rain and capillary rises. The concrete was made with a Portland cement incorporating calcareous fillers (cement II/A type), a marine sand and gneiss aggregates.

The strategy for prevention

The French recommendations for the prevention of damage due to DEF (LCPC 2007) were established from data available in the scientific literature and from the detailed studies of damaged structures, like the six cases presented above. From this analysis, a strategy was developed based on categorising the structure according to the level of risk of occurrence of cracking that can be accepted, and related to the environmental conditions to which the structure is exposed during its service life. This strategy (Godart 2012) defines four levels of prevention with precautions that have to be applied related to risk and environmental categories. The precautions are mainly based on the limitation of the maximum temperature reached within the centre of the concrete pour during hardening of the concrete, and on the choice of an appropriate composition of the concrete.

The structures (or parts of them) are classified in 3 categories that are representative of the level of risk of cracking with respect to DEF that are acceptable for a given structure (or a part of it). The choice of the structure category is a function of the nature of the structure, its

purpose, the consequences of the cracking in relation with the desired safety level, and its future maintenance: Category I refers to the structures for which the consequences of the occurrence of cracking are low or acceptable (for example: temporary structures), Category II gathers the structures for which the consequences of the occurrence of cracking would be detrimental (like resisting elements of most buildings and civil engineering structures), and Category III corresponds to structures for which the consequences of the occurrence of cracking would be unacceptable (like nuclear containment, dams or tunnels).

Table 2: Comparative study of factors encountered in the investigated bridges.

	Ondes	Bourgogne	Lodève	Bellevue	Beynost	Cheviré
Date of construction	1955	1990	1980	1988	1982	1988/89
Casting of concrete	August	August/ September	August/ September	August/ September	August	July/ August
Structural part	Cap beam	Base of pylon	Cap beam	Pier	Cap beam	Base of pier
Delay in occurrence of damage	27 yrs	6 yrs	9 yrs	10 yrs	10 yrs	8 yrs
Environ-ment	Waterproofing Problem	Immersed & variable immersion	Lack of drainage	Immersed & variable immersion	Exposure to rains	Rains and capillarity
T max (°C)	80	79	80	80	69	75
W/C ratio	0.50	0.45	0.47	0.54	0.49	0.48
Nature of cement	CEM I	CEM I	CEM I	CEM II/A	CEM I	CEM II/A
Cement con-tent (kg/m^3)	430	400	400	380	350	385
SO_3 cement (%)	2,5	2,8	2,6	2,5	3,4	2,5
C3A cement (%)	11,2	8,2	9,8	7,0	10,4	7,0

Three exposure classes take into account the fact that water or a high ambient humidity are factors necessary for the development of DEF: the class XH1 concerns a dry or moderate humidity, the class XH2 is related with alternation of humidity and drying or with a high humidity, and the class XH3 considers structures in constant with water (permanent immersion, water stagnation on the surface, tidal zone,...).

Then four levels of prevention are determined according to the risk category of the structure and the exposure class XH to which the considered part of the structure is subjected. The choice of these levels of prevention indicated by the letters As, Bs, Cs and Ds, is the responsibility of the structure owner in setting the design specification. Table 3 is helpful when making this selection.

The principle of prevention relies primarily on the limitation of the heating of the concrete characterised by the maximum temperature, Tmax, likely to be reached within the structure and, also, by the duration of the period for which a high temperature is maintained. The limiting values of Tmax were defined by a group of expert on the basis of a literature review,

a comparative study of factors encountered in damaged and non damaged bridges (Table 2), several French research thesis, and some foreign recommendations such as (BRE, 2001).

Table 3 : Levels of prevention defined by structure risk category with exposure

Risk category of structure	Moisture/water exposure class of the structural part		
	XH1	XH2	XH3
I	As	As	As
II	As	Bs	Cs
III	As	Cs	Ds

The precautions corresponding to the four levels of prevention are the following:

- Level As: Tmax < 85 °C. However in the case of a heat treatment applied on a precast element, it is authorized to go beyond the temperature Tmax = 85 °C up to 90 °C, provided that the length of the period during which the temperature exceeds 85°C is limited to 4 hours.
- Level Bs: Tmax < 75 °C. However if the maximum temperature reached in the concrete cannot remain lower than 75°C, then it must remain lower than 85 °C and at least one supplementary condition must be respected.
- Level Cs: Tmax < 70 °C. However if the maximum temperature reached in the concrete cannot remain lower than 70°C, then it must remain lower than 80 °C and at least one supplementary condition must be respected.
- Level Ds: Tmax < 65 °C. But if Tmax cannot remain lower than 65 °C, then it must remain lower than 75°C with the use of mineral additions and the validation of the concrete composition by an independent laboratory expert in DEF.

The supplementary condition should be selected from the following:

- Equivalent active alkalis of the concrete < 3 kg/m^3
- Use of a cement conforming to the standard NF P 15-319 (ES) for sulphate resisting cements, and some additional conditions (see LCPC 2007)
- Use of cements non conforming to the standard NF P 15-319 of the type CEM II/B-V or CEM II/B-S or CEM II/B-Q or CEM II/B (S-V) or CEM III/A or CEM V, with SO_3 of cement < 3 % and C_3A of the clinker < 8 %
- Use of fly ashes, slags, calcinated natural pozzolans or metakaolin in combination with a CEM I, with additions content > 20 %, and with SO_3 of cement < 3 % and C_3A of the clinker < 8 %
- Application of a performance test on the concrete mix according to (LCPC 2007)
- For precast elements, the concrete mix and heating cycle identical or similar to that having at least 5 references of use in similar conditions without any problem.

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Acknowledgment: The authors thank Jonathan G.M. Wood for his comprehensive review of the English usage and terminology, and his valuable technical comments.

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Forensic Engineering: Informing the Future with Lessons from the Past
ISBN 978-0-7277-5822-4

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doi: 10.1680/feng.58224.401

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Concrete Preservation Plan for Reinforced Concrete University Campus Buildings

John P. Broomfield, DPhil (Oxon), EurIng, CSci, FICorr, FIM^3, FNACE, FCS,
Broomfield Consultants, East Molesey, UK

Abstract
The original campus buildings of the University of East Anglia are grade II and grade II* historic listed 1960s "Brutalist Concrete" structures designed by Sir Denys Lasdun. Since the beginning of this century the University Estates Department has been undertaking a concrete preservation programme to ensure the safety and extend the life of the main teaching wall and other structures.

Corrosion of steel in concrete is the primary cause of deterioration of reinforced concrete structures. The university buildings suffer from deicing salt attack on the elevated walkways, cast in chlorides on some structures and extensive carbonation on others. The problems have to be accurately diagnosed and then treated cost effectively and sympathetically.

This paper will review the evolution of the investigation methods using the latest European standards and simple deterioration modelling techniques to predict deterioration rates and prioritise the concrete repair and rehabilitation programme. The issues of sympathetic repairs to historic listed concrete and investigation and repair work on continually occupied buildings are considered as well as the major parameters affecting the rate of deterioration. The options for repair are considered within the framework of BS EN 1504 Part 9 and the other European standards for concrete repair and corrosion control for reinforced concrete structures.

Introduction
The University of East Anglia was founded in the 1960s and the main campus buildings, including the "Teaching Wall" and the well-known "Ziggurat" residential blocks (featured on the English Heritage website) were laid out by Sir Denys Lasdun. The University is proud of its architecture which has been supplemented by other famous architects.

The "teaching wall" consists of a shallow "W" of reinforced concrete buildings approximately 500 m long, five and six storeys high as shown in Figure 1. The runs of offices, laboratories and lecture rooms are interrupted by lift and stair "towers" at intervals along its length with water tanks and plant rooms above the main building roof level. The exposed cast-in-place concrete façades are a feature of the teaching wall. Various parts of the campus were given grade II* and grade II listing during the process of the works. The sections of the teaching wall and other campus buildings are linked with elevated walkways and bridges.

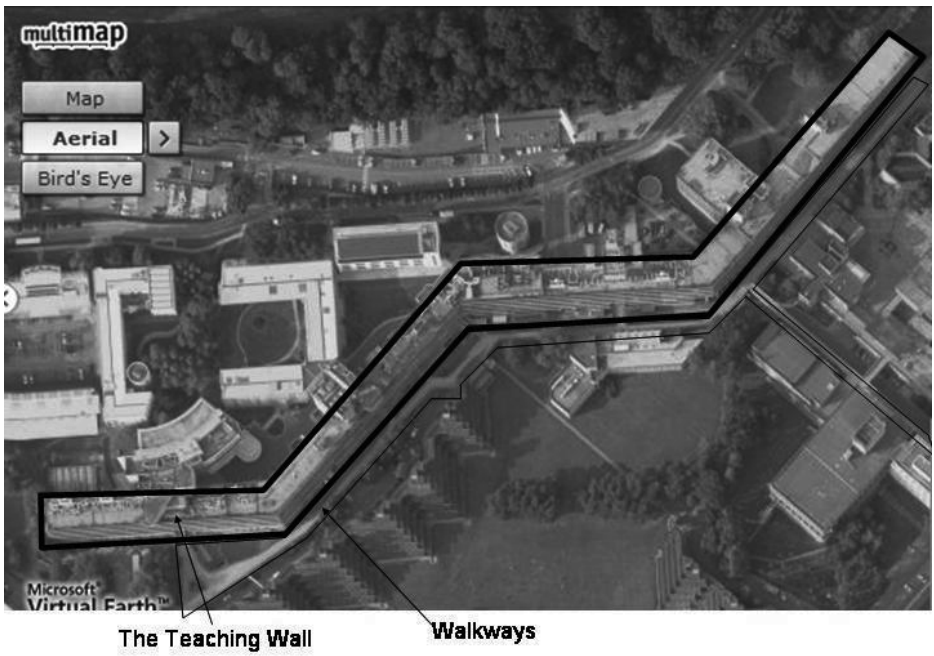


Figure 1: Showing the teaching wall main building and associated walkways.

Applying the Principles of EN1504 to the Rehabilitation Process

While the work was underway, various parts of the new suite of standards on concrete repair, BS EN 1504 parts 1 to 10 were published either fully or in draft for development (DD ENV) form. The set of standards was developed under the Construction Products Directive and is primarily aimed at manufacturers of construction products for the concrete repair industry so that they can achieve "C€" marking. These products can therefore be traded freely within the European Union and the European Free Trade Area. In BS EN 1504 part 9 (2008), a comprehensive approach is given to concrete repair including assessment, repair methods, products and systems. Since it is linked to repair products, the structure of BS EN 1504-9 is not always the most obvious from an engineering point of view. However, it does cover all the options so using it means that nothing should be overlooked.

The University of East Anglia's concrete preservation team made efforts to incorporate the principles and tests methods into the assessment and repair works. This gave some consistency as the makeup of the team has changed over the years. Some of the tests and methods are presented here with hindsight but in later phases the specifications were written to be as fully compliant with BS EN 1504 parts 1 to 10 as the needs of repair to a historic listed building allows.

Under EN1504 Part 9 Section 5.2 the following options are given for the actions to be taken when dealing with a deteriorating concrete structure:

a. Do nothing for a certain time but monitor
b. Re-analysis of structural capacity
c. Prevention or reduction of further deterioration
d. Strengthen, or repair and protect all or part of the structure
e. Reconstruction of all or part of the structure
f. Demolition of all or part of the structure

A number of factors are given for developing a management strategy. The reader is referred to the standard for full details.

Given that the university buildings structures are part of a listed site, that damage had been found and further deterioration could lead to health and safety problems in some areas and that the university has set aside a budget for its "concrete preservation plan" options c) and d) were relevant.

The range of options for intervention on a deteriorating concrete structure is given in Table 1 of EN1504-9 as:

Principle 1 – Protection against Ingress	
1.1 Hydrophobic Impregnation	(1504 Part 2)
1.2 Impregnation	(1504 Part 2)
1.3 Surface coating with or without crack bridging capability	(1504 Part 2)
1.4 Surface bandage cracks	
1.5 Filling Cracks	(1504 Part 5)
1.6 Transferring cracks into joints	
1.7 Erecting external Panels	
1.8 Applying Membranes	
Principle 2 – Moisture Control	
2.1 Hydrophobic impregnation	(1504 Part 2)
2.2 Impregnation	(1504 Part 2)
2.3Surface coating	(1504 Part 2)
2.4 Erecting external panels	
2.5 Electrochemical treatment	
Principle 3 – Concrete Restoration	
3.1 Hand Applied Mortar	(1504 Part 3)
3.2 Recasting with concrete or mortar	(1504 Part 3)
3.3 Sprayed Concrete or Mortar	(1504 Part 3)
3.4 Replacing Elements	
Principle 4 – Structural Strengthening – Not required here	
Principle 5 – Increased Physical Resistance – Not required here	
Principle 6 – Resistance to chemicals – Not required here	

The following principles are specifically related to reinforcement corrosion:

Principle 7 – Preserving or restoring passivity	
7.1 Increase cover	(1504 Part 3)
7.2 Replace contaminated or carbonated concrete	(1504 Part 3)
7.3 Electrochemical realkalization	(TS 14038-1)
7.4 Realkalization by diffusion	

7.5 Electrochemical chloride extraction	(TS 14038-2)
Principle 8 – Increasing Resistivity	
8.1 Hydrophobic Impregnation	(1504 Part 2)
8.2 Impregnation	(1504 Part 2)
8.3 Coating	(1504 Part 2)
Principle 9 – Cathodic control	
9.1 Limiting oxygen content (at the cathode) by saturation or surface coating	
Principle 10 – Cathodic protection	
10.1 Applying electrical potential	(12696)
Principle 11 – Control of anodic areas	
11.1 Active coating of reinforcement	(1504 Part 7)
11.2 Barrier coating of reinforcement	(1504 Part 7)
11.3 Applying corrosion inhibitors in or to the concrete	

The numbers in brackets are the relevant European standard numbers and parts where they exist.

Issues for Historic Listed Buildings

There are obviously additional specific issues relating to historic listed structures in addition to those detailed in BS EN 1504-9. Particular criteria are:

- All historic fabric is precious, not just the original
- Intervention should be avoided if at all possible, therefore minimise intervention
- Necessary intervention should be reversible where possible
- It should be in addition to the existing structure not replace it
- Intervention should be carried out with sympathetic materials (good engineering)
- Avoid destroying historical evidence
- Make clear modern changes while being sympathetic

Most 20th century listed buildings are Grade II listed which comes under the purview of the local authority, although they may include English Heritage in the process. For Grade II* and Grade I structures, English Heritage will be involved.

There is a fundamental conflict between the requirements of conservation and those of achieving a durable concrete repair consistent with BSEN 1504 part 9. It is necessary to show to the Listing authority that the original materials, design and construction are fully characterised. Obviously, if entire elements are being replaced such as a beam, column or panel, it would be possible to replicate the original mix design. However, with concrete repairs it is important that the repair is durable. Therefore shrinkage compensated polymer modified mortars or concretes are required rather than historically accurate mix designs. A logical proposal is therefore required as part of the planning application that shows the understanding of the original construction and materials, the cause of failure due to inadequate construction and materials, in the first place (or the effects of age) and need for modern replacement materials to achieve a safe and durable repair. Trials are then carried out to consider the acceptability of repair options.

When it comes to carrying out concrete repairs due to reinforcement corrosion, it is obviously necessary to remove original fabric, i.e. the damaged concrete. In the case of the concrete elevations of the Teaching Wall and the walkways, it was agreed that, as far as possible, the

original board markings on the elevations should be preserved and that the overall appearance and texture should not be lost behind high build coatings. This therefore excluded the use of conventional high build pigmented anticarbonation coatings. These were trialed on the building and considered unacceptable by the listing officers. An architectural coating with anticarbonation properties was subsequently trialed and agreed to be suitable by the listing authorities and the University.

Phase 1 works, Biotower

Phase I work of the Concrete Preservation Plan was on the "Biotower", a lift and stair tower with air conditioning plant room and a water tower above. Detailed investigation showed low cover and carbonation to be prevalent with some admixed chloride in at least one batch or "lift" of concrete, probably calcium chloride used to speed up setting during winter construction. These problems were all leading to reinforcement corrosion.

A number of options for repair were investigated, including the possibility of cladding the façade and "air conditioning" it to remove moisture and stop reinforcement corrosion according to EN1504 part 9 Principle 1.7, although in this case it is also a method under principle 8. There is a footnote in the standard saying that this method may apply under other principles. However, this was comparatively untried technology and it was considered that no contractor would offer any warranties on such an installation. Corrosion might continue behind the cladding. For that reason, impressed current cathodic protection was applied according to EN1504-9 Principle 10 (cathodic protection). The specification was according to BS EN 12696:2000 "Cathodic protection of steel in concrete" (recently updated as BS EN ISO 12696:2012). Recent developments in anode technology had led to a far wider range of impressed current anodes for cathodic protection becoming available, more suitable for listed structures. The design used probe anodes installed and wired up on the inside face the Biotower so there was no visual impact on the façades.

The advantage of using impressed current cathodic protection are:

- It minimizes the amount of concrete repair required.
- It is reversible, in that if new corrosion technology is developed, the anodes can be removed and the concrete is unaffected to any significant extent.
- It could be applied from inside the building leaving the external elevations unaffected except for the concrete repairs.

One issue was the appearance of repairs. Cathodic Protection was being applied to a number of areas, some of which had previous repairs and new repairs were also required. The existing repairs, even if well matched to the concrete when originally applied, had weathered differently and were very obvious and unattractive. With the agreement of the relevant listing authorities an architectural coating was applied over the concrete facades. This "toned in" the repairs, new and old, while preserving the board marked finish.

Figure 1 shows a "before" photograph with high build anticarbonation trials applied, repairs and exposed reinforcement and the stained concrete surface. Figure 2 shows the finished appearance. The difference between repairs and original concrete is subdued but not invisible and the original board makes can still be seen.



Figure 1a: Biotower plant and tank rooms before treatment



Figure 1b: Biotower plant and tank rooms after treatment

Figure 2 shows the interior of the cowl shaped plant room in Figure 1 with probe anodes installed The connecting wires were run on the concrete surface in conduits according to the client's requirements.

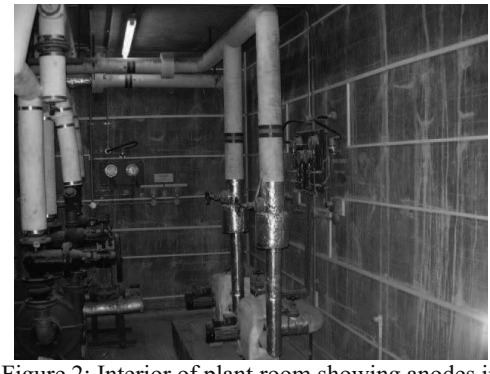


Figure 2: Interior of plant room showing anodes in the wall to protect outer steel with connecting wires in surface mounted conduit.

Phase II Investigation and Repair Works

The Phase 2 works were on the library walkway, shown in Figure 3, a concrete stairway to another walkway showing severe corrosion damage and two further stair/lift towers in the teaching wall showing some deterioration.



FIGURE 3: Walkway showing teaching wall behind. Deicing salts and leachate run down can be seen where the waterproofing and drainage has failed allowing corrosion of the slim pier supports

Investigation

A detailed quantitative condition survey revealed areas of concrete damage due to corrosion from carbonation. This was principally due to low cover, indifferent quality concrete and the age of the structure. Other areas were deteriorating due to deicing salt ingress, particularly on the elevated walkways and access stairways. Using the survey data, calculations were made of ongoing chloride and carbonation ingress on a 30 year life projection.

Corrosion modelling was done by using Fick's law of diffusion calculations on cover depth measurements combined with carbonation depths and chloride depth profiles according to Broomfield (2007). This showed that, other than the areas showing immediate damage, few other areas were found to be susceptible to future corrosion. Table 1 shows the ranges of cover to the reinforcement, the depth of carbonation, the chloride measurements by mass of cement and the half-cell potentials.

Measurement locations where chosen to be representative of the different parts of the construction and the different microclimates, e.g. substructure areas at leaking joints subject to water run down and others where no leakage was seen, side wall areas above and below chloride exposure. It is not feasible to take sufficient random measurements to achieve a statistically valid sample and therefore engineering judgement must be exercised by the engineer writing the survey specification and by the specialist survey company that executes it. The aim of the survey must be clear to all parties to ensure that the results can be used for their intended purpose as well as complying with the specification.

Table 1: Survey Results

	Min. Depth of Cover (mm)	Maximum Depth of Carbonation, (mm)	Chloride, % by mass of cement assuming 14% cement content				Half-cell Potential Values, mV vs. CSE	
			0 to 5mm	**5 to 25mm**	**25 to 45mm**	**45 to 65mm**	**Least Negative**	**Most Negative**
Max	58	50	0.81	0.96	1.24	1.29	317	187
Min	0	0	0.00	0.00	0.00	0.00	-265	-516
Ave'ge	27	14	0.12	0.09	0.06	0.04	118	-1
Std. Dev.	11.57	9.71	0.13	0.14	0.16	0.13	100.73	137.84

The results showed that carbonation induced corrosion had damaged areas suffering from low cover and poor quality concrete (e.g. honeycombing). However, the distribution of carbonation depths measured and the distribution of cover depths shows a low risk of corrosion for a significant time as shown in Figure 4.

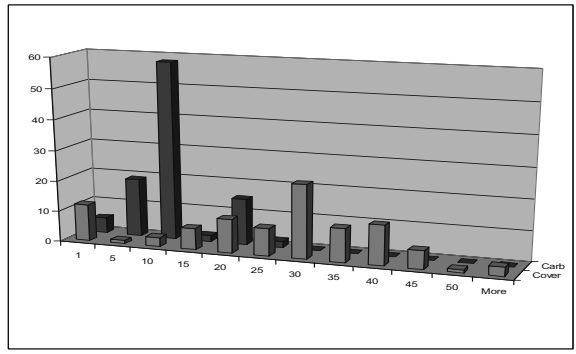


Figure 4: Statistical analysis of carbonation and cover data (depth in mm on Y axis vs. number of measurements of that depth on X axis) showing that generally cover (light blue, front distribution) is far greater than carbonation depth (dark red, rear distribution).

A simple power law equation can be used to model the rate of progress of the carbonation front.

$$x = k \cdot \sqrt{t} \qquad \text{(Eq. 1)}$$

where x is the carbonation depth, t is time and k is a constant for each location. The value of k was determined for each of the 125 measurements shown in the histograms in Figure 4.

The k value was then used to predict the time (or year) for the carbonation front to reach the reinforcement for each measurement location, i.e. the time to initiation of corrosion by calculating t when x is the cover depth in equation 1. The values are shown in Figure 5.

As can be seen from the figure, the vast majority of locations (86 out of 125 = 69%) will not corrode until beyond 2042, while (19%) are already showing corrosion. Therefore 14 locations (11%) should start to corrode between now and 2047. By applying an anticarbonation coating we can slow the rate of corrosion initiation in these locations at risk of corrosion.

The chloride results showed localised problems in certain areas where there was deicing salt leakage or splash onto unprotected concrete. These localised areas were treated with concrete repairs including galvanic anodes and hydrophobic penetrating sealers applied across vulnerable areas where chloride ingress was still possible. The University Estates department also moved over to use of non-chloride deicers to minimise future chloride ingress.

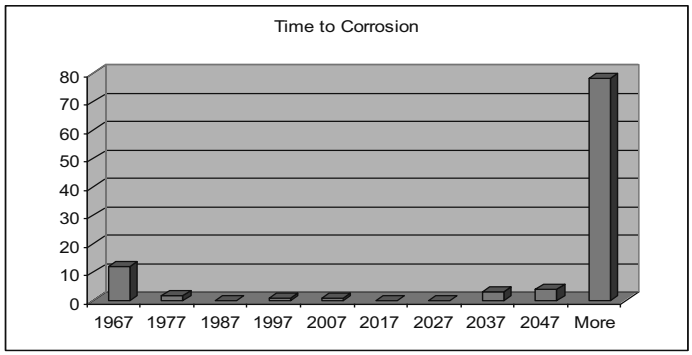


Figure 5: Statistical analysis of time to corrosion showing that areas of low cover and poor concrete quality (high carbonation rate) are already corroding. If these areas are repaired, negligible further corrosion is expected until 2037 or later.

Design and Specification of the Work

Techniques selected therefore included localised galvanic cathodic protection to minimise the incipient anode effect around patches in areas of high chloride (EN1504-9 Principle 10 cathodic protection). Penetrating sealers were required to keep out further chloride according to principles 2.1 and 6.1 and to reduce moisture (Principle 8.1). Anticarbonation coatings were required to reduce the rate of carbonation (Principles 1.2 and 6) and a renewal of the waterproofing membrane on the walkway decks was specified to keep moisture and chlorides out of the deck concrete of the walkways (Principle 1.1). The membrane and improvements of drainage reduced water leakage and protected the walkway substructure from deicing salt run down. These techniques were used along with conventional patch repairs to damaged concrete where required (Principles 3 and 7.2).

TABLE 2
TREATMENTS APPLIED AND THEIR JUSTIFICATION UNDER EN 1504

EN1504-9 Principal	Method/ Principle	EN Standard	Elements Treated	Materials Used
1 – Protection against ingress 8 – Increasing Concrete Resistivity	Hydrophobic Impregnation, EN 1504 Part 2 Principle 1.1 and 2.1	EN1504 Part 2 EN 1062-3 maximum value w =0.035 kg/m^2.h$^{0.5}$	Area below deck level, deicing salts applied & chloride level at rebar below corrosion threshold	Silane compatible with cosmetic coating used to "tone down" repairs
1 – Protection against ingress	Anticarbonation Coating, EN 1504 Part 2 Method 1.3c	EN 1504 Part 2 EN1062-6 Permeability to CO_2 S_D >50m	Parapets on walkways above deicing salts where chloride levels are very low	Cosmetic coating with anticarbonation properties
1 – Protection against ingress	Waterproofing membrane EN1504 Part 9 Principle 1.7	EN1504 Part 9 Principle 1.7 No membrane tests listed in 1504.	Walkway decks	Waterproofing system
3 – Concrete Restoration 7 – Preserving or Restoring Passivity	Hand Applied Mortar, Principle 3.1, 7.2	EN1504 Part 3 Class R4 Compressive Strength >40MPa Adhesive Bond >2 MPa	All Damaged elements	Pre bagged Patch Repair Material
10 Cathodic Protection	Local galvanic Anodes, Principle 10.	Galvanic anodes not covered in 1504 Now in EN ISO 12696:2012	Patch repairs with chloride levels in excess of the corrosion threshold	Zinc anodes encapsulated in a proprietary activating mortar

Detailed analysis of the condition survey results allowed the determination of treatments to different elements of the structures as show in Table 2, using the methodology of BS EN 1504 part 9.

The following specifications were written for the job:

1. A concrete repair specification based on:
    a. Materials according to EN 1504 Part 9 and Part 3 (Class R4 structural grade repair mortar).
    b. Patch Repair Preparation according to EN1504 part 10, section 7 and appendix A7 as well as manufacturer's data sheets.
    c. Materials application methods as given in EN1504 part 10 section 8 on application, Appendix A8 and manufacturer's data sheets.
    d. Special section on galvanic anode application from manufacturer's data sheets. There is now independent specification information in Concrete Society

Technical Report 73 and the new ISO standard now covers galvanic anode cathodic protection BS EN ISO 12696 (2012).

e. Testing on site and of site samples using test methods and values in Part 10 Appendices A7, A8 and A9

2. A coating specification for architectural silicate coating and silane impregnation – based on:
    a. Materials according to EN 1504 Part 9 and Part 2 (1.3C for anticarbonation coating and 1.1(H) and 1.2(l) for silane impregnation for moisture/chloride ingress control)
    b. Manufacturer's literature for application
    c. Surface Preparation according to EN1504 part 10, sections 7 and 8 and appendix A8
    d. Site Testing according to Part 10 Appendices A8 and A9
3. An application specification for a waterproofing membrane
    a. lifting and retaining the original paving slabs,
    b. conducting concrete repairs
    c. repairing and improving drainage and falls
    d. applying new waterproofing system
    e. replacing paving slabs

Site Tests

After applying coatings, cores were taken and sent for testing. Carbon dioxide permeability tests (BS EN 1062-6:2002) gave far better than the 50m (air equivalent) minimum values recommended in the specifications, but they were starting at 23m and 30m, which are reasonably good values.

The water permeability test results were:

Coated	0.03 and 0.04 $kg/m^2.h^{½}$
Partial coated	0.05 $kg/m^2.h^{½}$
Uncoated	0.11 and 0.12 $kg/m^2.h^{½}$

BS EN 1062-3:1998 Table 1 states:

I High	>0.5 $kg/m^2.h^{½}$
II Medium	0.1 to 0.5 $kg/m^2.h^{½}$
III Low	<0.1 $kg/m^2.h^{½}$

Given the requirements for a coating with architectural properties, and the fact that most areas of low cover were repaired, the coated values falling in the II medium range was judged to be an acceptable performance. Also, renewal of the waterproofing and the drainage would reduce the amount of water run down on the substructure, reducing further the rate of chloride ingress.

Pull off tests on concrete patch repairs can be conducted according to EN 1542, ISO 4624 and BS 1881 Part 201 and 207 as described in EN 1504-10 appendix A9.2 Test or observation No 35,. Recommended values are given in EN1504-10 Table A2. In this project, pull of tests achieved 0.8MPa or better.

It should be noted that the works were undertaken prior to full publication of all parts of EN1504 and the associated test methods. Not all testing on this project was compliant with the specific CEN test mentioned but used equivalent British Standards or other tests in use at the time.

Work was successfully completed in 2007. There was minimal disruption to campus activities and both the University and the listing officer were pleased with the final finishes on the listed elevations. Work has continued using comparable materials and specifications. The coating and repairs have performed well and in 2012 work was carried out on the "Zuggurats" that are the student halls of accommodation. The architectural coating used is specified as a colour matched product is available for this particular project and it has been shown to offer carbonation resistance, although not necessarily to the level specified in EN 1062-6 (2008)

Concluding Remarks

Further to the work presented here, the Concrete Preservation Plan has continued around the teaching wall as time and funding allows. The ziggurat structures have been assessed and are now being repaired as part of a rolling programme of work to be carried out in student vacation times over the next few years, budget permitting. The specifications are based on those above and the original repairs and materials continue to perform well.

BS EN 1504 Part 9 provides a methodology that ensures that all repair options are considered when deciding the best methodology for extending the life of deteriorating corrosion damaged reinforced concrete structures. It also provided consistency of specification with changes in the Concrete Preservation Team and changes in products.

The methodology may be aimed at materials but it is comprehensive enough to include other systems that lie outside the purview of the specific products covered in BS EN 1504 Parts 1 to 10.

The methodology of BS EN1504-9 is being used successfully to provide performance specifications for extensive repairs and rehabilitation of Grade II and Grade II* listed university campus buildings. The requirements have been modified to meet the needs of listed structures to cover specific products accepted by the historic listing authorities.

When analysing the condition survey data it was found to be important to combine together the carbonation depth measurements with the local individual cover measurements to ensure that the progress of deterioration could be modelled.

Once with was done, it was clear that carbonation induced deterioration broke down into two time periods, short term damage had already occurred, and long term corrosion which should not initiate for several decades. The ability to interpret the data accurately is lost when cover, carbonation and chloride data are presented as averages, maximum and minimum values.

It has not been possible to go back and check the model predictions in details. However, all repaired areas are in good condition and predicted by the models. There is now a 5 year plan in place to complete the concrete repair works subject to the availability of budgets over that period.

Acknowledgements

The author is grateful to the Estates Department of the University of East Anglia for permission to publish this paper, and particularly to Martin Lovatt who continues to guide the project as it continues to unfold.

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Forensic Engineering: Informing the Future with Lessons from the Past
ISBN 978-0-7277-5822-4

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doi: 10.1680/feng.58224.414

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Gorey Pierhead Jersey, Investigations and assessment of current structural integrity

Steve Hold, Arup, Cardiff, UK

Ray Hine, Ports of Jersey

Ana Ulanovsky, Arup Maritime

Introduction

The evolution of Heritage marine structures provides many engineering challenges in terms of identifying the composition of the structures and their behaviour in modern times. This

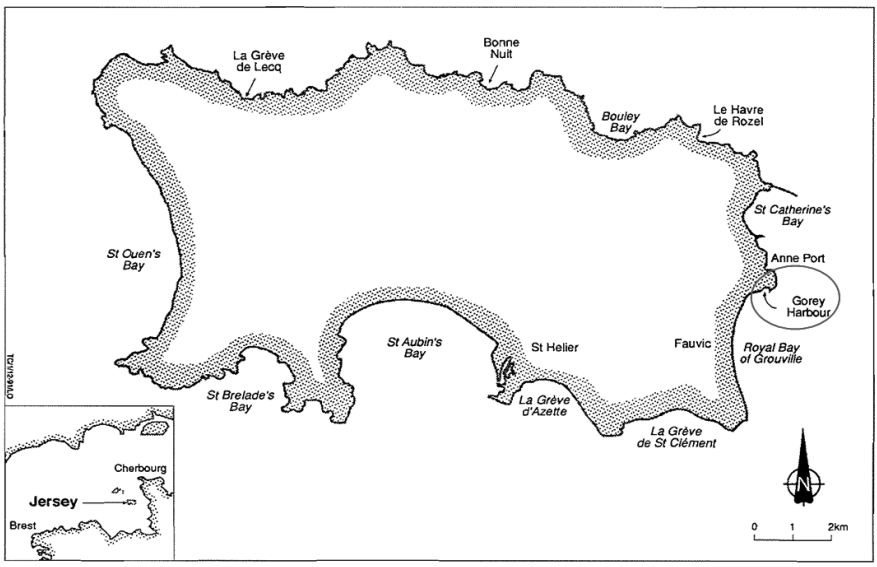


Fig. 1 Locations of Gorey Pier - East Coast of Jersey

case study of investigation work for the end of Gorey Pier on the island of Jersey in the English Channel (Fig 1) provides an insight into the scope of investigation works needed to understand such a complicated structure as well as the lessons learned from patient studied investigation. The Pierhead at Gorey has had a troubled recent past (in heritage terms) with a failure in 1965 and notable deterioration of this asset to the Ports of Jersey in recent years with an increasing risk of another failure. Analysis and research carried out on the Pierhead structure over a 13 year period has culminated in the design of a repair and strengthening project planned for 2013 (not part of this paper) for the vulnerable, vital end elements of the pier.

Invariably, Heritage structures as in the case of Gorey Pier dating back to the 1640s (Fig 2) do not provide their owners or custodians with significant revenue, therefore obtaining funding for substantial repairs and/or strengthening work is difficult. The study work investigating this structure has enabled an understanding of both the evolved construction as well as the behaviour of the pier in such a harsh marine environment. The paper will review the techniques and methods used during the investigation and present conclusions identifying the risks of failure.



Fig. 2 Heritage pier structure beneath Mont Orgueil Castle Castle

Three phases of investigation of the Pierhead

The custodians of the pier Ports of Jersey, formally Jersey Harbours, were aware of settlement cracks and increasing damage to the masonry buildings on the deck of the Pierhead in the late 1990s. The initial investigations were to assess the cracking damage to the building structures founded upon the deck and provide remedial solutions. However, in 2000 desk study research, coupled with a small, intrusive site investigation, which included boreholes and trial pits at the base of the external face of the Pierhead indicated that there were significant engineering deficiencies in the 'conglomerate' structure forming the Pierhead.

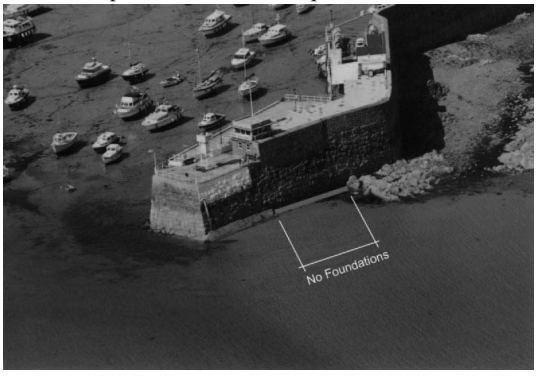


Fig. 3 No foundations between rocks and concrete apron

In 2000, mapping of the various crack patterns of the buildings and the pier deck structures, indicated that a 'shunting' process was probably taking place at the Pierhead. An area on the outside face of the stone pier had no foundations and corresponded with the 'thin' cross section of concrete jetty on the inside face of the pier that became the 'weakest section' of the pier and jetty to resist wave impact on the outside of the pier (Fig 3).

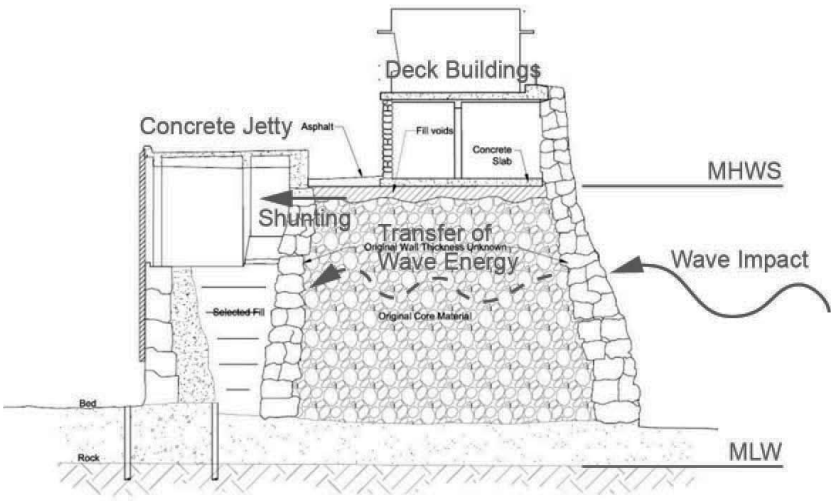


Fig. 4 Typical pier cross section as determined from archives and LiDAR profile before intrusive site investigations

The end of the Pierhead collapsed in a storm in 1965 and this end section was rebuilt with sheet pile mass concrete walls faced with granite. The concrete jetty was also built around this period. The insitu reinforced concrete jetty with two landing stage levels provides buttressing restraint for the original stone pier against wave impacts upon the outside the face of the pier. The jetty built on the leeside of the stone pier abuts and rests upon it (Fig 5).

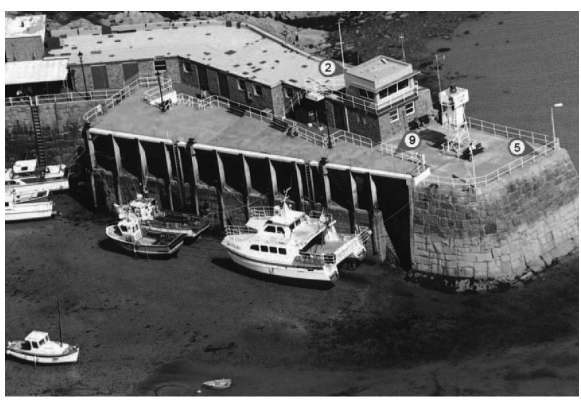


Fig 5 Inside face of Pierhead with concrete jetty, showing some of the main monitoring points

There is evidence seen on both the deck of the main pier and the concrete jetty, that is thought to be the resultant of the wave energy impact on the outside face of the masonry pier traveling through the secondary core material providing an impact force on the stone face of the inner wall of the harbour side of the masonry pier (Fig 4). This transmission of energy (somewhat similar to Newton's cradle) is thought to be one of the main factors 'shunting' the concrete jetty landwards (Figs 6, 7 and 8).

As part of the initial assessment in 2001, it was felt that the rate of the opening up of the cracks appearing on the deck needed monitoring to determine both the rate and significance of the opening up. Locations were chosen across the deck for monitoring using demec studs for levelling and crack width measurement with readings taken at regular six month intervals.

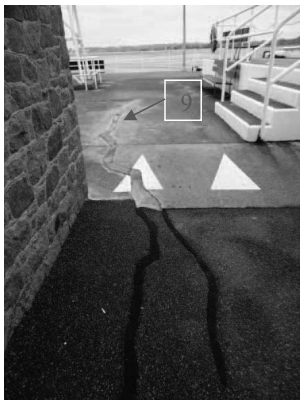


Fig 6 Location of point 9



Fig 7 Crack width studs 2001

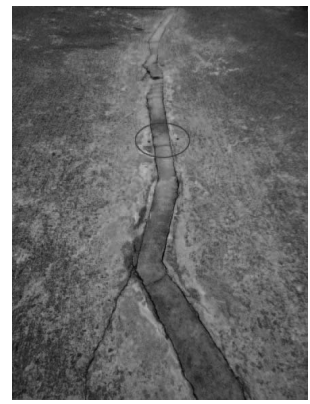


Fig 8 Crack width studs 2006

Key locations were also chosen for regular visual monitoring to facilitate easy regular inspections. Avongarde tell tales were installed over the cracks on the deck's building structures and from 2001 readings taken every six months; once in the warmer summer months and once in the colder winter months (the location of some of these major monitoring points are shown in Figure 5). The monitoring regime continued until 2006 when it was determined that the opening of the crack at point 9 (Figures 6, 7 and 8) was of a magnitude that at that time required a more detailed investigation. The rate of movement in 2001 that had been predicted was that crack widths would open up 1 to 2mm per year due to the "shunting" effects. As seen in the three photographs of deck monitoring point 9 (Figs 6, 7 and 8), an opening up width had significantly increased between the brass monitoring studs (Fig 7 and 8) during the five year period. The crack widths in 2006 were deemed that there was a need for remedial and strengthening works to the Pierhead to maintain the operational use of the Pier.

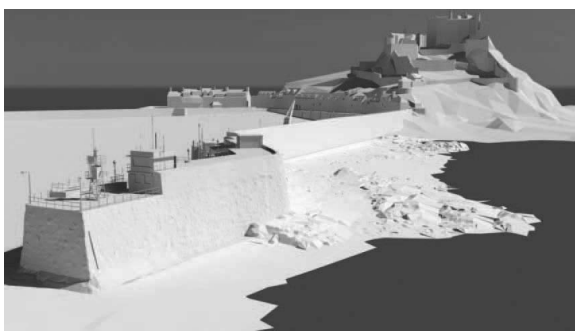


Fig 9 LiDAR model of the Pierhead and the surrounding bathymetry

In 2008 a Ground Penetrating Radar survey was carried out on the asphalt and concrete surfaces at the Pierhead but was not able to identify significant voiding beneath the slab. This was partly due to the depth of the slab and reinforcement within it as well as the discovery later of another previous concrete layer below the slab. In 2009 work was undertaken to determine more accurately the geometry and behaviour of the Pierhead along with the need to produce engineering drawings of the

structural arrangement. The second phase of the engineering investigations used LiDAR techniques to survey both the pier and the surrounding seabed. LiDAR is Light Detection and Ranging, often also referred to as 3D laser scanning that employs a laser and a rotating mirror to rapidly scan and image volume surficial areas. The LiDAR survey in 2009 enabled a 'virtual' model of the Pierhead and the surrounding seabed bathymetry to be modelled (Fig 9).

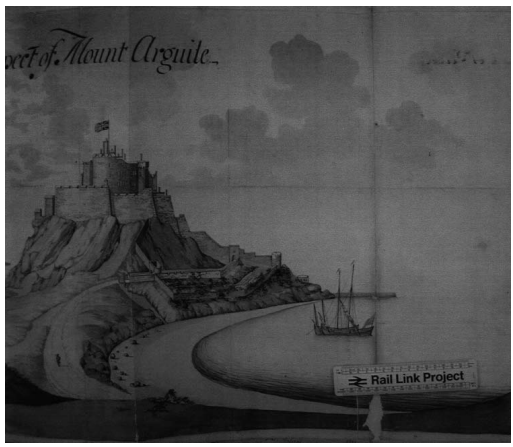


Fig 10 1640s drawing of pier and the castle from Maritime Museum archives

For the third phase of investigation and study Jersey Harbours provided funding for a Feasibility Study that assembled all the original data, historical research (Fig 10) together with the recent measurement and site investigation work. The LiDAR modelling of the evolved Pierhead together with an assessment of the forces contributing to the deterioration mechanisms i.e. load combinations were necessary to identify the areas requiring strengthening. As part of this exercise it was also necessary to determine the sea state conditions that combine tide, wind and wave direction. Then by reviewing the monitoring measurement data it was possible to establish 'cause and effect' and the rate of deterioration of the structure. Once the 'conglomerate' structure's strengths and weaknesses, i.e. structural integrity were understood the sea state wave loadings were applied to the 'virtual model' to help identify possible strengthening methods (Fig 11).

Establishing Deterioration Mechanisms

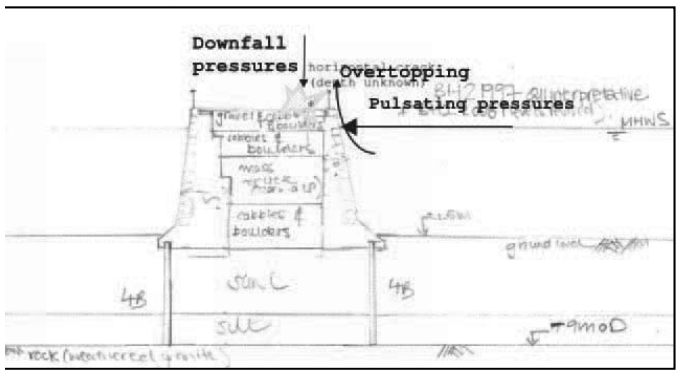


Fig 11 Major wave load conditions cross section diagram at Pierhead

The 2011 Feasibility Study identified that the key loading deterioration mechanisms were related to overtopping of the end of the Pier and wave impact at the outside face of the pier, in particular in the area of no foundations of the outer masonry wall (Fig 3). Historically, voids under the concrete deck slab supporting the buildings had been a problem in 1997/98 and this scenario was identified again during monitoring studies in 2011. It can be seen (in Figs 12

and 13) that after a wave at high tide passes along the face of the pier wall externally (Fig 12), water is seen to then be jetting out from masonry gaps in significant quantities (Fig 13). The conclusion was that the area of the masonry wall allowing such volumes of water to escape so quickly through the joints would correspond to a void internally under the concrete deck slab on the inside face of the wall of the pier.



Fig 12 Wave of 1.0m passing



Fig 13 Water flowing out of masonry after wave passes

It was found in 2000 that ‘wash out’ of core material from beneath the concrete in the Customs Building corresponded to the area of no foundations below on the outside face of the pier where ‘piping’ of fines was seen dramatically being washed out during a trial pit excavation here (piping of fines were being washed out as a result of the pressure head of water remaining in the core behind the pier wall when the tide recedes). The breaking up of the weaker section of the concrete jetty on the leeward of the pier, loss of material from the core and the voiding, beneath the deck floor slab could singularly, or in combination, lead to an inward collapse mechanism. It was identified during the 2011 sea state analysis work that although wave impact was a significant deterioration mechanism likely to be responsible for the ‘shunting', the Pierhead was more vulnerable to the effects of washout and wave impact upon the deck of the pier due to overtopping waves.

A precedent – the catastrophic pier failure of 1965



Fig 14 Newspaper photograph

Although detailed records are not available the collapse of the end of the Pierhead in 1965, the photograph of the newspaper event (Fig 14) indicates a possible similar mechanism of failure took place at the end of the pier then to that described above i.e. the failure was possibly due to weakening of the Pierhead by loss of material through wash out and piping of core material, leading to voiding and consequent instability during a trigger event. In such a structure with a vital navigation beacon at the Pierhead meant that the maritime importance of the collapse in 1965 could not be understated. The rebuilt end of the pier, as researched from the site drawing and records of the time, indicated that the

reconstruction consisted of a foundation of sheet piles driven into the beach, a mass concrete wall sitting on the sheet piles faced with the granite stones that had formed the original outer masonry end of the pier (Fig 5).



Fig 15 Gorey Pierhead from 'Storms in History'

The mass concrete structure was designed to take up load on the concrete nib through to the sheet piles and is visible externally on site only as a concrete apron (Figs 3 and 4). It was in the same era that the concrete jetty on the inside face of the Pierhead was constructed. The reasons for building the concrete landing stage upper deck level approximately 1m above the original concrete deck level are not known but could well be related to our recent sea state analysis findings regarding the need to resist ever increasing wave overtopping. It is speculated that for Heritage planning and visual reasons, the level of the Pierhead had to remain as previous, but that for practical maritime and safety reasons the new concrete landing jetty was built 1m higher for passenger safety. Shortly after this in 1970 the spine of the pier was pressure pointed in the seaward side and then pressure grouted throughout its length to reduce the risk of wash out of core material.

The importance of the concrete landing jetty

The crack monitoring regime originated in 2000 was focussed upon cracks appearing in the rebuilt deck structure at the end of the Pier. Cracks in the concrete jetty, in the deck and buildings together with a large crack at the top of the masonry parapet wall on the seaward side of the harbour master's office were all monitored regularly since that time. Monitoring points are (2), (9) and (5) – (refer back to Fig 5).

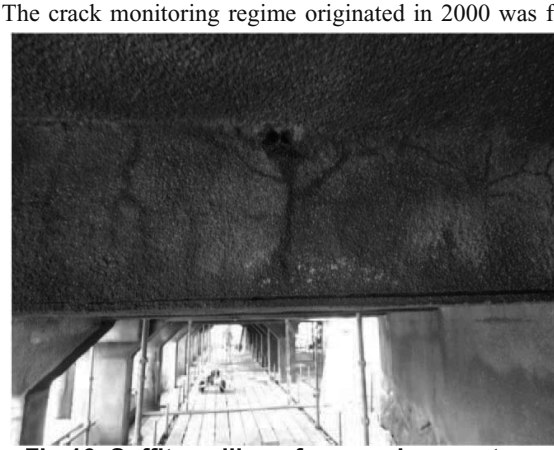


Fig 16 Soffit spalling of sprayed concrete protection

During the initial monitoring period between 2000 and 2006 and the subsequent monitoring periods 2006 to 2011, it was seen that the pier experienced 'shunting' movement of the inner harbour wall

and the jetty of between 1 to 2mm per year. Deterioration was also observed in the spray concrete applied in the mid-90s to 'protect' the 1960s reinforced concrete jetty against the further reinforcement corrosion. The regular monitoring of the corrosion staining and cracking appearing on the sprayed concrete protection, indicated that the jetty had several areas where the lower landing deck was vulnerable to large sections of spalling sprayed



Fig 17 Corrosion of reinforcement in the original



Fig 18 Removal of loose cover, repair and protection of concrete

concrete that posed significant health and safety hazard to lower deck users (Fig 15). It was also possible that the corrosion process of the reinforcement within the original concrete jetty under the sprayed concrete could be accelerating, further weakening the important buttressing effect of the jetty upon the inner original masonry wall at the end of pier. Site wide tests on the concrete jetty related to chloride penetration, carbonation and the delamination of the spray concrete protection during patch repair works in 2012 enabled the conclusion to be drawn that the structural integrity of the concrete jetty had remained intact. Detailed survey work during the patch repairs indicated that the main location of reinforcement corrosion were limited to 'hotspots' on the beam soffits (Figs 17 and 18) with little or no corrosion found in the reinforcement of columns or deck.

Feasibility study for the strengthening and repairs

The LiDAR model allowed production of accurate 2D drawings of whatever part of the Pierhead that needed to be studied further. This, when combined with historical drawing information and further intrusive investigation work in 2012 (Fig 19), allowed an updated cross section of the complicated structure to be produced for analysis (Fig 21).

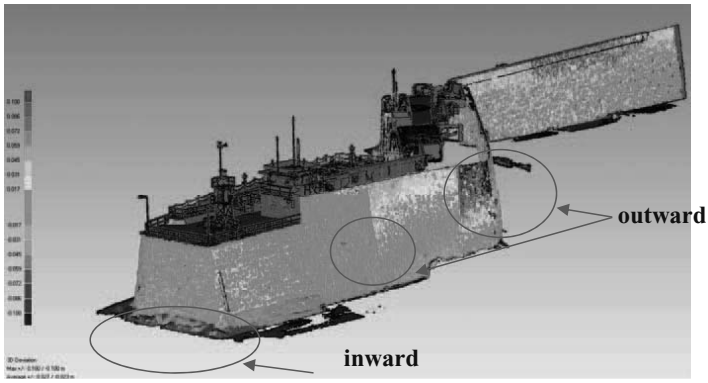


Fig 19 2009 and 2011 LiDAR models superimposed.

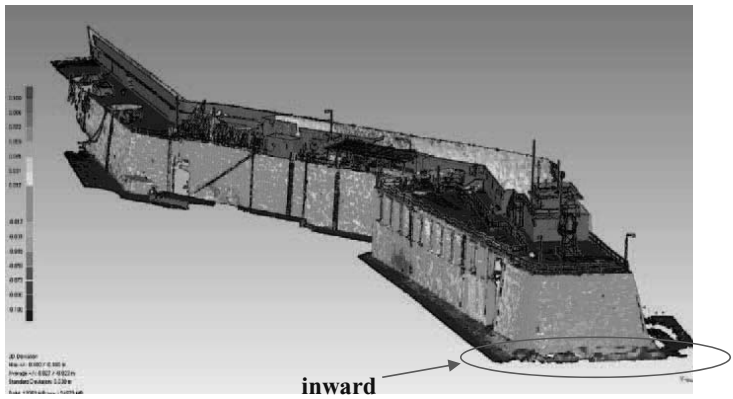


Fig 20 2009 and 2011 LiDAR models superimposed.

As part of a newer approach to the investigations for the third phase work in 2011 a further LiDAR scan was carried out and overlaid upon the original of 2009. Comparisons of the two models that are GPS referenced indicate areas of the surface that have remained in the same position during the intervening two years and areas that had moved. Within the variances and tolerances of the point cloud data interpretation from the data scanning of the two models the direction of external wall movement, at the specific areas could be identified (Figures 19 and 20). Using a colour scale where read is outward movement and blue is inward movement of the new model localised areas of masonry movement could be identified.

The 'bulging' or outward movement seen in Figure 19 coincides with the area where no foundations exist beneath the outside wall. The LiDAR monitoring models also show that the concrete apron of the 1967 rebuild has receded (or disintegrated) considerably in the last two years i.e. the blue shading at the base of the end of the pier.

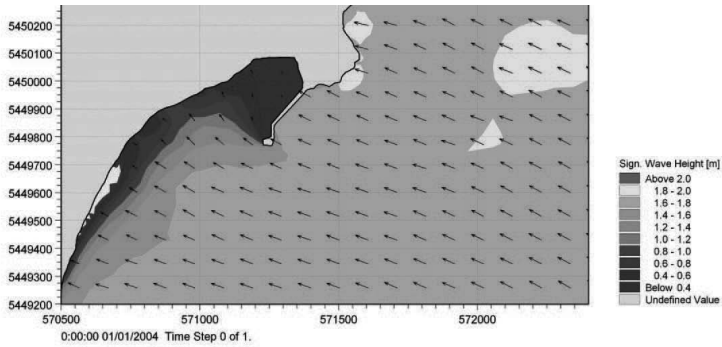


Fig 21 Example of MIKE model of sea state conditions for wave heights

The Feasibility Study considered options for remedial solutions that reinstate support foundations for the outside wall of the Pierhead. The Feasibility Study also discusses the filling of the voids suspected and found during the 2012 investigations beneath the concrete deckslab. The study concludes that should the different types and methods of construction forming the end of the Pier be 'tied' together forming one large structure, they would be better able to resist the predicted worst case wave energy forces and wave combinations summarised and identified using the MIKE model/analysis (Fig 21).

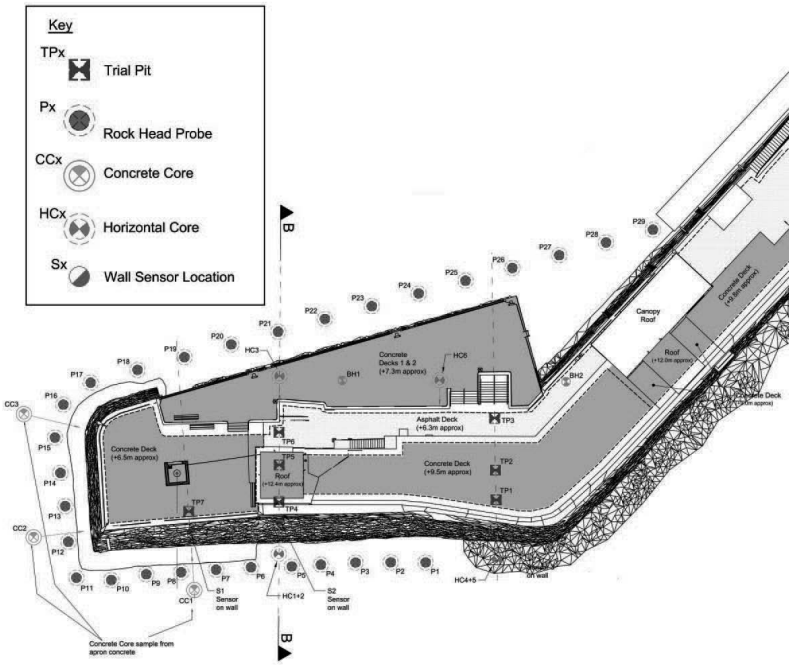
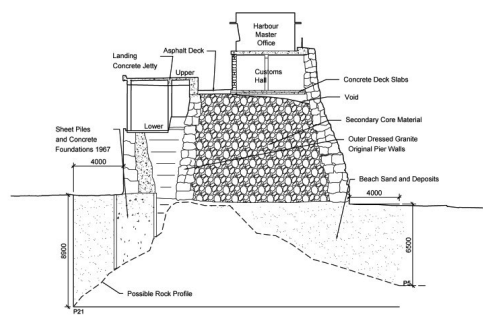


Fig 22 Plan of SI work

Current investigations and experimental work

Further engineering wave energy data is being gathered at Gorey Pierhead to 'value engineer' the number of 'secret fix' ties that will link the old masonry to the new concrete and existing remediated concrete. The concept of a 'secret fix' is favoured by Heritage Planners and Architects where a capping plug of the surface material is removed and replaced after a hole is cored through masonry, a reinforcing rod inserted and bonded to the surrounding surfaces. (in this case the CINTEC system is being used). The 2012 site investigation (Fig 22) included trial pits to determine the extent of voiding beneath the concrete deck, boreholes to determine the thickness and types of materials used but also provided the opportunity for gathering data on wave energy dissipation through the core material of a 1600 – 1700s marine structure. An



experiment using wave energy sensors across the core of the original stone Pier is being trialled as a follow up to the site investigation work to see if energy dissipation across the pier through the core can be empirically measured.

The experiment will try to identify what proportion of wave impact energy is imparted upon the loose masonry of the original inside face of the landward wall and may also provide a better understanding of the 'shunting' process landward of the inner wall and the concrete jetty (Fig 4, the problem). For previous similar strengthening work of heritage marine structures in Jersey at St. Aubins where piers and a breakwater were constructed in a similar way with similar materials, the lack of this type of energy dissipation information assumed that 40% of the impact energy was transferred through the core to the inner face of the inner wall. This conservative design approach is currently necessary until more accurate and site specific data is available either from the experiment described here or other research in the field. A more accurate dissipation figure will help the number of 'ties', anchors or supports required to be less conservative and more accurately engineered.

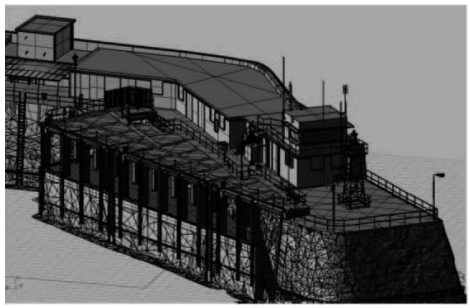


Fig 23 LiDAR model of existing pierhead and deck levels

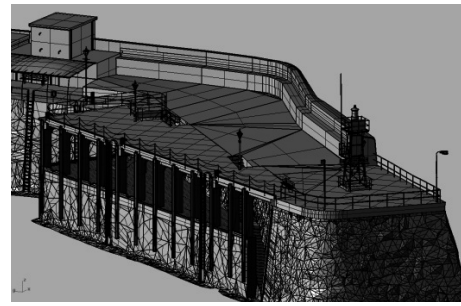


Fig 24 LiDAR model of raised deck level

The combination of all of the investigation methods discussed in this paper have enabled the types and condition of this materials forming the end of the structure to be identified and allow an understanding of their behaviour and interaction under load. The LiDAR model of the structure and the surrounding sea bed has been able to assess the feasibility through visualisation techniques of other methods for protecting and enhancing the life expectancy of the pier and breakwater. The model has allowed consideration of the issues that would be raised by reconstructing the deck level 1m higher to reduce the damage due to overtopping (Figs 23 and 24) and to even consider the protection that would be afforded from an off-shore reef that would reduce wave energy impact (Fig 25).



Fig 25 LiDAR model of off shore reef option to reduce wave energy impact

Summary

The techniques described in this paper have covered the more traditional methods of monitoring and intrusive investigations such as trial pits, boreholes etc., along with prolific desk study and research from various archives that have all helped to build the model of the

evolved marine structure. The long study period and the use of more modern techniques such as LiDAR modelling and now the use of modern 'secret fix', heritage approved, strengthening methods have enabled a repair an strengthening project to be designed for the Pierhead during 2013. The project is intended to reduce the risk of failure and protect the Pierhead and breakwater asset of the Ports of Jersey for a further design life of 50 years.

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Forensic Engineering: Informing the Future with Lessons from the Past
ISBN 978-0-7277-5822-4

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doi: 10.1680/feng.58224.426

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Investigation of the Electrostatic Precipitator Hoppers Detachment Incident at a coal fired power station in Hong Kong

Louis Tang, BEng(Hons) CEng MICE MIStructE, UK

Introduction

On 10th July 2010 a coal fired power station in Hong Kong experienced an incident of structural detachment of the dust hoppers from the Electro Static Precipitators (ESP) on one of the four units. Concern was first raised in the morning when dust was noticed leaking near one of the hoppers joints. Noticeable structural deformations continued and were photographed until final detachment of the dust hoppers and the supporting beams about two hours later. The damage was confined to one precipitator and associated hopper supporting structure. No injury was sustained by personnel in the event and the unit was put out of service for repair and extensive investigation.

Investigators from the United Kingdom, who were appointed to provide support to the local investigating team, were faced with two major problems. Firstly, at the time when the investigator was appointed, the damaged hoppers and supporting structure had already been removed from the scene to a storage area and stabilisation of the affected structure had commenced. This meant that some of the crucial primary evidence was not available to the investigators. Secondly, due to the pressure of an early resumption of power generation on other units which had been shut down because these units shared the same ESP structural configuration as the collapsed structure, a preliminary conclusion was required within the first week of the investigation. The investigation was therefore initially undertaken using numerous record photographs taken before, during and after the incident, by examination of the damaged structure and hoppers in the storage area and by studying witness accounts. Metallurgical testing and a more detailed structural finite element models (FEM) were developed to verify the preliminary conclusion and the sequence of detachment. A further computational fluid dynamics (CFD) model was developed to investigate the effects of any potential flue gas flow /structure interaction.

This paper shares the experience of the investigative team in their approach to identify the root causes and the process required to reconstruct the sequence of events in a tight time schedule and the lessons learnt on the technical issues as well as awareness and mindset towards the challenges of aging plant risk management and controls.

Description of the Power Station

The power station is one of the coal-fired power stations in Hong Kong, China. It was commissioned in 1982. The station consists of four generating units, with auxiliary facilities. Each unit comprises a natural circulation pulverised coal and oil fired boiler and a 350 MW GEC Turbo generator. The power station locates at a coastal site with an extensive area of reclaimed land. The turbine condenser is sea-water cooled. Fuel supplies are shipped to the site by sea and are offloaded at the specially constructed jetties.

The most significant influence on the power block and power station layout was that the reclaimed area would require expensive piled foundations and hence high costs for supporting heavy plant loads. As a result, the power station is arranged such that coal store dominates the reclaimed area, with the heavy power block including the affected ESP plant located on the original on-shore land where rock head is high and near the ground surface.

Description of the Precipitator Plant in detail

The coal used as fuel contains combustible materials, such as carbon and hydrogen, and non-combustible materials, which are deposited as ash during the combustion process. The ash

consists of two main forms, coarser furnace bottom ash (FBA) and pulverised fuel ash (PFA or dust). Fig.1 shows the areas in which ash is formed in a typical boiler plant.

Electrostatic precipitators are used to remove PFA dust from gas streams. They work on the principle that the dust laden gas passes into a chamber containing electrode wires at a high negative polarity dc voltage. The dust particle become negatively charged and are deflected out of the gas stream onto positively charged electrode collecting plates where they are retained until dislodged by intermittent blows on the electrodes – a process referred to as 'rapping'. The particles then fall into the dust hoppers below. (Fig.2)

The electrostatic precipitator involved in this incident was installed and commissioned together with the A1 generation unit of the power station in Year 1982. There were two precipitators (A & B) installed in each boiler unit and the unit was designed to operate with both precipitators in service in normal operation condition, and operate with either one of the precipitators out of service with reduced output (Fig.3)

Each precipitator comprises five rows of dust collecting plates in series and discharge electrode wires laid between each dust collecting plate in the flue gas paths. These are all housed within the main precipitator casing, which was completed with 12 hoppers below (Fig.4). The coal ash collected by the electrodes is periodically "rapped" and then dropped into the hoppers. Each hopper is equipped with an eductor pipe to discharge coal ash into a buffer tank in the generation unit. The coal ash will then be transferred by blowers to coal ash storage silos. The dusting operation is required to be carried out round the clock when the generation unit is on load to prevent potential overloading of the hoppers below caused by dust accumulation..

Sequence of Events

In the morning of 10 July at around 8:05 a.m., a coal plant attendant observed some dust leaking out from the front panel of A1 A side (A1A) precipitator when he walked across the area. He then called to Coal Plant Control Room (CPCR) about the dust leakage.

A shift engineer arrived at the site at around 08:10 and arranged water sprays with the plant attendant for dust suppression. The shift engineer then inspected the leakage and found that the dust was leaking out from the hill-side joint between A1 and A2 dust hoppers. (Fig. 5) He then carried out inspection on the precipitator and confirmed there was one vertical I-column deformed. That I-column was located in the middle underneath the precipitator. He also noted some misalignment of dust and water pipes underneath the precipitator hoppers. He then reported the findings to CPCR and the duty shift manager at Central Control Room (CCR).

They kept monitoring the situation and arranged to barricade the area as they could hear cracking sounds in that area and felt the structure was unsafe.

The duty shift manager instructed to shut down the unit at 9:25 a.m. as the falling hazard envisaged. The shut down process commenced and the A1 unit was put off-load at 10:02 a.m. The boiler purging sequence was still in progress.

At 10:11 a.m., it was noted at the A1 unit control desk that the furnace pressure increased, which might be due to detachment of the precipitator hoppers from the main structure. The

unit purging sequence stopped at 10:12 a.m. A plant attendant reported from the work site and confirmed the A1A precipitator hoppers were detached at 10:15 a.m. However, no body on site witnessed the moment of detachment.

Investigation Approach

A local Investigation Team comprising the power station staff from different disciplines was formed on 11 July 2010 to investigate into the root cause of this apparently unusual event. RWE was engaged as a third-party peer expert to support the local investigation and to undertake an independent forensic investigation of the incident.

The local Investigation Team adopted a Fault Tree Analysis (FTA) technique in the investigation, which is a well-established protocol widely used by the power station in root cause analysis of complex incidents. In reviewing the FTA approach adopted by the local team, it was felt that FTA was a powerful technique which can provide the opportunity to combine human and technical faults into one common framework, hence enabling a complete model of the problem. However, the fault tree developed by the local investigation team focused mainly on the physical / technical evidence. Furthermore, it should be remembered that the FTA method does not actually suggest the root causes, but provides a structure to aid diagnosis. The limitation of using such methods as a stand alone investigation technique is that the causes identified by the analyst may not represent failures at the top level of the system (if causes at level 2 and 3 are identified without investigating why the level 1 systems allowed such failures to occur, then similar or repeat incidents may happen again), thereby not representing root causes.

To complete the overall jigsaw of the 'root cause', all human activities and management issues were taken account of in the RWE's investigation. In the context of possible contributing human / organisational hazards, the detachment incident could be viewed as a failure in the internal control process that led to the materialisation of the hazards. The author, based on his experience in internal audit of business and operational risks, attempted to adopt the powerful COSO risk control model (Ref.1 and Fig.6) in the final jigsaw of the 'root cause' investigation.

Following initial discussion with the local team, the key steps involved in the revised investigation methodology adopted are summarised below:

Collation of Factual Evidence: Significant efforts were spent on soliciting witness statements, and assembling photographic and video evidence at the scene prior to and after the Incident. These provided crucial inputs in helping the Investigation Team to reconstruct how the Incident took place, especially in regard to the sequence of those split-second events which led to the final separation of the hoppers from the precipitator proper. These pieces of evidence are also useful tools to help screening and verifying hypothetical failure mechanisms and scenarios.
Examination of O&M Activities and Records: Detailed reviews were conducted on operation and maintenance practices of the precipitator and its related plant and equipment. Inspection records, plant operation data and defect history were examined carefully to help expose immediate triggers or otherwise concealed contributions to the Incident.
Scenario Examination: Using FTA, combined with the COSO framework of internal control, proposed scenarios on hypothetical failure mechanisms were established and cross-examined

against the evidence. Credible scenarios were identified through matching of hypothetical mechanisms to the collated evidence.
Verification: By using computerised mathematical modelling (Finite Element Modelling, or FEM), the most probable failure mechanism was verified using specialist FEM models. FEM was also used to re-construct the sequence of events leading to the final failure of the structure. The proposed failure mechanism was then verified critically against all the evidence.
Identification of Possible Root Causes: Once verified, the failure mechanism provided critical insight into the contributing factors leading to the failure, and enabled the Investigation Team to identify the contributing root causes, guided by the COSO internal control model.

Investigation Process

Following the initial scoping visit and official appointment, a multi-disciplinary RWE expert team led by the author was assembled immediately in the UK, and three delegates arrived in Hong Kong in early August 2010 for the fieldwork visit and site inspection. At this stage the damaged hoppers and supporting structure had already been removed to a storage area and stabilisation of the affected ESP structure had commenced. The investigation was therefore undertaken using the numerous photographs taken before, during and after the collapse, by examination of the wreckage of the damaged structure and hoppers in the storage area and by studying witness accounts, operation and event log, original design and drawings of the precipitator support structure, as well as results of all available laboratory testing.

The site inspection did not find any evidence of foundation failure. In addition, the fall pattern of the detached hoppers and the wreckage were confined to the footprint of the affected ESP unit. As a result, plant explosion was ruled out as a contributing factor to the incident. With the reported localised debris and the predominantly vertical collapsed mode, the investigation focused on the loading and the pre-existing condition of the structure that would initiate buckling failure of the support structure. An initial risk workshop was held when detailed discussion with the station staff was carried out to identify possible weakness in the control system. The following key risks were identified:

- Inadequate control of the dusting process
- Insufficient control in previous asset condition survey and the defects risk rating
- Insufficient control in preventing damage of key plant support structure by human activities on site (e.g. during outage works)
- Ineffective control of design changes on site during construction of the original precipitator support structure design

By examining all the evidence collected, investigators aimed at all facts being accounted for and none inconsistent with the proposed scenarios, which were subsequently verified by appropriate analytical models. During Phase 1 of the investigation, the investigators were required to report on the initial findings and likely mechanism of failure by the end of the first week of the investigation. As a result, only a simple 2D but realistic structural frame model was used to analyse qualitatively the behaviour of the plant support structure under static loading, and with simple structural calculations, for confirmation of the preliminary findings.

During Phase 2 of the investigation, based on the original construction drawings for the hoppers and the supporting structure, a refined 3D analytical model was assembled using Strand 7, a FEM programme with a time variable which allowed the progressive collapse of the structure to be studied. Finally, CFD analysis and thermal stress analysis using ANSYS were carried out to conclude the investigation.

The Structure

The structure of the precipitator supporting framework consists of 9 steel columns, 8 of which have pins at each end around the perimeter and one fixed column in the middle without pins at each end. The centre columns on each side are braced from the top to the bottom of the corner columns. A ring beam consisting of a steel plate with a channel top and bottom runs around the top of the perimeter columns and a matrix of six beams infill the centre to support the hoppers. The arrangement of these beams is illustrated in Fig.7.

There are a total of twelve hoppers, A1 – A12, laid out as in Fig.8. A1 to A8 being of a similar size and A9 to A12 being smaller.

The precipitator above is a steel box open on two sides where the gas enters and leaves and open on the bottom where the dust falls down into the hoppers. All the plates and wires are supported from the top which transfers the load into the side wall and hence into the bottom frame. (Fig. 4)

Thermal movement of the precipitator structure is allowed by the central column being fixed in position with the rollers at the top and bottom of the other columns being positioned to allow expansion and contraction to and from the centre fixed column.

Stability of the precipitator casing above the supporting columns is provided in the gas flow direction by the side plate-work of the precipitators and across the gas flow direction by portal frames at the entry and exit points.

The wind load at the bottom of the precipitator side wall is transferred into the framework at the top of the columns where all the bolted joints have been welded up to provide rigidity transferring the load to the columns beneath the end walls and the top of the fixed column.

Condition before the Incident

Since the construction of the precipitators in 1981, there have been no reported changes to the structural form of the hoppers. However, it was understood that there had been times when the precipitators had remained unused for lengthy periods which might have led to some internal corrosion at points where condensation was allowed to occur. This would be made worse by the practice of injection of SO3 into the gas flow which is carried out by the station to increase the conductivity of the dust.

Fig. 9 showing the centre column hours before the hoppers structure collapse shows buckling of the entire column in its minor axis. Fig.10 shows buckling to the flanges of the column which is inconsistent with compression buckling and suggests there might have been some sling damage to the column prior to 10th July 2010. Evidence of slings being put around structures was found elsewhere on the station during the site visit by the investigators. This would have seriously affected the strength and lateral stability of the column.

A report by an external civil / structural consultant dated June 2007 highlights several areas of corrosion which required painting but no other significant defects. It was also noted that the defect of the severe corrosion at the base of the centre column was identified, but a low risk rating was assigned to this defect. Further review of the risk assessment rating system adopted by the consultant was considered flaw as the system gives priority to defects that could generate hazards to people such as falling debris from height. As a result, all defects at low level were rated as low risk, regardless of the structural significance of those defects.

The investigation revealed that dusting problems had occurred with unit 1A for a week prior to the incident, manifesting itself in the tripping out of zone 2 (Fig.8) and later by zone 1. This was probably due to high dust levels in the hoppers shorting out the wires and plates above. This could have arisen if there was no reliable way of checking the dust level in the hopper and how much dust has been removed from the hoppers during the dusting process. Station management was aware of this control reliability issue, and O&M practices have been enhanced since the incident to improve dust level monitoring and dusting system effectiveness and availability in order to keep the dust level low within the precipitator hoppers.

Metallurgical Testing

Overall, metallurgical testing results indicated that the welds between the hoppers and the support structure generally looked only to have failed by tensile overload during the final detachment, suggesting they had not been a major contributing factor to the root cause of the incident. However, there was evidence to suggest that this structural failure could have been developing over a long period of time prior to the incident, with some of the hopper supporting beam welds cracked some time ago.

Furthermore, the investigation of the buckled area of the central column IB14 revealed that a transverse crack of 40mm in length and approximately 6.5 mm deep was present on the flange. The crack pattern and length suggests that this could be a corrosion fatigue crack. There were other smaller cracks initiating from pits, which are consistent with a corrosion fatigue mechanism. The presence of the crack on the apparently concave section of the buckle (compressive region) means that the crack might not have been caused by this incident, but could be present before. For a corrosion fatigue crack to have formed in this area, this region must have been periodically tensile some time prior to the incident. The presence of this crack therefore suggests that the loading on the column has been variable and that the column may possibly have been ‘flexing’ or ‘swaying’ according to loading and unloading situations.

The Hypothesis

By examining all the factual evidence, including witness statements, metallurgical testing results, and the forensic investigation of the fall pattern of the hoppers, the deformed shapes and nature of damage of the structural members as revealed from the wreckage, the investigators were able to reconstruct the likely sequence of collapse. The following summarises the assumed mechanism and sequence of failure:

- Buckling of the hopper central column led to significant downward movement of the hoppers’ support grillage and consequential tearing of the bottom flange off the ring beams

- This in turn led to dust leakage at the top of the hoppers A1 and A2, with separation of pipes on hoppers A1, A2 and A6, etc.
- Progressive failure of the end connections on the grillage members then led to the complete detachment of the hoppers from the support structure.

Verification (Incident Reconstruction with computer FE modelling and CFD analysis)

Analytical FE Models

A finite element package, Strand 7, was used to help develop an understanding of the structural behaviour of the dust hoppers and their supports. Models were developed to examine the basic elastic performance of the structure and to show the behaviour of the structure throughout the incident on 10th July 2010.

Analysis has been carried out in three stages:

1) Calculation of connection and member capacity to allow the model to be checked and connections strengths to be included

2) An elastic model was developed which demonstrated the structural behaviour of the hoppers working normally and allowed the calculation of an allowable dust load. Dust loading higher than this would cause the central columns weak axis compressive strength to be exceeded.

   Based upon this analysis it was suggested that the structure had not been originally designed to carry the amount of dust which the precipitator structures were occasionally required to hold if the dusting operation is stopped or was ineffective.

3) The collapse sequence was then simulated through a finite element detachment model. The model presumes that the central column fails first and rapidly buckles in accordance with the failure mechanism. This is simulated by rapidly decreasing the stiffness of the column and allowing it to deflect downwards.

The detachment model allows the progress of the connection and hopper weld failures to be followed through the sequence. Further robustness checks confirmed that any initial local beam joint failure would not lead to the disproportionate progressive detachment of the hoppers supporting structure. The simulation model successfully verified the assumed sequence of failure put forward in the hypothesis. (Fig.11)

Thermal Stress Effects

It was required to look at the behaviour of the central support column under the expected thermal loading on top of the hopper weight loads (especially the central column lateral displacements). A bounding thermal load is assumed to be metal temperatures of 150oC at the beams at the very top of the hoppers and above (height ≥7700mm) representing forced convection from the flue gas and 30oC below 7700mm.

The STRAND7 model was exported to ANSYS to model the central support beam discretely with 3-D solid elements instead of beam elements in order to be able to look at stresses and displacements in more detail..

Three cases of expansion joint conditions were analysed:
Case 1: All expansion joints operational
Case 2: Expansion joint at top of external column on the gas inlet side constrained*
Case 3: Expansion joint at top of external column on the gas outlet side constrained*
* constrained in gas flow direction and cannot rotate about direction normal to gas flow

For these cases, there was no indication of any significant thermal stress effects induced by expansion joint restraint that might have led to the hoppers detachment incident
However, restraints at the gas inlet or outlet side could induce a 6 – 9 mm lateral displacement at the top of the central column. This would magnify the bending effect on the slender column due to the resulting eccentricity of the ash weight loading at the top of the column.

Computational Fluid Dynamics (CFD) Model Analysis

The aim of the CFD analysis was to see if there were any fluid dynamic effects that might have contributed to the failure.

The CFD analysis was based on ANSYS-CFX v 12. The computer model represents turbulent flow in the ESP itself plus inlet and outlet ducts. No attempt was made to represent electrostatic effects or temperature effects. The aspects considered in the CFD model included (i) inlet flow conditions and location of inlet plane, (ii) turning vanes, (iii) inlet baffles, (iv) simplified representation of ash collection plates and (v) gas guides. A general view of the model is as shown in Fig.12.

The following cases were modelled

- Case 1. with all internal fittings and empty ash hoppers
- Case 2. As Case 1 but no inlet baffles (modelled by putting pressure loss coefficients to zero - see Section 2.3 of the full report)
- Case 3. As Case 1 but with ash hoppers full
- Case 4 As Case 1 but with ash levels based on that observed at the power station
- Case 5 As Case 4 but without inlet baffles.

The predictions that included the effect of the inlet baffles (case 1, 3 and 4) suggested that the baffles were reasonably successful in straightening the flow at inlet to the ESP. For these cases, there was no indication of any fluid dynamic effects that might have led to the failure.

The predictions that ignored the effect of the inlet baffles (case2 and 5) showed that the gas flow was predominantly near the bottom of the ESPs, with significant flow into the first row hoppers. This might have contributed to the failure, by depositing more ash into the first row hoppers.

The level of ash in the hoppers was predicted to have only a secondary effect on the gas flow in the ESP itself.

The very non-uniform flows predicted with no baffles could lead to poor ESP performance in emission standard.

Contributing Root Causes

Guided by the COSO risk control model and based on the findings of the investigation, contributing factors to the root cause of the incident were identified as follows:

Generic factors (applicable to all ESP units on site):

- Structural Design and Construction: There were apparent inconsistent design details of the hopper support structure from the OEM drawing. The central column was built and orientated with the weak section capacity to resist the bending effects from the grillage beams above. It was suspected that design changes might have been made on site during construction.

- Loading Condition: From the wreckage and the inspection of unit A1B after the collapse, there was evidence to suggest that high ash weight was possible in the A1A hoppers at the time of collapse.

  Although dusting was carried out right before the incident, there was an uncertainty in the performance of the dusting system of which the hoppers dust level could still remain high after the dusting operation.

  Other causes of excessive loading on the central column include:

  - the fact that the dusting transfer system was taken out of service in the early hours of the morning due to pipe burst, but boiler operation continued for up to 8 hours afterwards. Typical ash (or dust) content of coals is approximately 10% by weight. Therefore, with a boiler burning 2,000 Tonne of coal per hour, dust production would be 200 Tonne/hour. At this dust generation rate, up to 1,600 Tonne of PFA would have been collected in the unit A1 (A&B) hoppers, or over 800 Tonne of ash in the collapsed A1A hoppers.

  - the dust discharge assistance was provided by high frequency external vibrators which could cause dust compaction, which means that the loads could be higher than would normally be the case with the same volume of dust.

  - roof leakage or ESP casing insulation failure could let cold air enter into the ESP; the cooling air could have caused condensation which led to the wet particulates fusing together into a considerable mass, resulting in a higher bulk density than the dry ash density assumed in the original structure design.

Specific factors (unique to the collapsed unit A1A):

- Exposure Condition: Unit A1A is the most exposed to weathering effects caused by the surrounding environment. Investigation of the central column wreckage found corrosion at the web plate of the lower end. Inspection was conducted for other precipitators and no similar deterioration could be found.

- Construction Workmanship: Unit A1A was built first among other units. According to construction industry experience, this was likely to have poorer construction quality

and workmanship. Weld joint between the central column and its bottom end plate was poor. The weak weld joint could lead to a reduction of the buckling resistance of the central column, which in turn could result in the deterioration of the loading capacity for the whole structure.

- Pre-existing Damage: There was evidence of likely pre-existing damage to the central column potentially caused by previous maintenance activities. Evidence of slings being put around structures was found elsewhere on the station during the site visit by the investigators. Also, the metallurgical testing revealed the existence of a pre-existing transverse crack on the flange of the central column at the buckled region.

Conclusion

Having looked into the contributing root causes in conjunction with the failure mechanism, the Investigation Team concluded that the incident took place as follows:

- The detachment of the A1A precipitator hoppers was caused by deformation of the central column which had already been weakened by poor fabrication quality, corrosion due to weathering and pre-existing damage caused by human activities in the past.
- The effectiveness of the dusting process on the A1A hoppers was likely to be compromised, due possibly to reliability issues of the level indication and ingress of moisture. Ash was allowed to accumulate inside the hoppers despite regular de-dusting, which in turn overloaded the already weakened central column to deformation.
- Buckling of the central column led to the downward movement of the steel grillage beams at the top of the hoppers and tearing of the bottom flange of the ring beams, hence leading to dust leakage at the top of the hoppers A1 and A2, with separation of the eductor pipes on hoppers A1, A2 and A6, etc.
- Progressive failure of the end connections on the hopper supporting grillage members then led to the final separation of the hoppers from the precipitator casing on top.

The Incident was therefore most likely caused by the combined effect of these contributing factors: (i) poor workmanship of the central column base plate weld, (ii) weathering-effect-induced corrosion, (iii) pre-existing defects at the mid span of the column, and (iv) ash accumulation inside the hoppers.

Lessons Learnt

The use of the conventional FTA technique in conjunction with the COSO risk control model provides a powerful and effective root cause investigation approach.

The root cause investigation, guided by the COSO model, has successfully identified a number of control weaknesses in the ESP plant operation. Improvement recommendations were drawn up to address the contributing root causes identified. The following provides a brief account of these improvements:

Management of Design Changes made during Construction

Review the current controls and procedures of plant structure design check and design change control.

Management of Contractor Activities on site

Review the current controls and procedures of managing contractors working on site, in particular, the prevention of potential damage caused by slings putting around plant supporting structures.

In addition, adequate controls shall be in place to ensure the quality of workmanship for works with structural significance.

Inspection and Maintenance

- Review the risk rating methodology used in the civil asset survey inspection to ensure that structurally significant defects are prioritised.
- As part of the routine condition survey programme for the life of the station, thoroughly inspect all other units for potential damage caused by maintenance activities in the past.
- During next outage, programme in to carry out the following works:
  - roller block expansion joints, gas inlet baffles within the ESP and hopper supporting beams structural welds (by NDT) on the other ash hopper structures
  - repair all reported leakage at the roof or casing of the ESP units

De-dusting Operation and Ash Level Monitoring

- Review the current controls and procedures of the de-dusting operation to ensure that the hoppers are emptied on a regular basis
- Review the emergency procedures for disruption of de-dusting operation due to pipe burst
- Ensure the reliability and effectiveness of all ash level monitoring devices
- Removal of the vibratory discharge aids and replacement with fluidising mechanisms.
- Ensure that water or moisture from the ejector is prevented from entering the hoppers above. If the wet extraction system must be retained (because of existing downstream systems), provision of either a suitable break tank, or a water sensor upstream of the eductor to ensure that water can never enter the precipitator hoppers.

References

1. Committee of Sponsoring Organisations of the Treadway Commission (COSO) (1994) Internal Control–Integrated Framework, American Institute of Certified Public Accountants, Jersey City, NJ.

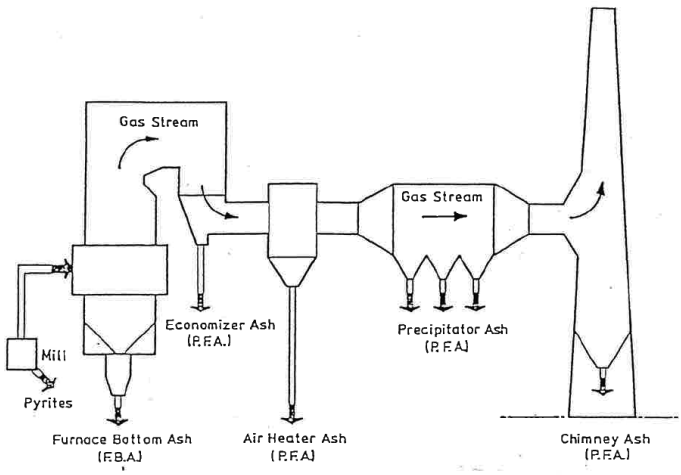

Gas Stream
Gas Stream
Economizer Ash
(P.F.A.)
Precipitator Ash
(P.F.A.)
Mill
Pyrites
Furnace Bottom Ash
(F.B.A.)
Air Heater Ash
(P.F.A.)
Chimney Ash
(P.F.A.)

Fig. 1

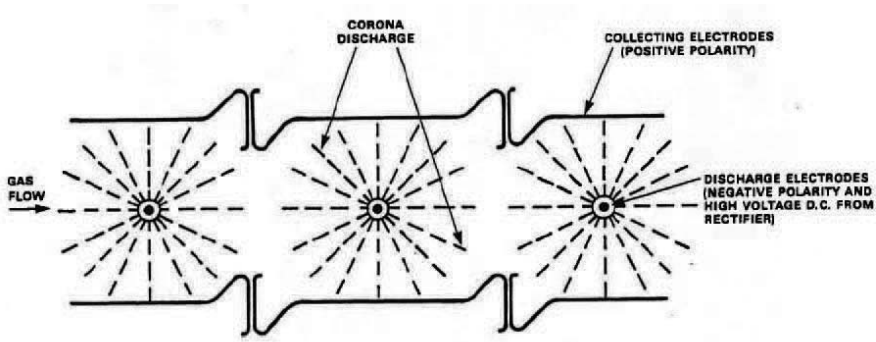

CORONA
DISCHARGE
COLLECTING ELECTRODES
(POSITIVE POLARITY)
GAS
FLOW
DISCHARGE ELECTRODES
(NEGATIVE POLARITY AND
HIGH VOLTAGE D.C. FROM
RECTIFIER)

Fig. 2

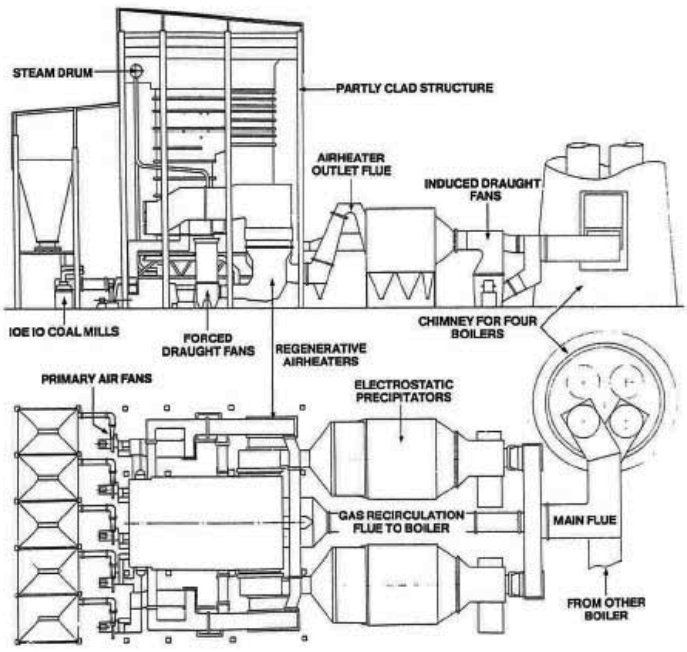

STEAM DRUM
PARTLY CLAD STRUCTURE
AIRHEATER OUTLET FLUE
INDUCED DRAUGHT FANS
CHIMNEY FOR FOUR BOILERS
IOE IO COAL MILLS
FORCED DRAUGHT FANS
REGENERATIVE AIRHEATERS
PRIMARY AIR FANS
ELECTROSTATIC PRECIPITATORS
GAS RECIRCULATION FLUE TO BOILER
MAIN FLUE
FROM OTHER BOILER

Fig. 3

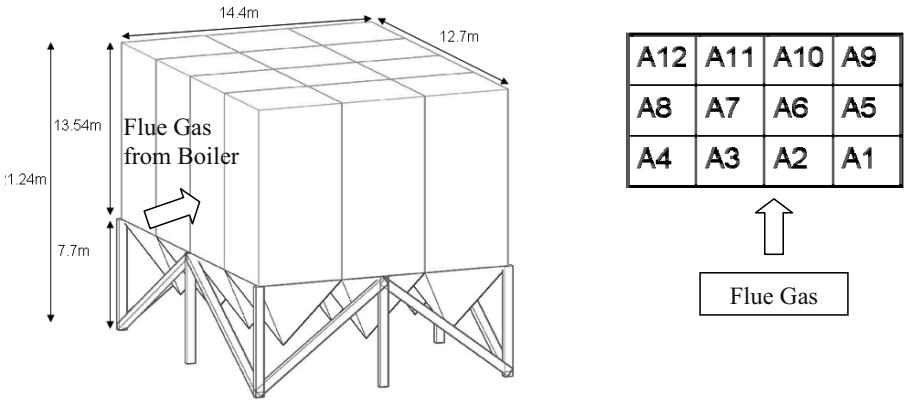

14.4m
12.7m
13.54m
Flue Gas
from Boiler
7.7m
A12
A11
A10
A9
A8
A7
A6
A5
A4
A3
A2
A1
Flue Gas

Fig. 4

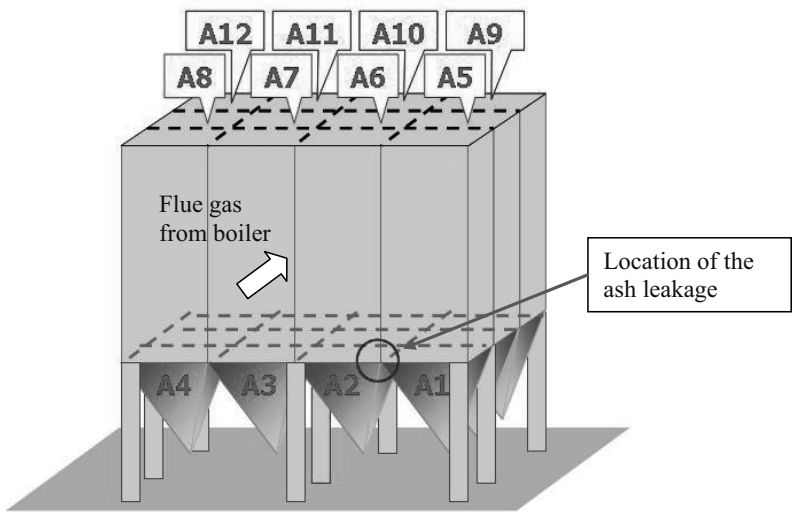

A12
A11
A10
A9
A8
A7
A6
A5
Flue gas
from boiler
Location of the
ash leakage
A4
A3
A2
A1

Fig.5



The *control environment* provides an atmosphere in which people conduct their activities and carry out their control responsibilities. It serves as the foundation for the other components. Within this environment, management *assesses risks* to the achievement of specified objectives. *Control activities* are implemented to help ensure that management directives to address the risks are carried out. Meanwhile, relevant *information* is captured and *communicated* throughout the organization. The entire process is *monitored* and modified as conditions warrant.

Fig. 6

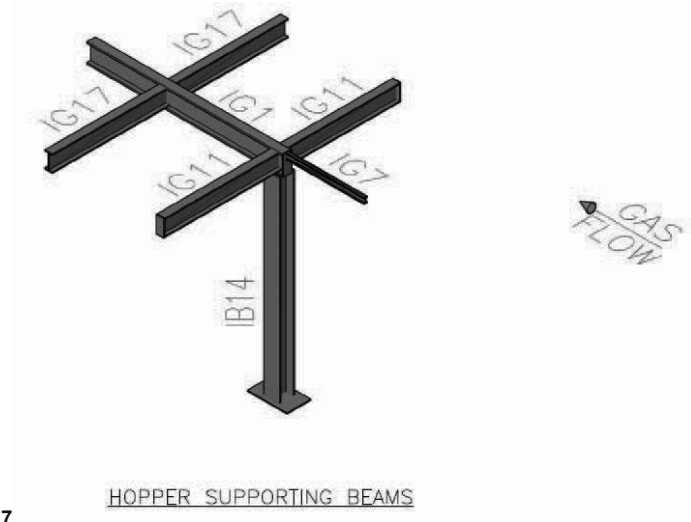

IG17
IG17
IG1
IG11
IG11
IG7
IB14
GAS FLOW
HOPPER SUPPORTING BEAMS

Fig. 7

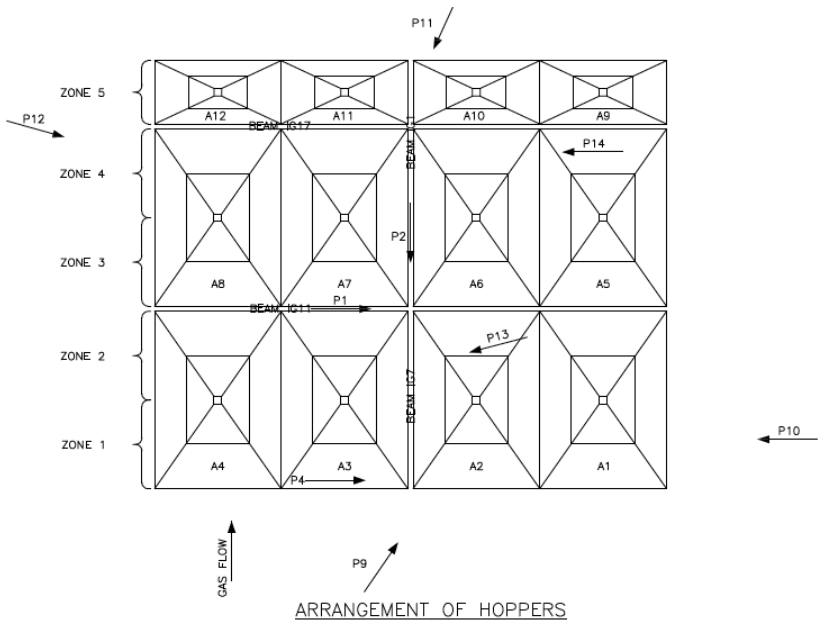

P11
ZONE 5
P12
A12
BEAM IG17
A11
A10
A9
BEAM IG1
P14
ZONE 4
ZONE 3
P2
A8
A7
A6
A5
BEAM IG11
P1
P13
ZONE 2
BEAM IG7
P10
ZONE 1
A4
A3
A2
A1
P4
GAS FLOW
P9
ARRANGEMENT OF HOPPERS

Fig. 8



Fig.9



Fig. 10

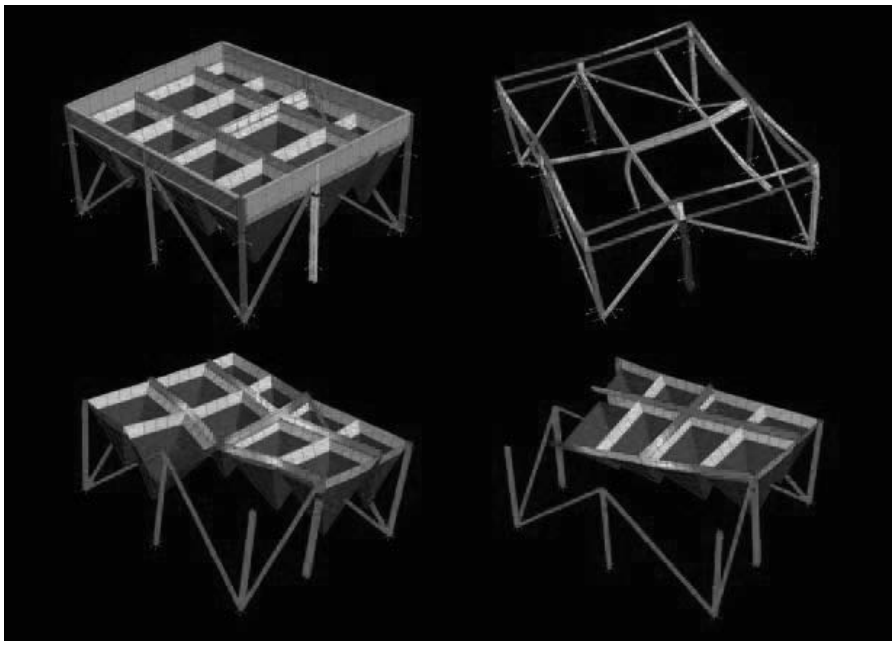


Fig. 11

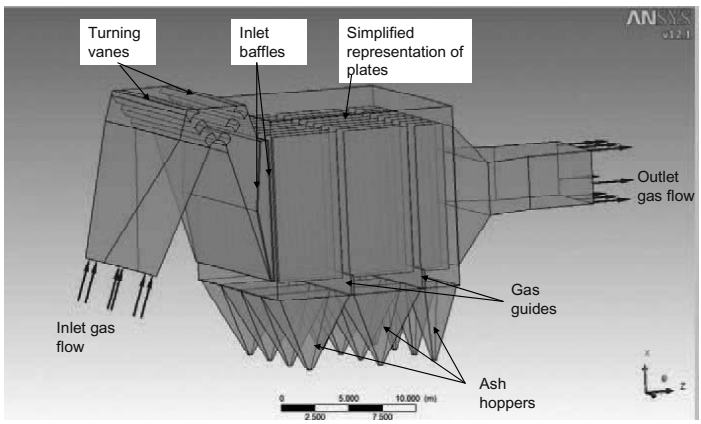

Turning vanes
Inlet baffles
Simplified representation of plates
Outlet gas flow
Gas guides
Inlet gas flow
Ash hoppers
ANSYS
v12.1
0
5.000
10.000 (m)
2.500
7.500

Fig. 12

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Forensic Engineering: Informing the Future with Lessons from the Past
ISBN 978-0-7277-5822-4

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doi: 10.1680/feng.58224.447

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Post-Tensioned Structures – Improved Standards Based On Lessons Learnt

G M Clark, CEng FICE, FIStructE, Director Ramboll UK, past Chairman Gifford, Deputy President *fib,* Southampton, UK

Introduction

Between 1979 and 1992 the UK Standing Committee on Structural Safety (SCOSS) published annual reports (SCOSS, 1979-1992) which included reports on suspected deficiencies of grouting of post-tensioning tendons and in 1992 the UK Department of Transport banned ducted grouted post-tensioning in bridges. This marked the beginning of a process of investigation and review of standards and procedures which have lead over 20 years to national and international bodies introducing improvements to help ensure the safety and reliability of post-tensioned structures. This is a prime example of where industry had to learn from the past, and has used that knowledge to introduce improvements, culminating in the publication (expected in 2012) of new ISO standards.

There are significant lessons to learn from this for structural designers, material scientists, and owners of structures both for new designs and for asset maintenance.

Historical Background of Post-Tensioning

Post-tensioning is a relatively young technology compared with reinforced concrete, and dates from the mid 20th century when due to shortages of steel during and after the Second World War, it rapidly penetrated the market based on being a competitive and economic form of construction and many different systems were developed and introduced around the world. Significant savings in materials were achieved. The theory and history of post-tensioning is well documented (Allen 1986) and, putting aside pre-tensioning, which is outside the scope of this paper, it mainly falls into two types – external and internal post-tensioning, with both being pioneered and leading to distinct structural forms.

Initial developments tended to use individual stretched wires to impart pre-compression force into concrete, as pioneered by Freyssinet, but the search for greater force rapidly led to development of spirally wound wires to make strands. These emerged as 19 wires in the 1950's and 1960's but have evolved into a mature product of 7-wire strands, due to more reliable technology to grip them and hold the tensioning force. CIRIA Report 106 (CIRIA 1985) contains a review of prestressing systems in the UK.

High carbon high strength steels are used to provide this tensile strength to concrete structures in the parts subjected to tension, with the concrete itself providing the compressive resistance in compression zones. However these high strength steels are more sensitive to corrosion in the presence of chlorides than normal reinforcement, and the common use of de-icing salts on roads has caused problems.

Common to most systems was the need to inject cement grout into the ducts to protect the steel from corrosion and, for internal tendons, to provide bond to the structure. Ducts for internal tendons were seen as a means to provide a conduit to place the steel where it was needed, and were sometimes initially simply cardboard tubes, or formed by inflated rubber tubes, but later became more commonly of steel. Some early systems of external post-tensioning, often originally used without ducts, relied on surrounding the tendons in cement mortar or fine concrete after tensioning, with or without bonding to the structure.

Other systems rely on grease and plastic sheathing to provide corrosion protection. Early use included polyvinylchloride (PVC) sheathing which was subsequently found to be susceptible to release of hydrogen ions and damaging to high carbon steels. Modern plastic sheathing is normally polyethylene (PE) or polypropylene (PP). External post-tensioning systems usually had purpose cast or forged steel deflectors connected or cast into the concrete structure to deviate the tendons.

One could describe such post-tensioned structures as “highly strung” in that they are finely tuned by the Designer to provide the tensile strength exactly where it is required, by designing the profile or alignment of the tendons to match and counterbalance the expected stresses in service. As such there does tend to be less scope for error in such structures, or less redundancy or reserve of strength, although this depends on the disposition of steel reinforcing bars and the structural form. For example precast concrete segments held together simply by post-tensioning are totally reliant on the prestressing, whereas an in-situ concrete structure with continuous reinforcement has some degree of additional capacity.

The development of prestressed concrete bridges in the UK is described by Smith (1996) and there is a useful summary of the Motorway applications by Sims (2000) in the Motorway archive.

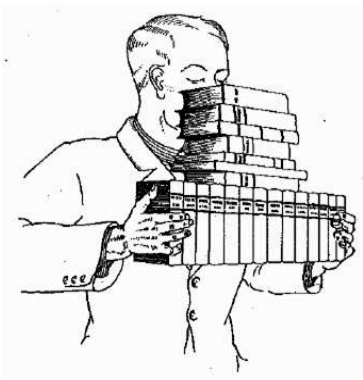


Fig 1 Principle of prestressing

The classical illustration used by Magnel [Fig 1] explains well the principle of post-tensioning showing a pile of books held each in vertical position as a horizontal beam by arm pressure; if the force is released the books would collapse unless there was any other tensile element.

In those early developmental days there would have been no anticipation of the potential impact of de-icing salts on our roads and how consequential chloride penetration could cause serious damage. There could also have been little thought that cement grout may not provide long-lasting protection to the vitally important tendons for the life of the structure, nor to how they might be inspected during future years. Many of the failures can be attributed to chlorides being channelled to the prestressing steel or due to poor grouting leaving voids in the ducts.

Review of Problems

From quite early on there were problems discovered in some UK post-tensioned structures, primarily bridges, for various reasons. Where inspection was possible, often this was attributable to leaking of water reaching the tendons of external post-tensioned bridges [Figures 2, 3]. Other problems related to materials or were unaccounted for. Two significant externally post-tensioned bridges, Braidley Road Bridge in Bournemouth and the A3/A31 Bridge over the A3 in Guildford, both had their external tendons completely replaced in the 1980’s. The fact that such replacement could be achieved is a pointer to one of the major advantages of external post-tensioning. In the case of Braidley Road Bridge, importantly, traffic continued to be able to use the bridge during the replacement works.



Fig 2 External Prestressing. Note one failed tendon visible on the floor in centre frame



Fig 3 Severe Corrosion to external tendon

Early recorded failures of post-tensioned structures are documented by Clark (1992) in work carried out for the British Cement Association. However this report concluded that performance was generally good.

The UK road building boom of the 1970's and 1980's had resulted in many post-tensioned bridges being constructed although it is believed that the railways of Britain actually have an older average age of such bridges, many of which are being investigated.

As most of the early evidence of problems had been concentrated on externally post-tensioned structures, simply because their cables were visible for inspection, there was a ban introduced by the UK Department of Transport in 1977 on use of external post-tensioning in bridges. With the benefit of hindsight, one can challenge whether this was a sensible decision. There is little doubt that external post-tensioning has a significant advantage in that it is relatively easily inspectable, whereas internal post-tensioning is buried inside the body of the concrete and despite significant investment in recent years has proved extremely difficult to inspect.

Nevertheless fortunately development of external post-tensioning technology continued outside of the UK and now forms an important part of many significant concrete bridges. In the UK the Camel Viaduct in Cornwall [Fig 4], completed in 1993 was the first to utilise new design rules developed by the Highways Agency (1994) and included innovative features to allow replacement of the post-tensioning cables in the future (Hollinghurst 1995).



Fig 4 Camel Viaduct, Cornwall

In 1985 the sudden collapse of the Ynys-y-Gwas Bridge in Wales (Woodward et al, 1988) had sparked investigations which raised serious concerns over the underlying durability of internal post-tensioning. In that instance it was a single span precast segmental bridge with internal tendons which passed through thin concrete joints between the units in ducts formed with cardboard. Over the years de-icing salts had penetrated and corroded the prestressing steel at the joints resulting in collapse.

In 1986 the bridge over the Mandovi River in Goa, India collapsed after less than 20 years in service due to corrosion of the post-tensioning cables. Cracks were unattended for six years and corrosion of prestressed wires, which was noticed in 1983, was neglected until the bridge collapsed. It is reported by Arullappan (2010) that the available workforce were probably not able to provide the required standard of workmanship to construct the bridge.

In 1992 a further bridge collapse occurred in Belgium. The Malle Bridge over the river Schelde relied entirely on buried tie-down prestressing tendons for its stability which failed due to corrosion (ASBI 1998). The tie down cables were completely uninspectable and this is a prime example of a high-risk form of structure. Such details should be avoided.

It was not only bridges which experienced durability problems with prestressing. In Berlin the roof of the West Berlin Congress Hall collapsed in 1980. The thin shell roof had post-tensioning bars in ducts which had corroded partly due to poor quality of grouting (Buchhardt et al, 1984).

Doubts were shed on the adequacy of grouting materials and techniques and some of the construction details, and following the SCOSS concerns a ban on grouted duct post-tensioning was introduced in 1992 by the Department of Transport. There already existed the ban on externally post-tensioned bridges due to previous failures. It was generally accepted that this did not mean that all post-tensioned structures were unsafe, but the immediate response was to carry out a study of different types of post-tensioned structures and systems and broadly categorise them by risk assessment of the potential impact of deficient grouting and possible corrosion of the tendons. At the same time a programme of physical intrusive investigations was embarked upon to assist in understanding the degree of deficiencies and current extent of damage with a view to establishing a programme of repair options. Early studies published by Woodward (1981) had shown a mixed picture of different conditions in existing structures. This manifested itself in some cases as fully grouted but in others as complete lack of grouting, or just a thin coating of the steel. However generally little corrosion was found. Clark (1992) had also presented a generally satisfactory picture.

The Highways Agency in 1994 started a significant structured series of inspections of their bridges and this gradually provided evidence that there was a growing problem with some post-tensioned structures. Problems included corroded anchorages [Fig 5] and broken wires [Fig 6].

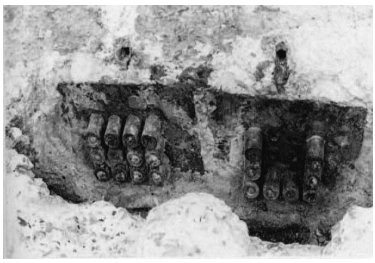


Fig 5 Corroded top slab anchors – some barrel grips completely corroded away



Fig 6 Significant wire and strand failures (Encasement concrete removed)

Lessons to Learn and Areas for Improvement

A joint industry working party was established in 1992 by the Concrete Society and the Concrete Bridge Development Group to plan and instigate a programme of studies with a view to making recommendations for improvements for future structures, with the intent to allow the ban to be lifted. This involved reviewing existing standards, carrying out development and testing of new materials, consultations with consultants, suppliers and contractors.

Publication of an interim Concrete Society report in 1995 presented a preview of proposals and was followed by the first edition of Technical Report No 47 (Concrete Society 1996) which resulted in the ban being lifted. It has been revised over the years and is now Technical Report No 72 (Concrete Society 2010). This has a comprehensive record of the work carried out and contains many recommendations. Most of the materials and grouting related issues have found their way into standards but the document contains important guidance for detailing of both bridges and building structures.

In the rest of the world there was a growing awareness of the problems in the UK disseminated through bodies such as the federation internationale du beton (fib) and gradually it became apparent that the problem was not just limited to the UK. In the USA important bridges in Florida revealed problems at Sunshine Skyway, Niles Channel and some other major bridges and other states. Swiss, German, Japanese and French experience was shared. Fib established a working group to prepare guidance on grouting published as Bulletin 20 (fib 2002) and another to give guidance on use of plastic ducts.

Grout

For many post-tensioned structures grout is used to protect the steel from corrosion. This has proved to be a fundamental weakness in many cases and something to which major attention has been paid over recent years to improve. Long used test methods were found to be inadequate to distinguish between good and bad grout formulations, which resulted in some tendons having voids in their protection or poor quality of grout leaving the steel susceptible to attack by penetrating chlorides. An understanding has been gained through research of the key qualities of grout needed to assist ability to fully grout the ducts and to provide long-lasting protection, with a particular focus on control of bleeding. Use of factory formulated pre-bagged grouts has now become almost the norm in the UK and a certification scheme is in operation by CARES (2010).

Ducts

The lack of any form of long term protection provided by the ducts was identified at an early stage as an area for possible improvement. In aggressive environments such as bridges steel ducts are not considered adequate where de-icing salts may penetrate to corrode them, especially at joints in concrete members. However proposing an entirely new product for what is essentially a public sector market was not easy, especially as there were no purpose made products on the market at the time. It started with a development of rather flimsy corrugated plastic conduits and drainage pipes, and various suppliers designed their own more robust systems. Test methods had to be trialled and evolved, but the requirements were essentially simple – what was needed was a sufficiently robust non-corrodible watertight duct to be able to be set to the required profile, withstand the duress of concreting and strand installation and stressing operations and be fully fillable with grout. It also had to be able to transmit cable force by bond to the concrete body.

The challenge of coordinating introduction of requirements for new non-corrodible plastic ducts was taken up by fib who published an authoritative guideline Bulletin 7 (fib 2000) which has become referenced internationally. Plastic ducts had been previously used in Switzerland with good experience but a new generation of ducts needed to be brought to market. This has now happened internationally [Fig 7, 8] and the guidance is currently being updated by fib with a view to standardisation in the future.



Fig 7 Typical Corrugated Plastic duct and coupler



Fig 8 Corrugated Plastic duct and coupler with vent

Quality and Skills

The UK also proposed an industry certification scheme which ensued from the process in direct response to requirements of the new recommendations, covering requirements for post-tensioning companies to satisfy certain levels of training and experience of personnel, as well as having documented QA procedures for carrying out post-tensioning work. This also raised a classic dilemma; how to require companies to demonstrate skills and experience with a new set of standards before the standards were in operation.

The solution was to introduce, with the support of the UK Highways Agency, an interim pre-certification scheme to allow close scrutiny, monitoring and recording, which would lead to full certification. This was supported by the UK Post-Tensioning Association and has become an almost universal requirement in the UK. The scheme is administered by CARES. Similar schemes are not universally used around the world although the Post Tensioning Institute in USA has a comprehensive skills and training qualification for firms and their operatives, and many global post-tensioning companies now have their own documented requirements in this area. In Europe similar requirements are now covered by EN13670 (BSI 2009) and the need for companies to hold a European Technical Approval for their system (EOTA 2002) and to comply with a CEN Workshop Agreement for installation (CEN 2003).

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Detailing

In addition to the practical aspects of improving the materials it was considered vitally important to learn from design details which had failed or proved to be troublesome and provide guidance for structural designers.

Experience of corrosion of tendons in or near their anchorages, informed the need for added protection to anchors placed in vulnerable locations under expansion joints or construction joints. This was developed to provide the concept of multi-layer protection whereby a second or third line of defence was provided to the tendons rather than simply relying on one single protection method. It was recommended to avoid anchorages in pockets in the top slab of bridges as these appeared particularly vulnerable to leakage. Additional double layers of deck waterproofing was recommended in certain locations and guidance given on placement and sealing of duct vents which had been shown from trials to help complete grouting at vulnerable high-points in duct profiles.

Full scale prototype trials were advocated on projects to verify the suitability of proposed grouting materials and techniques prior to full scale use [Fig 9].



Fig 9 Cores taken from full scale prototype test



Fig 10 Core showing satisfactory grouting

Experience on these was invaluable to reassure clients and designers that the ducts could be fully grouted [Fig 10] and also inform simple but crucial detailed changes to, for example, grout vent locations and grouting procedures.

The concept of multi-layer protection and suggested design details to avoid such problems are described in Concrete Society Report TR72 (2010) and in fib bulletin 33 [fib 2005).

Inspection

Owners of post-tensioned structures, especially bridges, have had a wake-up call and special inspection procedures and risk assessments for these structures were introduced by many authorities. Even so it is still the case that defects are in some cases a surprise when discovered such as recently on the Hammersmith Flyover in London which is currently the subject of a major emergency strengthening programme (New Civil Engineer 2012). This precast segmental bridge, post-tensioned both internally and externally was built in 1962 and has classic features of high risk with insitu concrete joints. It does not seem surprising to the author that the post-tensioning cables in ducts at the top slab over the piers could be suffering from chloride induced corrosion, the extent of which has only now been discovered 50 years on. It is fortunate that this was discovered before the structure suffered more serious damage, or even possible collapse, and that remedial measures are, at the time of writing, underway on site.

Methods of inspection are well documented (Concrete Society 1996) and intrusive investigation is generally recognised as the only reliable way to check condition of internal post-tensioning. In many cases this is difficult but initial examination of design details can and must be used to inform targeted investigations.

It is clear that in most cases a post-tensioned structure will be unlikely to show visual signs of distress when close to failure. This makes it all the more important to carry out full investigations where there may be vulnerable high risk details.

Owners of post-tensioned structures must be helped to understand that these structures are unique and very careful consideration needs to be given to their inspection.

Maintenance

Experience of repairs of post-tensioned structures is relatively limited. Several bridges have been so complex to evaluate the overall state with confidence that they have been demolished and replaced. However where there has been significant confidence to design strengthening or replacement, various techniques have been used, including supplementary tendons, as in the case of Hammersmith Flyover, passive FRP strengthening, or tendon replacement. The latter has to the author's knowledge only been carried out on externally post-tensioned structures to date.

The bridge at Palau in Indonesia collapsed suddenly in 1992 only a matter of weeks after completion of a strengthening programme and a protracted legal case resulted in details not being revealed until several years later. This was an extremely long span internally post-tensioned box girder cantilever bridge with a pin at midspan. It was initially hypothesised by some that the additional prestressing which was applied to

strengthen the structure and help correct significant sag of the main span may have contributed to the failure. However a later published paper (Burgoyne et al, 2008) postulates (with a cautionary note) that there were weaknesses in the original design and workmanship which could have been the main reason. Tendon corrosion was not apparently a contributor.

It is clear that many poor details on existing post-tensioned bridges should be corrected to ensure longevity and arrest possible deterioration. For many, however, without any obvious poor details, experience is that they should remain trouble-free. Post-tensioned structures when designed and detail properly certainly have the potential to be one of the most durable and long-lasting forms of construction.

New Standards

UK, European and International Developments

Publication of substantial new recommendations in the first edition of Concrete Society Report TR47 (1996) had led to the Highways Agency announcing a lifting of the ban on internal post-tensioned bridges. This was welcomed by industry but it was recognised that further development was necessary, assisted and informed by the International community.

In parallel CEN had instigated a review of existing standards for grouting, ENs 445, 446 and 447 (BSI 2007a, 2007b, 2007c) and publication of these was the product of a consensus view across Europe for improved grouting materials and injection techniques.

A second edition of TR47 followed in the UK in 2002 to reflect the European experiences. In the meantime international developments were being followed and fib had published bulletin 33 (fib 2005) which further disseminated state-of-the-art knowledge.

By the mid 2000's Engineers were gradually coming to a consensus view of the ways to improve the durability of grouting for post-tensioning, so an ISO Committee was convened in 2008 to develop International Standards for grouting informed by the ENs, new National standards in USA, Japan and many European countries. It is expected that ISO14824 Parts 1, 2 and 3 will be published in 2012 which will cover basic requirements, test methods and grouting procedures. This would mark a real milestone after more than 20 years since the work to improve grouting started in the UK. However there is still remaining much to learn. Not the least of this is to not necessarily leave unchallenged anything contained in a standard.

Experience from developing these standards has revealed that the current European requirement for grout to contain less than 0.01% of sulfide ion is apparently impossible to ascertain from any current ISO test (it is below measureable tolerances). It has emerged that there has been very little research into threshold limits for sulfide ions in contact with prestressing steel and even more concerning, that it appears that although this has been a required limit in the current EN for some 10 years suppliers appear not to have tested their compliance. This is a note of caution for standard developers to always seek firm evidence as back-up before including requirements proposed by one country.

Recent Experience and Future developments

Recent problems

In the USA it was revealed in 2011 that SIKA, a pre-bagged grout supplier, had been supplying grout which had chloride levels significantly higher than the USA specification limit of 0.08%. This apparently had been supplied undiscovered and untested for several years, with the supplier relying on the supplier of the bought cement to comply with its certification regarding chloride levels and no further testing being carried out.

It is reported (US FHWA 2012) that this grout has been used on over 30 bridge projects in the USA and may have serious implications which are currently under investigation.

This is potentially seriously damaging for the credibility of self-certification as well as for post-tensioning in general. We must learn from this that Audit testing MUST be a routine part of every supplier's procedure.

The 2011 findings of major corrosion to tendons in the 50 year old Hammersmith Flyover in London have luckily come in time to potentially save that vitally important structure. We must learn from this to be more questioning and reinforce to owners of structures the need to make deeper and more searching investigations where it appears fairly likely that our older structures have vulnerable details, even though it may be more costly.

Successes

We should also champion successes. Relatively few post-tensioned bridges have been constructed in the UK in the last 20 years but those which have will have benefited from much improved technology. Better still is the fact that many post-tensioned elements of our buildings are also now using much improved standards having learned from the experience gained in bridges. This is an important example. The latest edition of the National Structural Concrete Specification (Concrete Centre 2010) embodies many of the key features of good practice for grouting learnt by industry as well as embracing Eurocodes.

Techniques have developed considerably in recent years for investigation and monitoring of post-tensioned structures, and in particular the health of their prestressing cables. However there is certainly further to go and scope for further development. These include acoustic and ultrasound technology as well as use of sensors embodied within the cables at the time of construction. Time has yet to tell whether the latter will prove valuable for long term health monitoring.

New materials are always being developed. Currently there are higher strength steels, carbon based composites and new epoxy fillings for coated strands. Each of these will need to be proven but what we have learnt from the past should enable us to take care about their use.

Conclusions

- There is little doubt that much has been learned over the past 20 years and more will be learned in the future to enhance the durability and longevity of post-tensioned structures.
- SCOSS were absolutely correct to start raising concerns as early as 1979 about the sufficiency of grouting, which has been a key influencing factor in starting the drive for improvements.
- Poor detailing has been as much to blame as poor grouting.
- Although properly designed, well detailed and constructed post-tensioned structures have potentially long life if well maintained, we should and must not be complacent about post-tensioned structures – especially the older ones. Owners and Engineers should be pro-active to ensure these have full and proper investigations, especially where tendering requirements may act against thoroughness of approach.
- The importance of suitably frequent audit testing for materials must be emphasised, however comprehensive other requirements for compliance with standards may appear to be.

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Forensic Engineering: Informing the Future with Lessons from the Past
ISBN 978-0-7277-5822-4

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doi: 10.1680/feng.58224.457

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Learning from confidential reporting

Alastair Soane BSc PhD CEng FICE FIStructE, Director Structural-Safety, London, UK

History

CROSS, the Confidential Reporting on Structural Safety, was established by SCOSS, the Standing Committee on Structural Safety, in 2005. The objective was to improve structural safety by using confidential reports to highlight lessons that have been learnt, to generate feedback and to influence change. It aims to help protect the public from potential dangers and those in the industry who could be affected by failures and collapses. Principle sponsors and funders are the Institution of Structural Engineers, the Institution of Civil Engineers, and the Health & Safety Executive and support has been given by government departments, consulting firms and contractors. In 2011 CROSS and SCOSS were amalgamated as Structural-Safety.

The models for confidential reporting are the American Aviation Safety Reporting Scheme (ASRS) and the UK equivalent Confidential Human Factors Incident Reporting Programme (CHIRP). CROSS is only such scheme for the engineering industry but it is hoped that participation will be extended to other countries.

The process

Measuring success in any safety scheme is difficult as there are no parameters about incidents which did not take place, only about those which have occurred and there are fortunately very few major collapses in the UK. Many of the reports however are about precursors which, when discovered in time, may prevent more serious events from occurring.

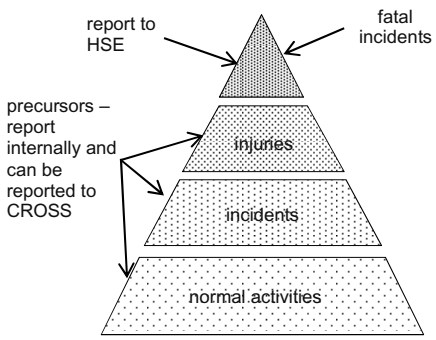


Figure 1 Pyramid of risk

Most safety activities are the result of actual or near miss events. ASRS uses a pyramid of risk to differentiate between normal activities, those which may be considered as near miss incidents, and fatal incidents. Figure 1 shows how this relates to structural safety where precursors (i.e. warnings of danger) can be reported to CROSS as well as being dealt with internally. Fatal

incidents of course will be dealt with by HSE in the UK or other authorities elsewhere.

Reacting to precursors is an important preventative measure and, by recognising this, an increase in safety awareness can be promoted. The cost of a significant event, particularly when there are serious injuries or fatalities, is huge in human terms, in financial terms, and to the reputation of firms and individuals. Failures involving death or serious injury may also result in prosecutions. In comparison the cost of learning lessons from previous concerns, including near misses is negligible. Most organisations have mandatory accident reporting systems but fewer have reporting systems for near misses, or indeed for near hits which may be a more effective term.

CROSS does not replace such systems but is complementary and provides a central reservoir of information that may be of use within the construction industry. CROSS and SCOSS (Standing Committee on Structural Safety) are combined under the banner Structural-Safety[i] which collects, analyses, and publishes information from both publically available and confidential reports. There is no formal mechanism for investigating causes or carrying out forensic analysis and the information provided by confidential reporters is taken as correct although it may be amplified during correspondence or discussion with the reporters.

Reports are usually sent by email through the Structural-Safety web site although there is a facility for submitting them by post. The postal system leaves no electronic trail and hence may be preferred by those whose want maximum security. Reporters must give their names so that follow-ups can be made but the first part of the process is for names to be removed and reports de-personalised. The next stage is to remove identifying features so that reports become anonymised and there are no identifiable references to projects, products, or firms.

The system of processing is such that only the project director sees the original reports and also de-personalises and de-identifies them. Other personnel in Structural-Safety see only the anonymised reports.

Categorisation and analysis

After the concern has been described further information is requested including:

- Professional or other affiliation
- Job title and location
- Type of organisation
- Area of Concern
    - Construction
    - age of Structure
    - Structure Type
    - Material

Reports are categorised according to:

- Type of structures
- Main materials used
- Building elements involved
- Nature of concern and stage of project
- Process at the time e.g. design, construction, normal use, or demolition

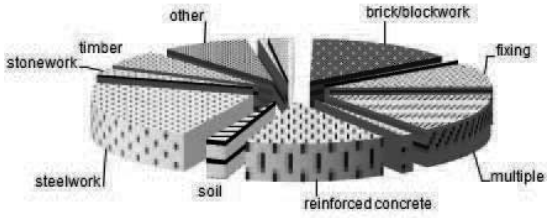


Figure 2 Principle materials involved

Figure 2 shows the proportions of the principal materials about which reports have been sent and similar information is collected for the other categories. Descriptions of what to report and how to report are given on the web site. Before publication the de-identified reports are reviewed by a panel of experts and comments are given with the intention of pointing out the lessons that can be learned so that similar concerns or events can be prevented. It is stated that whilst Structural-Safety has taken every care in compiling its publications, they do not constitute commercial or professional advice and this is because specific circumstances will differ from case to case. Reports and comments are stored on a data base on the site and it is hoped that this will prove to become a useful resource for practitioners.



When a trend is detected, usually when three or more similar reports have been received, then consideration is given to issuing an Alert from Structural-Safety. These are also issued when a trend is seen from published information on failures that have happened anywhere in the world. The value of Alerts is that they are circulated widely to engineers who can use the information to avoid similar events.

Figure 3 Cycle from reports to Alerts

Case studies

The cases where the cycle shown in Figure 3 has been completed, and which have been included in Structural-Safety publications, include:

- The selection and installation of top fixing anchors (usually for ceilings)
- Aging multi storey car parks
- Temporary stage structures
- Snow loading collapses
- Demolishing large panel structures
- Tall asymmetric buildings

The selection and installation of top fixing anchors

Anchors and top fixings are critical to the safety and security of suspended ceilings and associated equipment. Their importance is sometimes neglected when they are not seen to be structurally important. However there have been many failures in the UK and elsewhere of anchors which have resulted in the collapse of ceilings, including a number that have been reported to CROSS. Failures of heavy ceilings have occurred in some cinemas and other

buildings and in underpasses structures. Fortunately there have been no fatalities or injuries but had there been people underneath any of these at the time then the consequences could have been severe.



Figure 4 Acoustic ceiling collapse

The causes for these failures, and advice on how to avoid them in future, are given in the report '*Selection and installation of top fixings for suspended ceilings'*, 2012,[ii]. It stresses that the costs and impact of a collapse can be huge and affects the lives of those involved whether they are victims or those who are responsible for the failure. Using the correct approach and ensuring the proper selection and installation of anchors will lead to reduced risks. The report deals with the contractual and statuary responsibilities of those involved in the design and supply chain, and recommends the approach to be followed to ensure safe anchorages. Very important is the selection of suitable anchors which is done by identifying application parameters such as applied load, base materials, Environmental factors and relating these to the type of structure providing the support and the type of anchor. Anchor installation information is given as is preliminary and proof testing. The CROSS reports were also considered when drafting the British Standard in development on suspended ceilings, 2012[iii].

Aging multi storey car parks

Several reports were received concerning the condition and quality of aging multi-storey car parks (MSCPs) which are common in city centres in the UK and elsewhere, and usually here large numbers of people may congregate. The reports suggest that there are structures which have deteriorated and could be at risk of failure and these echo concerns that have been expressed over a long period. Indeed in the relevant year it was reported that several car parks had to be demolished on safety grounds. The condition of others is unknown. Figure 5 is an example of the degradation that has occurred on the surface of an MSCP slab as a result of weathering and wear. This can affect cover to top reinforcement leading to corrosion and reduced capacity.



Figure 5 Degradation of car park slab

Headline events such as the at Pipers Row car park collapse in the UK in 1997, a Montreal parking garage collapse in 2008, and others show that severe, and sometimes fatal, failures occur in such structures.

MSCPs are unique structures in many ways as they are:

- subject to relatively severe weather conditions due to exposed structure, carbonation of the concrete, reinforcement corrosion, and spalling
- Sensitive to waterproofing finish failure

- Sensitive to expansion joint sealant failure
- Subject to extremes of thermal loading akin to bridges but rarely detailed to the same standard as bridges
- Subject to localised very high chloride concentrations from de-icing salts.

The typical pattern of deterioration is such that failure is likely to be indicated by spalling but may be sudden and brittle with little or no warning. Given the history, and the likely consequences of failure, there may be a case for making it a legal requirement to have regular inspections of these structures. In the absence of such requirements CROSS said that those involved should ensure that their concerns are expressed in clear terms to their clients, with the consequences to individuals and organisations of failing to act being outlined. It may be that some owners and operators tend to ignore current legislation, not to mention their own liabilities, on the basis that a collapse of a MSCP is a rare occurrence. This is true, but they may be unaware of how close inadequately inspected and maintained car parks can come to collapse. The British Parking Association (BPA) in *Parking Practice Note 30, Liability for car park maintenance*, 2011[iv], therefore recommends that: "Owners and operators set aside funds from income streams to finance periodic structural inspections and essential maintenance of car park structures. Local Authorities practices differ and some carry out their responsibilities very thoroughly and have excellent MSCPs. In the aftermath of the Montreal collapse a coroner called for tighter inspection rules after finding that a car park that collapsed and killed a man was badly built and maintained. She says the structure, built around 1970, was in a sorry state and had surpassed its useful life.

The reports were published in CROSS Newsletter No. 26, 2012,[v] which was sent to local authorities and consultants, many of whom have involvement with MSCPs. The BPA have raised their concerns with government, and have assembled a body of evidence about structural conditions. Awareness has been raised as a result of publishing these concerns and is hoped that more attention will be paid to improve the levels of inspection and maintenance.

Temporary stage structures

In 2011 there were collapses of temporary stage structures in the USA and in Europe during entertainment events resulting in fatalities and numerous injuries. Reasons why the failures have taken place have generally not yet been published, although initially wind has been reported as a contributory cause in some cases. The temporary structures which have been involved in these collapses may be similar to those used in the UK. One of the most dramatic was the collapse of stage structure at the Illinois State fair in 2011.



Figure 6 Braced temporary stage

CROSS had been sent two reports of concerns about temporary stages so, coupled with information on publically reported failures; a SCOSS *Alert on Temporary stage structures*, 2012,[vi] was published. It was aimed at those who may commission or procure or licence temporary stages and other temporary structures, for example large outdoor TV screens, which offer potential risks to the public. These included owners of sites and venues, promoters, contractors and their designers, local authority licensing officers and building control officers, and insurers. Whilst the Alert was primarily concerned with temporary

stage structures the principles apply equally to all temporary demountable structures. A prime responsibility of the client is to make sure that competent persons are employed to design, independently check, erect, inspect, monitor on site and dismantle the structure. An example of a properly braced temporary stage is shown in Figure 6.

It was stressed that contractors should be able to demonstrate to the complete satisfaction of the client, or the client's engineers, that the proposed structure has been specifically designed to accommodate all vertical loads, including self-weight, lights, and sound equipment and snow (if applicable). It should also be capable of withstanding agreed wind loads applied to the structure and equipment and any roofing and side cladding, such as advertising banners, and, if relevant, vehicle impacts. The design should be for the structure as a whole taking account of overall robustness, strength and stability as well as the capability of the individual components. Elements such as trusses which are connected together should be able to withstand local member loads. To carry lateral loads there must be appropriate connections between vertical and horizontal elements and all loads must be taken safely to the ground through suitable foundations. Frames must be braced in each axis to avoid undue distortion and lack of stability.

Regardless of the competence of any engineers involved in the design process, systems sold to end users frequently rely on that end user making site specific judgments about the anchorage of temporary structures to the ground. Positions of ground anchors or kentledge shown on drawings may be changed for operational reasons. End users may not appreciate that by taking decisions on anchorages they are attracting design responsibilities.

Almost all temporary stage structures are designed for an upper limit wind speed and guidance is given in *Temporary demountable structures, Guidance on procurement, design and use, Third edition, 2007*[vii], on how to consider wind loads including advice on operational site management and for temporary structures there should be a wind management plan. Wind speeds at which the recent overseas collapses have occurred have not yet been published except in the case of the Indiana collapse where they were comparatively low, but there have been cases in the UK of wind induced failures of temporary structures where recorded wind speeds have been within foreseeable limits. The Alert concluded by listing key points to look for:

- Design documentation has to include the basis of design, loadings, calculations, drawings and specifications and a clear definition of the operating limits of the structure including the effects of all loads.
- Evidence of competence of the designer by way of qualification and experience and insurances carried
- Evidence of independent design check by a chartered structural engineer
- Evidence of independent erection check by a competent person
- Completion certificate from the event organiser to confirm all of the above
- Monitoring of wind speed and a plan of action to be in place if there are restrictions on safe wind loads.

The Alert was widely circulated throughout the UK and internationally and prompted discussion amongst suppliers, contractors and others and raised awareness amongst local authorities. A detailed forensic analysis of the Indiana collapse was published by Thompson Thomasetti *Indiana State Fair Commission, August 13 2011 Collapse Incident Investigative*

Report in April 2012[viii] and the conclusions had many similarities to aspects of the Alert including:

- Inadequate lateral load resisting system
- Failure occurred at low wind speed
- Gravity ballast inadequate
- Inadequate structural analysis
- No third party review
- Gaps in code requirements
- No review by sound company to check capacity
- No record drawings or engineering data

Snow loading collapses



Figure 7 Warehouse roof collapse

Severe weather has been the trigger for many collapses, and snowfalls in the winters of 2009/2010 and in 2010/2011 caused over three thousand agricultural and associated buildings in Scotland to partially fail or to collapse. As previously reported, Soane, 2011[ix], the likely causes included the build-up or drifting of snow on roofs, the age of the buildings, weaknesses in original design or construction, and lack of maintenance. Prolonged periods of snow deposition and very low temperatures were experienced which contributed to snow compaction and increased snow densities. Throughout the north of Scotland depths of 1,100 – 1,200 mm on roofs were reported with snow overlying ice.

Most collapses were of agricultural and associated buildings with steel or timber portal frames as described in a Scottish Government report in 2011[x] but there have been other examples including the roofs of a series substantial warehouses – see Figure 7. There were also reports of rainwater channels freezing in the prolonged sub-zero temperatures and collapsing under the weight of ice. Substantial, and potentially dangerous, dumps of snow were experienced from the sloping roofs of domestic properties. CROSS received a number of these reports directly and their technical content was shared with the Scottish Authorities. An alert was published by SCOSS on *Snow loads on agricultural and other buildings,* 2011[xi] which recommended that designers take account of the recent collapses when considering structures where significant snowfalls may occur.



Figure 8 Progressive collapse

Demolishing Large Panel Structures

In 2009 a Local Authority group was engaged in a scheme for the demolition of two thirteen storey large panel structure (LPS) tower blocks which had been manufactured by a company, now out of business, in the 1960s. A report was sent to CROSS and subsequently published together with comments in CROSS Newsletter No 18, 2010[xii]. A high reach demolition rig with a concrete nibbling attachment was used to reduce a corner of the first building to ground level. The same method was used for five storeys of the opposite corner, but then the wall panels and floor slabs forming the bottom eight

floors collapsed progressively and unexpectedly. After precautionary measures were taken, work recommenced to remove the damaged portion of the structure which was now considered to be unsafe. However this also suffered progressive collapse after the removal of panels from the top. The method of demolition was then reviewed and a new system was adopted of reducing four storeys at a time across the rest of the block working around the building in a circular pattern. This was successful.

An analysis was then made of the second block. However, evidence gathered in the early stages led to such concern that the investigation was abandoned on safety grounds. Client and contractors then worked to determine a methodology and control measures for the demolition of the second block. After taking down three or four storeys in one corner there was an uncontrolled collapse which affected the complete corner bay over its remaining height. Of considerable concern was the very clean shear failure across the slab to cross wall panel junction indicating a significant lack of connection or mechanical tying between floor panels and the cross walls forming the structural cells.

CROSS considered that there are two main issues: firstly the demonstration of the lack of robustness in these particular structures when exposed to demolition loading, and secondly the nature of the demolition process adopted. The forces involved during demolition should be taken into account by designers, and CROSS has had previous reports of failure during demolition. There is always the legal need for a good management system to avoid excessive collapse which could introduce unnecessary risks to operatives, to people who may be nearby, and to surrounding buildings. Learning from this example, the system of work for demolishing similar LPS structures should take into account that sudden progressive collapse may occur. The pictures are very reminiscent of the Ronan Point collapse in 1968 which was the subject of a Public Enquiry in 1968 and Guidelines on strengthening similar LPS tower blocks were later published. The lack of adequate ties between walls and floors and between adjacent floor panels, shown in the photograph in Figure 8, means that the buildings would not comply with current Building Regulation A3, but at the time of construction the need for horizontal and vertical tying through mechanical anchorages, or other suitable measures, was not appreciated.

Tall asymmetric buildings

One reporter touched on the topic of analysis and design of the structural systems for tall buildings which are asymmetric and may be irregular in plan and published in CROSS Newsletter No 22, 2011[xiii]. These might have a concrete core and an external steel frame with floors spanning between core and frame. Tall buildings which are complex in plan raise interesting issues concerning the methods of analysis that should be used, and there is conjecture that a first order linear analysis may not represent the structure adequately. It has been argued that non-linear geometry effects should be included so as to provide a better representation of the behaviour of the structures.

CROSS commented on the problems of validation of analysis models. The questions to be expressed and answered are:

- Is the model satisfactory in its representation of structural behaviour?
- Are the software and the way it is used appropriate and suitable?
- Are the results correct?

Analysis software must be used within the limitations of its applicability. It is all too easy to believe that because a structure has been computer modeled the output is therefore accurate. As buildings become taller and more complicated, issues customarily ignored in smaller structures start to become critical. In tall buildings, for example, differential elastic shortening of vertical supports can cause significant redistribution of horizontal member moments. Moreover non-linear effects such as those due to elastic sidesway from non-uniform vertical loading start to become important as do P-Δ effects. The design of connections and junctions at the ends of members is generally left to specialists, but the size and stiffness of the nodes will affect the analysis to a considerable degree. This is particularly the case with complex inclined buildings when many members intersect at a point. It is vital to be able to check whether the software has a built in ability to consider both global and member buckling resistances.

Asymmetry in a structure may exacerbate such effects under both vertical and horizontal loading but whether these could lead to potential instability issues would depend upon the example being studied. In designing a non-standard structure, the process should start by ensuring that all possible aspects of behaviour can be represented by the model until there is confidence that they can be ignored. Some of the validation issues can be resolved by carrying out sensitivity analysis e.g. making runs with and without non-linear geometry and comparing the results. Aspects not to be ignored are the horizontal and vertical elastic changes that occur during construction (perhaps exacerbated by asymmetry), which will have an effect on the stress distribution and stability in the completed structure. These are dependent on which separate parts are rigidly linked. Thus the modelled stress distribution in the final structure is influenced by the sequence and manner of phased construction.

There is now an international consensus that 'robustness' ought to be a consideration in all structural designs yet there is no agreement about what ought to be done for tall, novel structures (Class 3 structures in UK terminology). The Eurocode rules for lateral load envisage some proportion of this load is notional (as in UK practice) but some lateral load will arise due to out of plumb in the erection of the columns. In tall asymmetric structures this ought to be a consideration, but the effects may not be apparent from the model representation of the structure. The Institution of Structural Engineers report *Practical guide to structural robustness and disproportionate collapse in buildings*, 2011[xiv]", whilst not dealing with Class 3 structures, makes the following very important point. "A key presumption is that for any one building, there should be one engineer in overall charge of both stability and robustness and not least when multiple structural disciplines are involved as in hybrid structures." In view of the complexities involved the question can also be raised as to whether buildings like these should have a mandatory check and, separately, a review, as part of Regulatory Control procedures. Should there, speculated CROSS, be a process for checking on the competency of organisations and individuals to be responsible for such designs in addition to general safety legislation in the UK?

The validation of software, and its proper use, is a matter that needs to be addressed both in practice and in education and the publication of this report and others of a similar nature help to support the argument.

Conclusions

These case studies demonstrate the range of topics that have concerned reporters to CROSS. They illustrate how confidential reports can be the starting point for creating evidence that can be used to inform and influence engineers, and possibly regulators and the writers of

standards. Without the programme much of the evidence would not have been linked, trends would not have been observed, and the Alerts could not have been published. Data has been passed to code committees, government departments, professional institutions or others who have influence of best practice. Alerts have been widely published and circulated.

The Structural-Safety web site is viewed from around the world with 60% of the visits from the UK and the balance from many other countries. The intention is to develop the scheme so that the data base can be international and information can be shared between engineers whatever their location. Discussions are taking place with organisations in several countries to determine the best way to achieve this ambition.

Acknowledgements

The support and contribution of the members of the CROSS expert panel and the SCOSS committee is gratefully acknowledged. Without the confidential reports the case studies would not have been possible so the reports are much appreciated and valued.

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Forensic Engineering: Informing the Future with Lessons from the Past
ISBN 978-0-7277-5822-4

ICE Publishing: All rights reserved
doi: 10.1680/feng.58224.468

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Rebuilding Woolwich Arsenal Clock Tower

Jeremy Foster (Ramboll, London, UK.)

There are few things more likely to create a hush around a structural engineering office than the words "there has been a collapse on site."

On the 23rd of October 2008 The Warren Development at Woolwich Arsenal was 90% complete. A £77 million redevelopment project involving the adaption and preservation of historic military ranges and two, new build, seven storey RC residential blocks to modern design. Then the call came through from the contractor – the north range clock tower had collapsed.

Even without any witness accounts, arriving on site it was hard to avoid the obvious conclusions as to the root cause.

The specifics behind the collapse are not the subject of this paper. Rather to demonstrate that the success of heritage forensic engineering is intrinsically linked to adhering to the conservation principles as laid down in the *Australia ICOMOS Burra Charter, 1999. The Venice Charter and English Heritage Conservation Principles.*

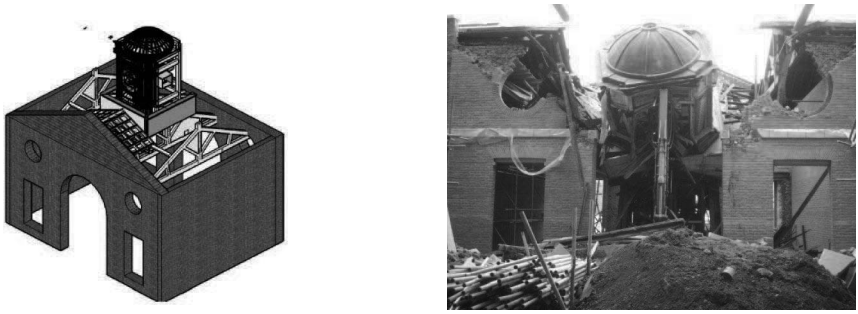
However, in order to understand the full scope of issues that faced Conservators and Structural Engineers the following series of events were relayed.



Figure 1 – trusses straddle digger

The grade II listed 19th Century clock tower consisted of a 3m high louvered timber bell tower with lead cupola sitting on twin timber king post trusses. These trusses also supported root purlins and sat on the load bearing masonry walls. The rear wall having recently been rebuilt with the arch formed via a concrete frame. A temporary tie rod was still in place when an unauthorised large earth mover attempted to traverse from one side of the range to the other.

The arm of the digger collided with the tie rod, pushing the arch from its bearings. With the arch removed the truss bearings dropped and the rest of the clock tower along with them.


Figure 2 –3D sketch before and picture after collapse

Immediate action

Safety of course was the immediate concern and remained the top priority; thankfully no-one had been injured in the collapse. Even at a distance it was apparent that the collapse had the potential for further failure and significant instability remained. The area around the collapse was cordoned off whilst engineers assessed what had happened as best they could from neighbouring buildings in safe proximity.

In the first instance the Northern bearings to the twin trusses remained in place acting as the pivot point as the southern bearings dropped away with the arch.

The trusses straddled the digger with main carcass of the clock tower striking the cab and the extended arm. The impact shattered three of the louvered sides of the bell tower, but crucially one of the sides remained largely intact. The lead cupola was also seemingly unscathed, albeit that the connections to the surrounding framework were now ruined under the impact of its considerable weight.



Figure 3 – hanging trusses

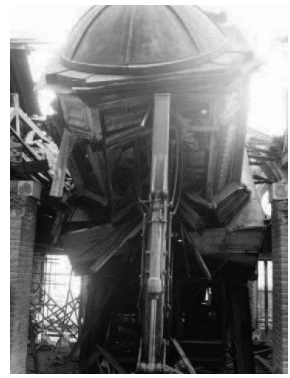


Figure 4 – collapse on digger arm

The twin trusses not only supported the clock tower but also the purlins spanning to the adjacent trusses. Had the purlins being configured as 'butt purlins' they would have allowed the two adjacent trusses to collapse. As it was the 'through purlins' now cantilevered from the intact sections of the roof providing the only means of support to the adjacent trusses which now seemed very precarious.



Figure 5 – pediment vulnerable

The pedimented front wall was similarly in a questionable state of stability. As the twin trusses had dropped, the rotation of the bearings had levered away considerable sections of the masonry surrounding the bearings. The pediment itself had laterally displaced by some 25mm and was also out of plumb by 50mm.

The weight of the collapsed clock tower was now partially supported on the digger and part on the twin trusses, which now leaned against the inside face of the front wall. The southern rear wall was pretty much demolished down to the stone band course and the bulls eye windows were also corbelling in a precarious state.

Immediate action items were drawn up and issued, principally focusing on safety concerns. No access whatsoever was allowed to the immediate area both internally and surrounding the building. Whilst time was of the essence it was clear that the rigours of fact finding, assessment, testing, analysis and interrogation were going to be crucial in establishing effective remedial proposals. These proposals had to be cost effective, time efficient, appropriate to the scheme and, not least, heritage sensitive.

Information gathering / assessment

As the engineers involved in the original proposals for adaption Ramboll had already accumulated significant amount of information on the Clocktower to assess and preserve the heritage. *{Principle 3: Understanding the significance of places is vital}[3]* Desk top research by engineers and the architect had been conducted and backed up by extensive site investigation. Through the first phase of the work this understanding of the structural form was employed in assessing the state of the collapse.

Records indicated that the evolution of the structural form centred around its semi-industrial development as the Royal Arsenal from 1805. Solid load-bearing masonry with lime mortar and heavy timber king post trusses had been recorded as surviving the ravages of time reasonably well. Some remediation had been required where water ingress had caused damage to truss bearings in valley gutters. Restricted access and the sheer bulk of these timbers resulted in whole scale intrusive remediation being unwarranted. *{6.1 the cultural significance of the place and other issues affecting its future are best understood by a sequence of collecting and analysing information before making decisions. understanding cultural significance comes first then development of policy and finally management of the place in accordance with the policy.}[1]*

At the time of the collapse the clock tower area of the site was in the process of being adapted to become the principle entrance to the development when approaching from the river. Several heritage features had been reclaimed from other areas of the site for reuse in this entrance, including cast iron columns and riveted steel beams. Significantly these riveted steel beams were to be employed structurally and had been established on their bearings prior to collapse and were still in place.



Figure 6 – view from man-basket

Whilst the pre-collapse structural form could have been described as ‘robust’ it was clearly now in a perilous state.

As soon as it was viable suspended access was arranged via man-basket from a mobile crane to further assess the damage and stability of the precarious elements as close as safely possible. It was evident that small-scale instability of tiles and broken timbers posed a threat to any operatives approaching the building on foot.

It was clear in the immediate days after the collapse that some significant elements of the wreckage were still 'on the move'. As can be seen here from Figure 7, taken two days apart, the lead cupola is still slipping down the arm of the digger. Not only a hazard to anybody approaching the wreckage, this significant heritage feature appeared to have weathered the collapse largely intact and was in danger of falling further and becoming damaged beyond recovery.



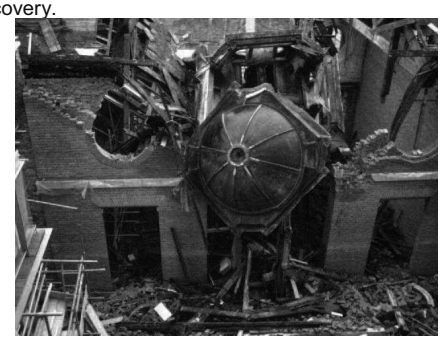


Figure 7 – Cupola slippage within the first 4 days



Figure 8 – Front wall laterally loaded

It was imperative that primary support was installed to prevent total loss of significant heritage elements. However, before this could happen the small unstable ancillary elements of tiles, timber and battens were either established as stable or made to fall by testing with a bar from operatives suspended in the man basket above the debris.

In the meantime temporary shoring proposals to the front wall were established with the demolition contractor Keltbray. The principle being that two lightweight scaffold towers would be constructed remotely and manoeuvred into position prior to the kentledge being installed.

This would offer protection to any operatives should the facade fail during the installation work. Once the primary structure was stabilised and immediate danger of collapse had been removed the site was assessed to install the props and remove the principal structural elements from amongst the debris.

Interrogation of information

At the time the Woolwich Arsenal contained several disused engineering workshops which were already employed as storage for some of the salvaged historic beams and columns reclaimed from elsewhere on the site.

Recording the disposition of the elements amongst the wreckage, careful deconstruction of the collapsed structure now took place. The workshops now served as layout and storage space for the retrieved sections of the clocktower.

Piece by piece individual elements were mapped and assessed. Working with the record information and photographs from the original survey individual elements were identified and painstakingly recorded by the architect, AQP. This enabled the construction of a 3-dimensional computer model of the superstructure of the clocktower. Timber experts Hutton and Rostron were appointed to survey the salvaged timber for damage and extent of rot.

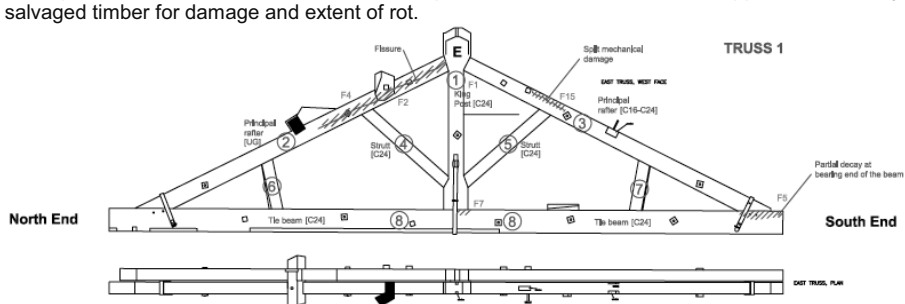


Figure 9 – Truss damage schematic

The entire design team, including the contractor, gathered to review the information in consultation with English Heritage. *{122- intervention demands: a skilled team, with the resources to implement a project design based on explicit research objectives}[3]* The significance of the clock tower within the development and surrounding community had already been established. Fortunately the internal workings of the clock had mercifully been transported away for restoration at the time of the collapse. Further demolition and rebuilding, whilst potentially the most expedient solution, was universally rejected in favour of restoration. *{2.1 places of cultural significance should be conserved.}[1]*. The strategy devised was to maximise the salvage of recoverable material. The question now was what was realistically recoverable?

In some small way the collapse had afforded the team an opportunity that would not have otherwise been available. The complete deconstruction of the clock tower timber carcassing allowed access to areas of the super-structure previously not viable.



Figure 10 – Tie rot

Lack of maintenance and some poor weathering details were the likely culprits in much of the rotten bearings and walls plates. Whilst these had partly been addressed in the previous small-scale remediation, the team were now in a position to fully assess these otherwise hidden issues.

Other than the bearings, the primary trusses themselves were largely free from rot; another exception being where water had gathered at the junction of the central hanger to the principle tie member.

It was apparent that the primary structural twin king post trusses had suffered under the impact loading of the collapse. The principle rafters had split in at least one place and wrought iron strapping had broken. The internal integrity of the connections was now questionable. There was no way to inspect these joints internally without destroying the integrity of the connection that was trying to be proved. Were this a new build there would be strong financial arguments for the existing timber to be abandoned and a new timber roof designed as a replacement.

However as with all historic structures it is necessary to let the building dictate its own constraints and abilities. The bespoke nature of the original clock tower, from a heritage perspective and from dimensional distortion, merited further investigation of the capacity of the trusses. The existing timbers were seasoned and had found their level with the surrounding structure before the collapse. Any new timber structure would struggle to match the unmolested strength and the dimensional constraints. The conservation argument was reinforced with a sound financial argument for re-proving the trusses.

Exploiting every available technique

In order to fully assess the potential for reuse it first need to be understood how these trusses were loaded and would be again. Using the 3D computer model, formulated from the historic information and detailed mapping of the salvaged timber, we were able to accurately assess the load paths through the intended reused and replacement clocktower super-structure elements. *{4.1 Conservation should make use of all the knowledge, skills and disciplines which can contribute to the study and care of the place.}[1]*

During the successful recovery of the cupola a relatively accurate assessment of its weight was recorded at 30kN. This was provided via the load gauge on the mobile crane which arrested the slipping cupola. This combined with the previous survey information and measurements of elements laid out in the workshops, allowed an accurate load assessment of what would be applied to the trusses under dead loading conditions. Standard live load theoretical analysis completed the model.

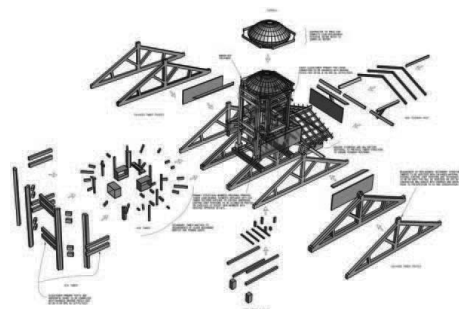


Figure 11 – 3D salvage & replacement

As anyone dealing with Heritage elements knows they often don't bear analytical scrutiny. However but the model was primarily formulated to provide member forces for the design of the repairs, the rest of the undamaged structure being seen to have proved itself over time. Also the trusses supported an array of transfer beams and posts that formed the carcass of a clock tower. The arrangement of

spreaders and props didn't necessarily coordinate with the internal members of the truss and most of the primaries were also acting in flexure.

Not only did the model give an accurate assessment of the design parameters for the repair sections, crucially defections were allowed to be predicted under different loading conditions. This was particularly important in the next phase full-scale load testing.

Proof of the pudding

However another factor loomed on the horizon. The main Warren Development was largely complete at the time of the collapse, one of the last elements being the testing of the drainage serving hundreds of dwellings. Unfortunately the main connection to the adopted sewer via a 325mm diameter pipe was intended to pass directly between the arches of the clock tower. All the infrastructure had been in place prior to the collapse other than the final drain run.



Figure 12 –Temporary works and kentledge

Even with all the trusses, carcassing and remaining clocktower elements in storage and the digger removed, a large amount of kentledge supported the facade retention and sat directly on the route of the trench.

With dozens of families expected to move in by March 2009 having signed contracts, this element of work was now critical. Various heading options were considered along with phased removal of the kentledge. However it was finally decided that Ramboll's option of supporting the front wall via an internal 'flying' set of temporary steelwork was the most appropriate to free up the site in the safest and fastest manner.

Prior to the collapse two heavy riveted steel beams had been reclaimed from another part of the site for reuse as structural elements. These remained in place and an assessment of the structure demonstrated that the damage was almost entirely above the line of these beams which were now significantly assisting the stability of the front wall.

The design consisted of a series of 'push/pull' raking stays supporting the walers sandwiching the facade. These raking stays terminated at a line of beams supported off the heavy riveted steel beams. Lateral forces were resolved through bracing fixed back to undisturbed shear walls flanking the clock tower.

At the highest points of the pediment the external waling restraints were fixed via 600mm long 8mm diameter threaded bars. These were carefully positioned by drilling externally through the mortar bed joint so as to avoid disruption of the historic brickwork.

{ 11c.The proposals are designed to avoid or minimise harm, if actions necessary to sustain particular heritage values tend to conflict.}[3]



Figure 13 – push / pull raking stays

Towards the lower levels ties passed through the bull's-eye windows and arched entrance to avoid even this low-level intrusion. By using a series of CHS struts between the walers sufficient distance was maintained from the facade to avoid damage of fenestration details; the final restraint interface being via folded timber wedges.





Figure 14 – waling arrangement around bulls eye windows

The internal position of the raking stays was carefully coordinated to ensure that the installation of the permanent works could be executed without adjustment of the temporary restraint.



Figure 15 –Internal temporary works



{2.4 places of cultural significance should be safeguarded and not put at risk or left in a vulnerable state.}[1]

Figure 16 – Lyndapter connection

To avoid drilling the exposed flanges of the reclaimed beams large Lyndapter connectors were employed at the junction with the bracing beams.

As well as releasing the kentledge and allowing freedom of movement around the clock tower ground floor this arrangement of 'flying' temporary works most crucially allowed the installation of the drainage connection via traditional trenching methods whilst ensuring the stability of the original fabric.

Running in parallel with this operation the principle truss repairs had now taken place. Rotten timber had been cut out and, where substantial integrity had been lost, additional steel plating with replacement infill timber had been introduced. This effectively replaced the action of the original unimpaired timber member. *{118. While sufficient work should be undertaken to achieve a lasting repair, the extent of the repair should normally be limited to what is reasonably necessary to make failing elements sound and capable of continuing to fulfil their intended functions.}[3]*



Figure 17 – Tie strap

The splint repairs at the breaks in the primary rafters were carried out with steel plates and single sided shear plate connectors. Whilst the integrity of the original handmade bolts was unknown they were given the benefit of the doubt as being structurally sound, as well as contributing significantly to the character of the trusses. Consequently templates were crafted prior to fabrication of the steel plates to ensure that the handmade bolts remained undisturbed. *{3.1 conservation is based on a respect for the existing fabric, use, associations and meanings. It requires a cautious approach of changing as much as necessary but as little as possible.}[1]*



Figure 18 – historic fixings preserved

With all of the repairs completed the timber trusses were placed in temporary testing rigs. The primary loads supported by the twin timber trusses had been assessed as being transferred via the sizeable (300mm x 300mm) timber bulks supported from the main tie members.

In consultation with the testing engineers (Train & Kemp) it was decided that the load that would be applied at the purlin positions would be static and the incremental load would be applied where the bulk timbers met the principle trust tie.

{120 The use of original materials and techniques for repair can sometimes destroy more of the original fabric, and any decoration it carries, than the introduction of reinforcing or superficially protective modern materials. These may offer the optimum conservation solution if they allow more significant original fabric to be retained.}[3]

Strain gauges were positioned beneath the tie members as the load was applied incrementally and the figures compared with the theoretical finite element model. If at any time the gauges exceeded 20% of the theoretical values or the reading failed to come to rest, the load would be removed. The truss was loaded at 5kN intervals to 110% of its theoretical maximum loading and passed all of its criteria. The predicted deflections of 5mm under full dead load were measured as 3mm in the test rig.



Figure 19 – full load test

{ARTICLE 10. Where traditional techniques prove inadequate, the consolidation of a monument can be achieved by the use of any modem technique for conservation and construction, the efficacy of which has been shown by scientific data and proved by experience.}[2]

Concurrently with the full-scale load test of the trusses conservation and materials experts, Carrig UK, were employed to assess the suitability for reuse of the existing lead cupola. With careful repairs this was deemed worthy of reinstatement.

With the principle heritage members having proved themselves fit for reuse there now remained the task of reestablishment into the rebuilt and re-stabilised load-bearing masonry of the clocktower.

The front face of the pediment superficially appeared unscathed from the collapse. Certainly the damage seemed minor in comparison with the substantial amounts of masonry lost around the truss bearings internally and the bulging brickwork beneath the coping stones. However, the pediment was now some 50mm out of plumb leaning inwards, there was 2mm to 5mm horizontal and diagonal cracking where the pediment had translated 25mm inwards. The bulging brickwork had been removed

and re-used in the reinstatement in matched lime mortar. The repairs around the truss bearings in the front wall were adjusted from the original removing detailing weakness and allowing for a 25mm air gap around the sides of the timber bearing. *{119 Once failure occurs, stabilising the structure depends on addressing the underlying causes of the problem, not perpetuating inherent faults.} [3]* Each bearing had four course engineering brickwork padstones. The design of the rear wall reinforced concrete arch remained the same as was previously constructed. The remainder of the reinstatement was completed in reclaimed brickwork forming pockets to accept the truss bearings.



Figure 20 - bearing reinstatement

As the 2½ tonne trusses were being lifted into place the careful planning of the temporary works proved its worth as the trusses swung into position unhindered. The line and levelling operation would have been much more complex had new timber trusses been fabricated. The original structure that remained provided fixed positions in relation to members that had deflected through the course of time. 'Straight and true' they may have been at one time following the original construction, but the rebuild was now marrying in members that had been reused, deflected, subject to rot, catastrophic failure, repairs and now reinstatement with additional members.

The success of this operation was dependent on flexibility. Not only in the position of the members but in the ability of the detailing to be crafted to suit the site constraints. In this timber and traditional detailing proved itself the most malleable. The high strength steel plating repairs had been detailed with sufficient tolerance to allow the original timber to timber connections at the seating of the through purlins against the principle rafter to be crafted traditionally. *{4.2 Traditional techniques and materials are preferred for the conservation of significant fabric. In some circumstances modern techniques and materials which offer a substantial conservation benefits may be appropriate.}[1]*
The original pediment was not connected to the timber framework other than at the truss bearing positions. During the collapse the pediment had moved out of plumb and fractured. Whilst some of the integrity of the pediment was restored with stitching of the principle cracks, it still remained significantly out of plumb.

Investigation of the walls makeup and subsequent calculations suggested that it was prudent to employ a series of strap restraints at noggin positions between the pitched rafters at the rear of the pediment. The visible impact of these was minimised by installing them in advance of the timber rafters.

The approach taken with the timber was to reinstate as much as salvageable and, where this could not be achieved, replace with matched timber where required. The salvaged complete face of the bell tower proved an essential template in crafting the three other sides that had been destroyed. In part the successful marriage between new and old was in being honest. Not trying to disguise the new elements but to celebrate the traditional detailing.



Figure 21 - bearing reinstatement

Similarly no attempt was made to try to disguise any of the steel repairs as being contemporary with the original. Modern bolts and strapping were finished with the same black paint as the rest of the metal work but remained distinct. The reclaimed steel beams were similarly finished and augmented with reclaimed columns to create a colonnade entrance beneath the clocktower.

{ARTICLE 12. Replacements of missing parts must integrate harmoniously with the whole, but at the same time must be distinguishable from the original so that restoration does not falsify the artistic or historic evidence.}[2]

The reclaimed columns, themselves easily capable of supporting the floor and wall loads, were in fact made non-structural by a hidden movement joint. The reason for this was the ground conditions were so poor, consisting of variable made ground encountered to a depth beyond 2m. The shallow pad foundations had such low bearing capacity (40 kN/m^2 from plate tests) that any load attraction from the floor had the potential of causing small settlements which in themselves would have little structural impact but still cause disruption to the stone plinth and so deemed unacceptable. This allowed reinstatement of the columns that expressed the vernacular of the arsenal to be reinstated in a position which both served to reinforce the importance of the clock tower and salvage these heritage features in a sympathetic context. *{9.3 if any building, work or other component is moved, it should be moved to an appropriate location and given an appropriate use. Such action should not be to the detriment of any place of cultural significance.}[1]*

Now all that remained was the reintroduction of the clock itself. Mercifully at the time of the collapse the mechanism of the clocktower and clock faces had been removed for repair and refurbishment. Much of the original plinth on which the clock mechanism sat was able to be reclaimed and reinstated.



Figure 22 - clock reinstated

All told when the repairs were finally complete more than 85% of the original timber was reused and cost less than £350,000. Such was the success of the detailing between the historic and new elements that the superstructure in this relatively small aspect of the Warren Development is expressed and celebrated as one of the jewels in this project. *{14c. The proposals aspire to a quality of design and execution which*

It is the efforts made in the application of heritage best practice that should be acknowledged here. Conservation principles are established to allow heritage practitioners access to the most appropriate measures to ensure survival of the cultural value of a place. Using the opportunity the collapse presented to apply forensic engineering to the potential weak points in the heritage structure has helped to ensure its survival. The dedication by the contractors and design team to maximise the conservation, using traditional techniques and materials alongside modern methodology and appropriate interventions in no small measure contributed to this being shortlisted for the Institution of Civil Engineers 2011 London Heritage Award.



Figure 23 – Finished front



Figure 24 – finished rear

References:
[1] The Burra Charter: The Australia ICOMOS Charter for Places of Cultural Significance 1999
[2] The Venice Charter / La Charte de Venise 1964 - 1994
[3] English Heritage: Conservation Principles Policies and Guidance 2008

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