Advances in Cable-Supported Bridges

Edited by Khaled M. Mahmoud

ADVANCES IN CABLE-SUPPORTED BRIDGES

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Advances in Cable-Supported Bridges

Edited by

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Preface

Cable-supported bridges are notable for their visual elegance, aesthetic appeal and ability to link long spans. The extent of issues of concern associated with these structures is commensurate with their size and vast scale. Significant advances in the technology of assessment, design, construction and maintenance of cable-supported bridges have been achieved in the past few years. This was due, in part, to increasing awareness, collaboration and information exchange. In the framework of sharing knowledge between engineers from different countries, the 5th International Cable-Supported Bridge Operators' Conference was held in New York City on August 28–29, 2006.

The Bridge Engineering Association organized the 5th International Cable-Supported Bridge Operators' Conference with a distinguished group of renowned bridge engineers from all over the world. Throughout the conference, bridge engineers, representing owners, designers and contractors presented state-of-the-art material, assuring an authoritative account on the developments in the field of cable-supported bridges. This book contains a selected number of papers that were presented at the conference.

The proceedings lead off with a paper by Carter and Hussain on "Stonecutters Bridge – designing for operation." The bridge, which is located at the entrance to one of the world's busiest container port at Kwai Chung, Hong Kong, will be one of the longest cable-stayed bridges in the world. The authors describe the measures taken during the design to provide a durable structure with all the necessary facilities for operation. Stainless steel was used in the towers, both in fabrications and in reinforcement. A comprehensive set of facilities for inspection and maintenance of the structure was included in the design and the bridge is fully instrumented with a real time Wind and Structural Health Monitoring System (WASHMS). In "Second Bridge over the Orinoco – design, construction and operation", Humpf presents the unique design features, special inspection and maintenance program compared to Venezuela's typical operation of infrastructure. The design features of the bridge include combined rail-roadway traffic, two back to back cable-stayed bridges, independent ship impact protection, and a delta pier as fixed point. Self anchored suspension bridges present a challenge in their erection scheme. Currently, the methods considered for this structure type are to either build the bridge on falsework, erect the main cables and then self anchor them, or to build the bridge as a cable-stayed bridge, remove the stays once the main cables are in place, and self anchor it. "A construction solution for self anchored suspension bridges" by Lawrie and Huang provides a third option which limits the necessity to build two complete bridges prior to erecting and self anchoring the suspension cables. In "Mitigation and monitoring of cable vibrations for cable stayed bridges", Barrett discusses a case study in detail in which dampers were installed on a bridge already in service. Beginning with documentation of the cable vibration, the chosen solution is discussed and monitoring techniques for verifying the effectiveness of the solution is presented.

The Forth Road Bridge, in Scotland, is a suspension bridge with a main span of 1006 metres and side spans of 408 metres. Approach viaducts to the north and south bring the total length of the structure to about 2500 metres. The bridge was opened in 1964 and traffic has grown significantly from 4 million to 24 million vehicles using the bridge each year. All work carried out on the bridge has to be done under a regime of overnight or limited weekend carriageway or lane closures to minimize delays to users on this important strategic and commuter route. Following a recent internal inspection of the main cables, significant corrosion was found leading to remedial works being planned. In their paper "Forth Road Bridge – maintenance challenges", Andrew and Colford present major maintenance projects on the bridge to strengthen and improve the structure. Recent developments in stay cable systems have enabled the design and construction of longer spans. Due

to the sensitivity of these long spans to the effect of wind, seismic loading, vibration and traffic, monitoring and observing the structure's response has become an essential condition indicator. Several existing cable-stayed structures are showing signs of damage, often caused by wind-induced vibrations and corrosion. Inspection and monitoring provide confidence in the integrity of the structure and enable early detection of potential problems. In order to protect the substantial investment to design, construct, operate and maintain these bridges, owners must consider ways to mitigate the risk of loss and extend the functional life of the bridge. In "Inspection and monitoring of cable stayed structures – design, implementation & performance", Hovhanessian and Stieb describe a risk-based monitoring, inspection, and maintenance management system that has been utilized on several signature bridges including the Rion Antirion in Greece, Penang Bridge in Malaysia, and France's Millau Viaduct and Seysell Bridges. The Seto-Ohashi Bridge is located in the center of three routes that connect Honshu, Japan's main island, and Shikoku. It consists of three suspension bridges, two cable-stayed bridges, and one truss bridge on the strait and some viaducts on the islands. The highway-railway combined bridges have an overall length of approximately 10 kilometers. The bridge, mainly comprised of steel structures, is in a severe corrosive environment. The bridge has served for 18 years since its opening, and the repainting program includes modified paint system for 1,800,000 m^2 of repainting area. In "Repainting of Seto-Ohashi Bridge for increase of durability", Kawaguchi et al describe the development of the repainting specifications and schedule, and the quality control plan for the field repainting work.

The condition assessment of cable-supported bridge cables presents a challenge due to the limited access during visual inspection. In the past decade, new monitoring and inspection technologies have been developed, providing engineers with better information about the condition and rate of deterioration of bridge cables. In their paper "Instrumentation and monitoring of critical structural elements unique to suspension bridges", Hovhanessian and Laurent provide an overview of current cable inspection practices and outline developments in the instrumentation and monitoring of bridge cables. The authors present the evolution of new monitoring technologies and discuss the limitations and challenges associated with case studies. Suspension bridges and through arch bridges with suspended decks use suspender ropes that must be periodically inspected and assessed. Corrosion damage typically starts and is often most extensive on the interior of the rope where the damage cannot be observed with conventional visual inspections. Thus a visual inspection of the exterior of the rope does not necessarily provide a representative assessment of the actual conditions of the rope. In his paper "Non-destructive testing of suspender ropes with magnetostriction", Higgins describes a non-destructive testing (NDT) technique used on suspender ropes to detect either cross section loss or broken wires located on either the interior or exterior of a rope. The technology relies on the principles of magnetostriction to generate a guided acoustic pulse that travels up and down the rope. Where there is a defect in the rope, resulting from corrosion or fatigue damage, a portion of the pulse is reflected back to the source. These reflections can be measured and quantified to determine the relative size and location of corrosion or fatigue damage on a suspender rope. Using destructive testing, Mayrbaurl presents "Wire test results for three suspension bridge cables", where full range stress-strain curves for wire samples were developed. These curves were found to vary from each other by as much as 13%, and calculations were made to determine the effect on the estimated cable strength of this variation. The author concludes that the simple Brittle-Wire Model underestimates the cable strength by approximately 3%. The tensile strengths of the wires on one bridge are compared with those of two formerly inspected bridges, and the cable strengths using these data in the Brittle-Wire Model are calculated. The author also concludes that the variation in the stress-strain curves was caused by variations in the carbon contents of the wires.

Long-term planning to meet future traffic demands on suspension bridges present significant challenges. Suspension bridges usually carry major transportation corridors whose traffic demands continue to increase over time. Several traffic studies of the Bronx-Whitestone Bridge and its surrounding highway network in New York City have predicted the increase in the traffic demand. Based on these predictions, several planning studies investigating the feasibility of providing for the future traffic demands have been performed. In "Planning and engineering for the future: capacity increase

and cable replacement at the Bronx-Whitestone Bridge", Lorentzson et al present the two main concepts that were investigated for increasing traffic capacity at the Bronx-Whitestone Bridge; the addition of a second level and the widening of the bridge. The authors discuss the associated technical challenges along with the environmental impacts, maintenance of traffic, and flexibility for accommodating future traffic demands. The Bear Mountain Bridge spans the Hudson River between Fort Montgomery, New York on the west and the mountain known as "Anthony's Nose" on the east, near Peekskill, New York. The bridge was completed in 1924 and was listed on the National Register of Historic Places on November 23, 1982. During an in-depth inspection of four panels of the main cables in 1991, numerous wires were found broken due to stress corrosion cracking and it was decided to rehabilitate the cables by unwrapping, oiling and rewrapping them. This work began in March 2000 on the southwest backstay. It was observed that the broken wire ends were moving further apart as the wedging and oiling progressed – an indication that the broken wires were slipping. Immediate action was taken to stabilize the backstay with clamping bands. Subsequently, the other three backstays were similarly clamped prior to allowing them to be unwrapped. The southwest backstay continues to be monitored by acoustic monitors that can detect broken wires. While a few new wire breaks have been detected, the condition appears to be stabilized since the addition of the clamping bands. The bridge owner and operator, New York State Bridge Authority, performed a detailed study and developed a design to replace the entire suspension system if it becomes necessary. As an interim measure, the most severely damaged section of the main cables, the southwest backstay, is currently being reinforced by the addition of supplemental structural strands that will be connected to the existing cable saddle and tower steel and to a new anchor frame on a new concrete block foundation anchored to bedrock with high-capacity rock anchors. Permanent strain gauges will be installed to monitor the loads in the rock anchors and structural strands. In "Bear Mountain Bridge cable reinforcement", Moreau et al provide the details of reinforcing the cables of this historic structure. The Jeokgeum Bridge is a suspension bridge with 850m main span, a concrete pylon and stiffening girders. The bridge is located in Goheung Peninsula on the southern coast of South Korea. The completed bridge will support 2 motorway lanes. However, its girder design is generally considered for the future extension to 4 lanes due to increasing traffic. The diameter of its cables is 491 mm, which consists of 380 bundles of parallel wires and 19 strands, the height of pylon will be 137.8 m. Its span-length-to-width ratio of 43.15 constitutes a significant wind design parameter. Park et al present the various design features of the Jeokgeum Bridge in their paper "Planning and design of Jeokgeum Grand Bridge".

Once completed, the reconstruction of the Manhattan Bridge lower roadway will finalize the rehabilitation of all roadway levels on this historic East River suspension bridge in New York City. The advanced deterioration of the deck and steel framing required a complete replacement of these elements, which created complicated construction staging/maintenance of traffic issues. If the lower roadway were closed to traffic, only two lanes would be available to motorists in the peak direction for the morning and evening rush hour traffic, a configuration which had never been used on this structure previously. A study was undertaken with two separate construction staging/MPT alternates; complete closure of the lower roadway, and partial closure of the lower roadway using temporary ramps. A full traffic study was undertaken to determine the effect that each of these alternates would have on the traffic flow and possible measures which could be used to mitigate the adverse impacts. In "Rehabilitation of the lower roadway of the Manhattan Bridge", Perahia et al present some of the complicated planning issues associated with each of the two concepts studied. In 2003, the Forth Estuary Transport Authority (FETA), the owners and operators of the Forth Road Bridge in Scotland took the decision to carry out a first internal inspection of both main cables. The inspection revealed significant corrosion to the main cables, as previously mentioned in an earlier paper in this volume. In their paper, "Forth Road Bridge – first internal inspection, strength evaluation, acoustic monitoring and dehumidification of the main cables", Colford and Cocksedge present details of the inspection findings and evaluation of the current and predicted future strength of the cable. The authors also discuss the decision to install an acoustic monitoring system and implement cable dehumidification on the bridge. It is evident that degradation due to moisture of main cables on suspension bridges is a major problem. Many cases in different countries

have illustrated that traditional corrosion protection systems for main cables do not prevent corrosion, but merely slow it down. This has led to the development and application of dehumidification of main cables as a preventive measure. In "Prevention of main cable corrosion by dehumidification", Bloomstine and Sørensen describe the details of the dehumidification system, its design considerations, results of life cycle cost analysis and experience from systems in operation.

Recent visual inspections of vertical lift bridges have raised concerns that wire ropes used in the systems contain established internal and external corrosion and severe external wear where the ropes rub and vibrate against spacers and attachments. The majority of these lift bridges exceed 40 to 50 years in age with some as old as 90 years. With the exception of a few bridges, only visual inspections have been conducted on these wire ropes. Improvements and reductions in the physical size of existing instrumentation have made it possible to inspect these wire ropes nondestructively on a periodic schedule. In "Electromagnetic inspection of wire ropes – vertical lift bridges", Hall presents the theory and operation of electromagnetic inspection procedures for wire rope in vertical lift bridges. Major advances in the technology used to design bridge wire rope and strand assemblies have been accomplished recently. In "Wire rope and strand assemblies in bridge applications", Klein discusses the advancements in corrosion inhibitive materials available for hangar assemblies along with their advantages and disadvantages. The author presents the importance of proper assembly fabrication including prestretching and proofloading techniques and why this must be done to ensure proper installation of the completed assembly. The paper sheds light on how the socket selection and attachment style will affect the fabrication tolerances and provide simple installation and tensioning onsite. Corrosion protection for prestressing steel materials used for prestressed and post-tensioned structures is one of the most important factors needed to increase the structure service life. The corrosion protection is usually applied at the construction site by cement grouting or other means. However, the corrosion protection quality is dependent on the operator's skills, grout quality, and weather, among other factors. Several prestressing materials with factory applied corrosion protection have been used in various applications, such as prestressing, post-tensioning, stay cables and ground anchors. Epoxy coated and filled prestressing steel strand introduce a common material produced with a factory-applied corrosion protection. This material is protected by epoxy coating on both the outside and inside of the steel strand. It has been applied to various bridges since 1983 such as cable-stayed bridges, extra-dosed bridges and box girder bridges. In his paper "Epoxy coated and filled prestressing steel strand cables", Shinichi introduces several evaluations conducted using epoxy coated strands to maintain long service life of strand-supported bridges.

The international contributions of an outstanding body of technical experts from all over the world assure the lasting value of this set of proceedings. Presenting state-of-the-art material and providing an authoritative account on the developments in the field, this volume forms essential reading to anyone working on cable-supported bridges.

The editor thanks the authors and expresses a special note of gratitude to the reviewers. This publication is a result of the sacrifice of time and effort, dedication and collective wisdom of all contributors.

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New York City, August 2006

1 Design, construction and operation

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Chapter 1

Stonecutters Bridge – designing for operation

Matt Carter & Naeem Hussain
Ove Arup & Partners, Hong Kong

ABSTRACT: Highways Department (HyD) of Hong Kong SAR is building a bridge across the Rambler Channel at the entrance to one of the world's busiest container ports at Kwai Chung. The bridge will become one of the longest spanning cable-stayed bridges in the world. Due to the spectacular location adjacent to Hong Kong's Victoria Harbour, HyD decided to hold an international design competition to select the bridge concept. This led to the scheme of a cable-stayed structure with freestanding towers on land and a clear span of 1018 m to leave the navigation channel unobstructed. The deck is a twin box girder which passes either side of the towers.
This paper describes the measures taken during the design to provide a durable structure with all the necessary facilities for operation. Stainless steel was used in the towers, both in fabrications and in reinforcement. A comprehensive set of facilities for inspection and maintenance of the structure was included in the design and the bridge is fully instrumented with a real time Wind And Structural Health Monitoring System (WASHMS).

1 INTRODUCTION

Stonecutters Bridge is part of Route 8 (formerly Route 9), a new east-west expressway providing a further link between Hong Kong International Airport at Lantau Island and the urban areas of West Kowloon. Tsing Ma Bridge and Kap Shui Mun Bridge completed in 1997 also form part of this route (see Figure 1).

Unlike the other major bridges of the link to the airport, Stonecutters Bridge will be clearly visible from the populated areas of West Kowloon and Hong Kong Island including the Central business district. Highways Department of Hong Kong saw the potential of the site, with the dramatic setting of a world record cable-stayed span in one of the busiest harbors in the world, and decided to call an international design competition to select the bridge concept. The design competition took place in the first half of 2000 and the winning concept was taken forward as the reference scheme for further development and detailed design (Gimsing et al., 2003).

The contract for the detailed design was awarded to Arup in March 2001, with COWI as principal sub-consultant. A thorough technical review of the reference scheme was undertaken, and completed in early 2002. This led to a number of modifications to the concept all of which have been incorporated in the detailed design which took place over the following 18 months. Tender was called in August 2003 and the US$350M construction contract was awarded to the Maeda – Hitachi – Yokogawa – Hsin Chong Joint Venture in April 2004. The estimated completion of construction is the end of 2008.

2 THE BRIDGE

The bridge, with a vertical clearance of 73.50 m over the navigation channel, is an innovative cable-stay bridge with a total length of 1596 m and a main span of 1018 m (3,340'), and on completion will be one of only two cable-stay bridges in the world with a main span in excess of 1 km. (Figure 2)

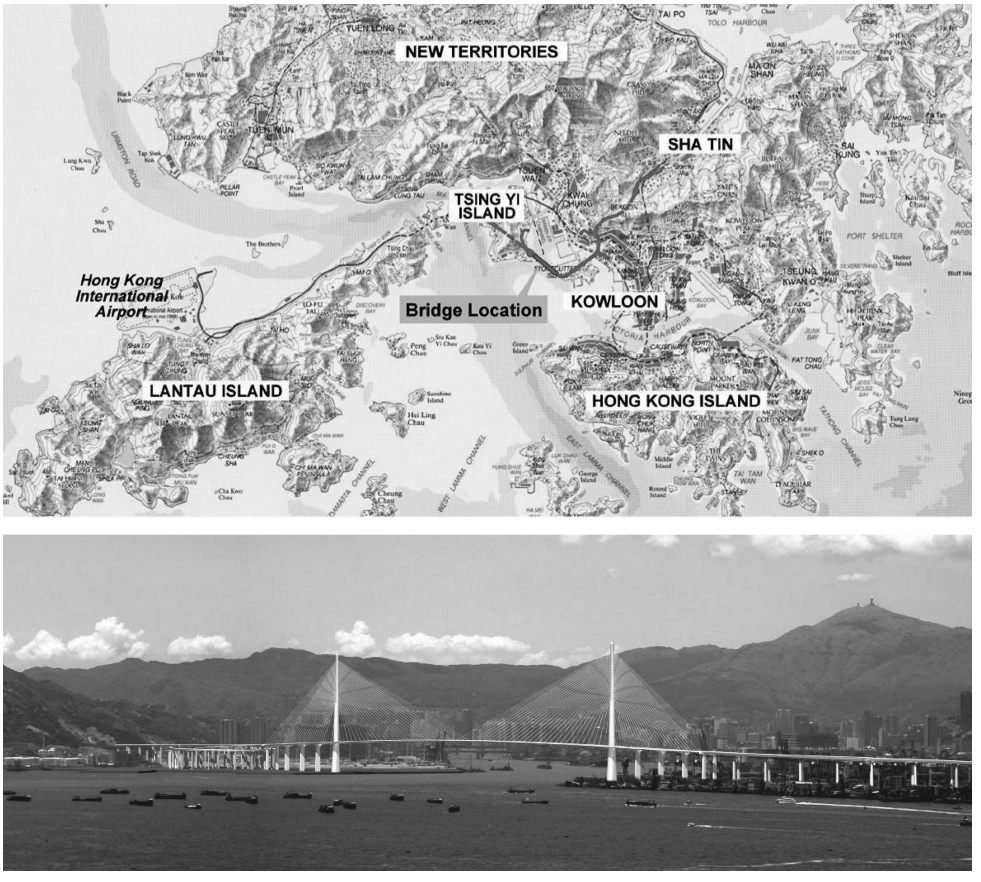


Figure 1. Stonecutters Bridge in Hong Kong harbour.

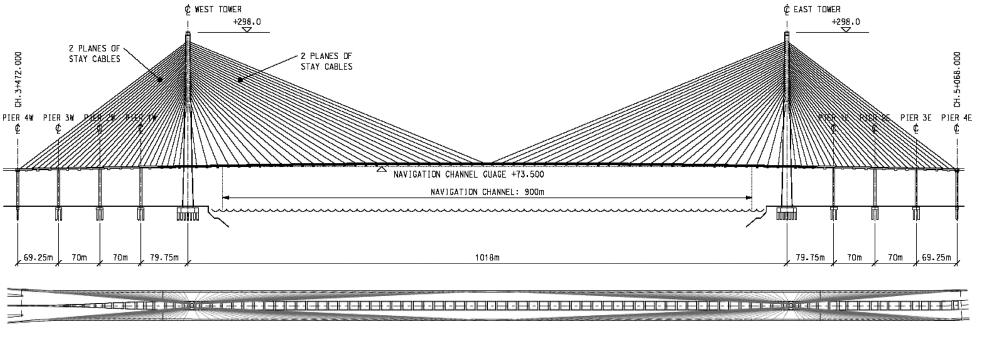


Figure 2. Elevation and plan.

The 53.30 m wide main span comprises of two separate orthotropic steel box girders connected by transverse cross girders spaced at 18 m centers, which coincide with the stay-cable anchorage spacing. The back-spans are in prestressed concrete and are monolithic with the backspan piers. Stay cables are in two planes arranged in a modified fan layout and attached to the outside edges of the deck girders. The mono column towers are located in the gap between the decks, and will be constructed in concrete up to level +175 m, and in composite construction with an outer stainless

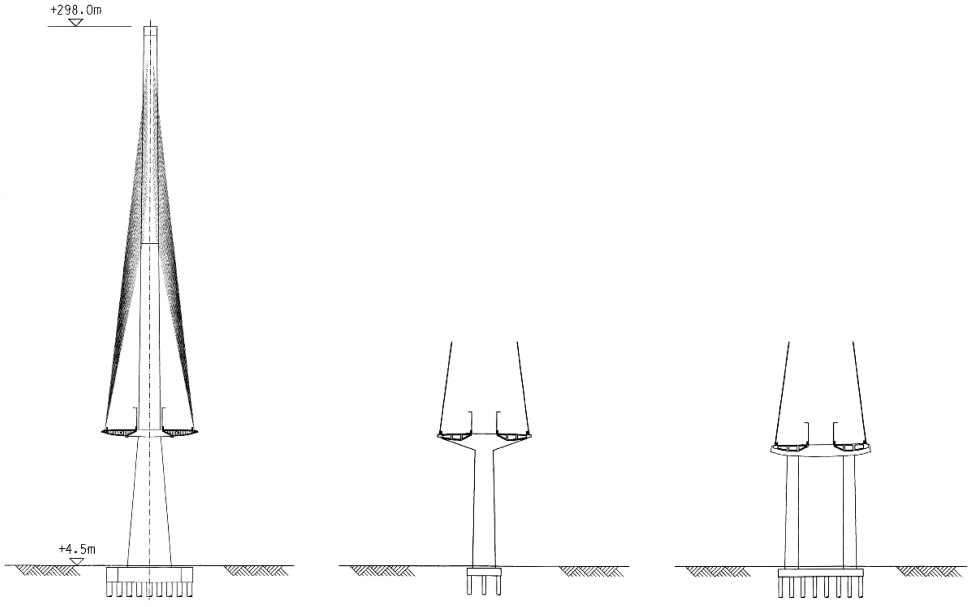


Figure 3. Tower, intermediate pier and end pier.

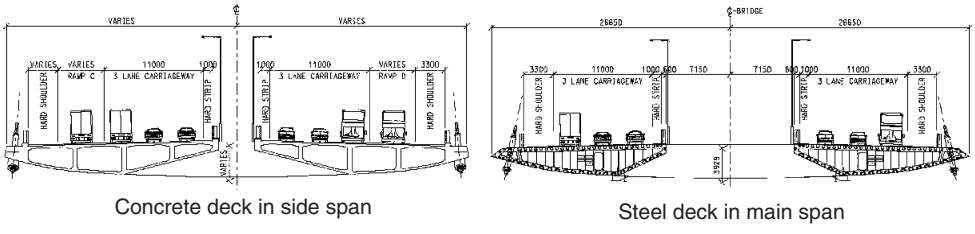


Figure 4. Deck cross sections.

steel skin to +293 m and topped with a light feature to +298 m. The towers are circular from deck level upwards, with a diameter of 14 m at deck level and 7 m at the top. (Figures 3 and 4)

The backspan piers are spaced at 79.75 m, 70 m, 70 m and 69.25 m. The three intermediate piers have single rectangular tapered column shafts, whilst the end piers at the interfaces to the adjoining viaducts are twin column portal structures. Both sides of the bridge are on reclaimed land and foundations are large diameter bored piles taken down to rock. Pile lengths are between 50 m and 110 m and diameters are between 2.2 m and 2.8 m (Figure 5).

2.1 *Articulation*

In the reference scheme the interface between concrete deck and steel deck was in the main span and the concrete deck was fully monolithic with both the towers and the side span piers thus eliminating bearings and reducing maintenance. This kind of fully monolithic solution was achieved on the Pont du Normandie where thermal expansion of the deck was assisted by the high camber due to steep approach gradients.

However, the review of the reference scheme showed that it would be advantageous to eliminate the monolithic joint at the tower and introduce instead a so-called "open joint", where the deck is supported laterally on sliding bearings and longitudinally by hydraulic buffers for short term loading

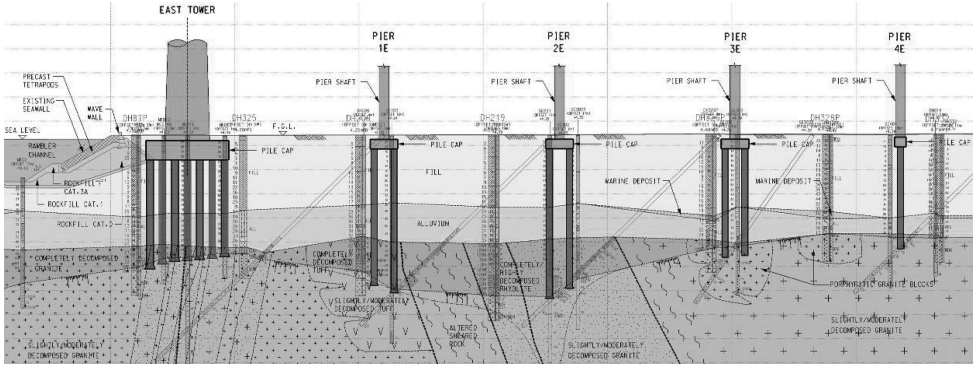


Figure 5. Foundations – east side.

(dynamic loading from wind and seismic action). For long term loading (highway live loads, temperature effects, mean wind, shrinkage and creep etc.) the deck can move longitudinally relative to the tower. This arrangement has a number of advantages over a monolithic connection. It reduces the restraining forces on the tower from temperature effects for the bridge in-service and it reduces the torsion in the tower during construction as well as in-service. Buffeting analysis showed that, with the monolithic joint, the tower section just below the deck was overstressed during the free cantilevering construction of the main span, because side sway of the deck was resisted as torsion in the tower. The open joint arrangement also eliminates the "hard point" in the deck for bending about a horizontal axis avoiding a combination of high bi-axial bending and compression.

Once the open-joint was introduced, further optimization studies showed that the loading on the steel-concrete deck interface could be reduced if it were relocated to a position in the back span 49.75 m from the tower. Thus the steel deck continues past the tower.

3 OPERATION OF THE BRIDGE

There are three major cable supported highway bridges currently in operation in Hong Kong; Tsing Ma Bridge, Ting Kau Bridge and Kap Shui Mun Bridge which were opened in 1997–98 and formed part of the construction works associated with a fixed link to the relocated Hong Kong International Airport.

These three bridges are in close proximity to each other and fall within the Tsing Ma Control Area (TMCA), a 21 km expressway network illustrated in Figure 6. In addition to the three cable supported bridges the TMCA also includes various other elevated structures, a tunnel and other facilities such as a Traffic Control and Surveillance System (TCSS) and Wind And Structural Health Monitoring System (WASHMS).

Management of the TMCA is carried out by a private contractor. Tsing Ma Management Limited (TMML) was awarded the original Management, Operation and Maintenance (MOM) Contract by the HKSAR Transport Department and in January 2001 was awarded a further 6 year contract. The contract is for a fixed price but additional or non-scheduled works are paid separately. Payment to TMML is mainly funded by toll revenue.

The Transport Department has also indicated its intentions to establish a Route 8 Control Area which would be managed under a similar open tendered contract. The operation of Stonecutters Bridge would fall under the Route 8 Control Area MOM Contract.

Whilst the Control Area MOM Contractor may carry out maintenance of the WASHMS, the operational responsibility rests with a Government Monitoring Team from Highways Department. Due to the highly specialized nature of monitoring the WASHMS, the TMCA team will also take up responsibility for operation of the SCB-WASHMS.

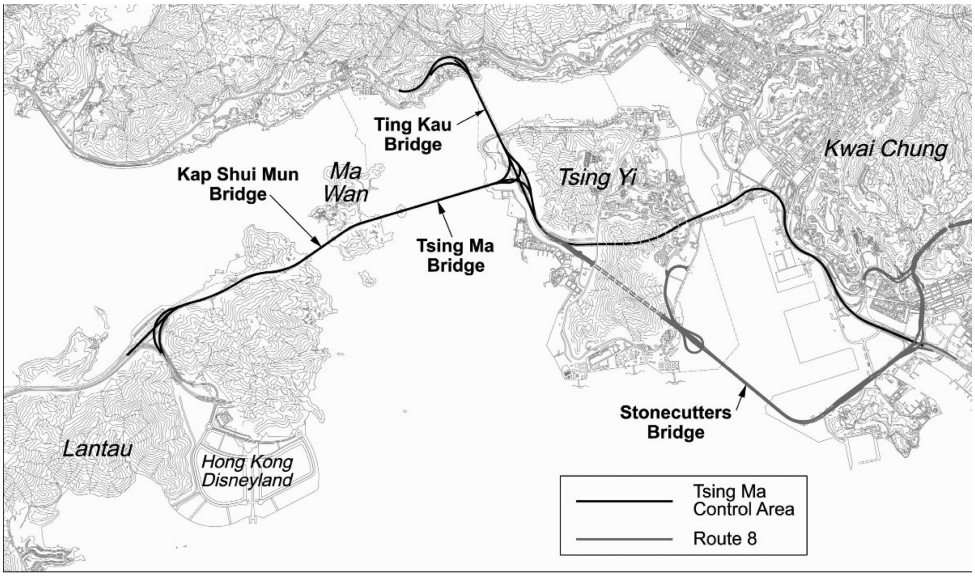


Figure 6. Tsing Ma Control Area (TMCA).

4 DURABILITY DESIGN

4.1 *Durability assessment*

A structural durability assessment was carried out to provide the required degree of confidence that the 120 year design life of the bridge could be achieved. The durability assessment recognized that some maintenance and repair is implied by the achievement of a design life. Furthermore the definition of the design life was that the design criteria should be achieved throughout the life of the structure, not that the materials or components should remain unchanged. Thus the durability assessment examined three aspects of deterioration:

1. The processes of deterioration
2. The means of diminishing deterioration by the use of inherently durable materials or the protection of less durable materials
3. The means of mitigating deterioration by maintenance.

Whilst the primary structural elements must achieve the design life without replacement, for other secondary elements replacement is feasible and a shorter service life is assumed.

The bridge is situated on reclaimed land close to the sea at Stonecutters Island and Tsing Yi Island. The below ground environment is of saturated marine reclamation fill down through alluvial deposits to rock. Above ground the atmospheric exposure for all parts of the bridge is dominated by the presence of airborne marine salts. Various microclimates were identified in relation to the deterioration mechanisms that exist for different materials.

The main output of the durability assessment was to define the material specifications and identify key construction issues which need to be controlled.

4.2 *Steel deck*

4.2.1 *External corrosion protection*

Steel highway structures in Hong Kong are generally required to be galvanized or coated with metal spray prior to painting. For the very large welded fabrications which make up the steel deck,

Table 1. Steel deck paint specification.
Preparation: Blast cleaned to Sa 2½ of ISO8501-1

Coating	Material	Thickness (μm)
Primer	Zinc Rich Epoxy	40
Barrier 1	Epoxy Micaceous Iron Oxide	150
Barrier 2	Epoxy Micaceous Iron Oxide	150
Finish	Polyurethane	50

All coats to be shop applied.

galvanizing is not a practical option. A high-build epoxy based paint system on a zinc rich primer has been specified (Table 1). The big advantage of this specification is that it eliminates flame sprayed metal thereby making the system very much quicker, easier and cheaper to apply with no real loss in performance. Cost savings over the conventional metal spray and paint are up to 30%. This departure from standard Hong Kong practice was achieved by demonstrating the successful use of this system on existing structures including the Oresund Bridge.

The twin box decks are fabricated in sections from welded steel plate and then assembled into erection segments in a yard in Shatian, close to the site and adjacent to the Pearl River for convenient transport. The full paint specification is applied in the controlled environment of a paint-shop established at the yard and checked for conformity before being shipped to site by barge. Site welds or areas that require making good will be treated on site by grit blasting and application of the full paint specification under controlled environmental conditions and will be subject to the same rigorous conformity checking as the main steelwork.

The system is expected to have a life to first maintenance of 20–30 years after which touch up work and repainting will be required. The large flat surfaces which make up the steel deck combined with full access from an under-bridge gantry make this maintenance task readily achievable.

4.2.2 *Internal corrosion protection*

Although research by a number of workers has shown that the incidence of corrosion and the rate at which corrosion occurs in box sections and other spaces in bridges is usually much less than would be anticipated it has become increasingly common practice to protect the inside of large steel box girders by dehumidification.

This system is applied for Stonecutters Bridge, with the aim being to control the internal environment to provide a relative humidity of less than 60%. This will effectively prevent corrosion of the internal surface of the steel boxes and so eliminate the need for the sophisticated corrosion protection scheme used on the outside. However a light-colored primer will be provided for the internal steel surfaces in order to inhibit corrosion occurring during transportation and erection and to provide a lighter internal environment.

4.3 *Stainless steel reinforcement*

The base of the towers is exposed to sea spray even though the tower is some 18 m away from the seawall and is categorized as Exposure Class – Very Severe in accordance with the Hong Kong Structures Design Manual for Highways and Railways (SDMHR). However, the towers are provided with a very high level of durability protection in excess of that required to satisfy this exposure class. This was justified by the special nature of the structures both in terms of difficulty of access for repair and their considerable aesthetic importance. Furthermore, the exact chloride loading in the aggressive marine environment is not certain.

In particular, for the lower part of the tower up to level +175 m, stainless steel reinforcement is being used for the outer layer of vertical reinforcement together with stainless steel links. The total quantity of stainless steel reinforcement used in the two towers is 2,800 t of which nearly half is

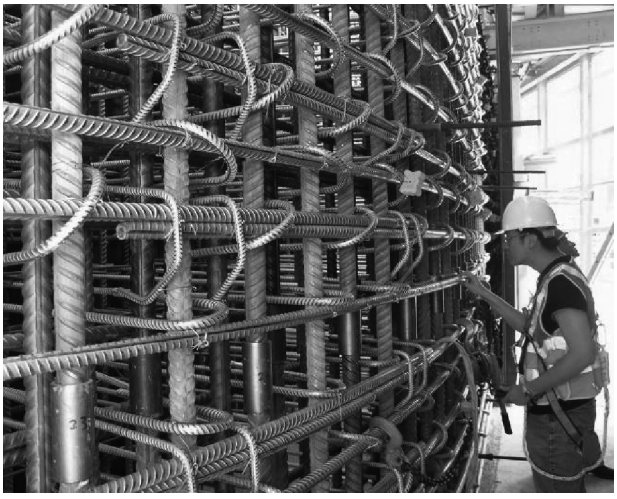


Figure 7. Stainless steel reinforcement in tower.

50 mm diameter bars (Figure 7). This compares to a total of 6,300 t of ordinary carbon steel reinforcement also used.

Although there is a cost premium associated with the use of stainless steel reinforcement it is probably the most reliable way to provide a robust solution for a long design life with a very low risk of structural maintenance or repair. Highways Department was keen to specify this material in order to ensure a structure with a lasting high quality finish.

In addition to the stainless steel reinforcement, a high strength concrete is used (60/50 MPa cube/cylinder strength) and the cover to the reinforcement links is 60 mm. Furthermore a silane finish has been specified to avoid mould growth which can occur on concrete structures in Hong Kong's humid environment.

Stainless steel reinforcement is not being used in the side span piers. This is due to their increased distance from the sea wall meaning that they are not directly exposed to sea spray, the easier access for repair of these below deck structures and also their lesser aesthetic importance compared to the towers.

4.4 *Stainless steel upper tower*

Access for maintenance of the outer steel skin of the upper tower was identified as being of particular difficulty leading to a high maintenance cost if the corrosion protection was based upon a traditional system of coating. Instead, stainless steel was adopted to provide a low maintenance solution. The duplex grade 1.4462 is specified for its high strength and good resistance to general and localized corrosion. A non-reflective finish is achieved by shot peening. A total of 1,600 t of stainless steel plate will be used in the skin assemblies which are being fabricated by GDSF in Zhongshan, China.

During the design, a prototype fabrication (see Figure 8) was manufactured in full scale to confirm the effects of fabrication (shear studs, splice plates etc.) on surface finish and appearance; to develop indicative welding procedures; to confirm the practicality of forming the cone shape and; to develop the bolted splice between lifts. It is important to achieve a tight fit between steel skin segments as compression forces are assumed transferred between segments.

Slip resistance of duplex type stainless steel is not well documented, therefore a series of tests were carried out to establish the slip resistance of the bolted connection and the roughness required to meet the requirement of a minimum slip factor of 0.2.

Figure 8. Prototype of stainless steel skin.

5 INSPECTION & MAINTENANCE FACILITIES

A comprehensive set of facilities for inspection and maintenance of the structure has been included in the design. In addition to fixed access facilities throughout the bridge (walkways, stairs, ladders etc.), a suite of motorized access machines will be provided including under-deck inspection gantries, lifts within the towers, an internal deck shuttle, a stay cable inspection gantry and an access platform to be suspended from a derrick crane at each tower top.

The overall approach is to ensure that, as far as possible, normal inspection and maintenance activities can be carried out with minimum disturbance to the traffic. At the same time easy access for maintenance personnel must be provided. This is achieved by arranging the main access at the tower base. From here virtually every location in the bridge can be reached.

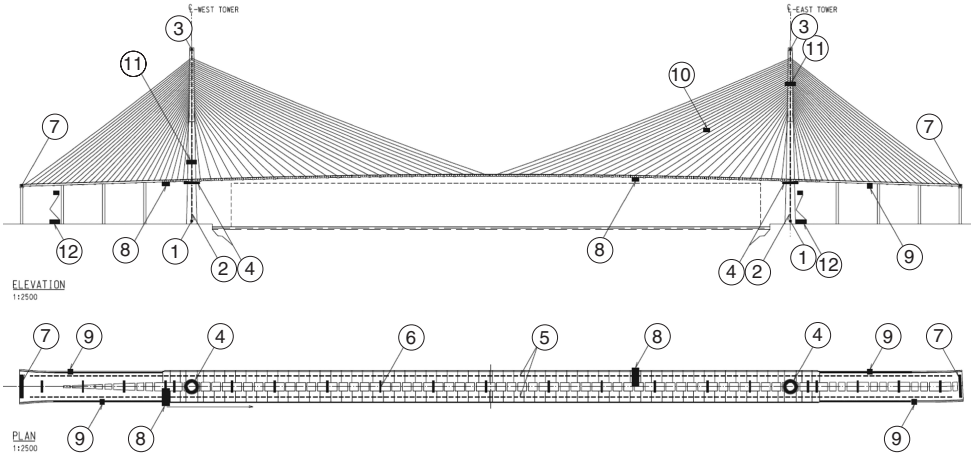
5.1 *Overview*

Figure 9 gives an overview of the access facilities. From the main entry point at the tower base the full height of the tower is accessible by an internal rack and pinion lift.

An exterior platform is provided around the tower just below deck level which is accessible from a chamber within the tower. From this platform entry is gained to the interior of the deck. The lateral bearings and hydraulic buffers are also accessed from this platform.

Inside the deck a walkway and a shuttle train provide access along the entire length for personnel and equipment. Access is provided to every cross girder although certain cross girders are nominated “cross passages” with improved access facilities. At either end of the deck a chamber allows access to the expansion joints and approach viaduct bearings.

Additional exterior access is provided by gantries. The exterior of the towers above deck level is accessible from a cradle suspended by a derrick on the tower top. The exterior of the steel and concrete decks is accessible under-bridge gantries. Finally a gantry is used for access to the stay cables which would be deployed from a lorry temporarily parked on the hard shoulder.



1	Access door at tower base	7	Internal chamber for expansion joint access
2	Lift & emergency stairs in tower	8	Pair of under-bridge gantries for steel deck
3	Tower top maintenance unit derrick	9	Four under-bridge gantries for concrete deck
4	Exterior tower platform	10	Stay cable maintenance gantry
5	Access walkways and shuttle in deck	11	Cradle suspended from derrick on tower top
6	Cross passages (including emergency escape)	12	Mobile elevated working platform

Figure 9. Overview of access facilities.

Access to the exterior of the lower part of the tower, the backspan piers and the concrete deck soffit is via a mobile elevated working platform. However inspection of these plain concrete surfaces is expected to be relatively infrequent.

In addition to these main access systems a number of emergency or contingency access facilities are provided.

5.2 *Internal access*

5.2.1 *Deck-tower junction*

The focal point of the access facilities within the bridge is at the deck-tower junction which is immediately accessible from the main entry point via the tower lift (Figure 10). Inside the tower two platforms are provided, an access platform at Deck Level (+72.5 m) and the tower structural diaphragm provides a further access level at +79.0 m denoted Diaphragm Level.

From Deck Level it is possible to exit onto an exterior platform which encircles the tower and allows inspection and maintenance of the hydraulic buffers and lateral bearings as well as providing access to the deck. The parking position of the main under-bridge deck gantries is also adjacent to the exterior tower platform and access between the two is provided.

An access door is also provided in the tower at Diaphragm Level. This is not intended for normal use since it opens onto the fast lane of the carriageway. However, it provides greater flexibility for extraordinary maintenance operations or emergency escape.

5.2.2 *Deck interior access*

Figure 11 illustrates the deck interior access in the main span. The main access is along walkways located within the center of each of the longitudinal girders.

A motorized deck shuttle is provided in the south girder with a variable speed drive up to 4 m/s allowing it to traverse the main span in just over 4 minutes. Because the shuttle shares the walkway space it is fitted with an operating alarm and lights. In addition maintenance workers will be

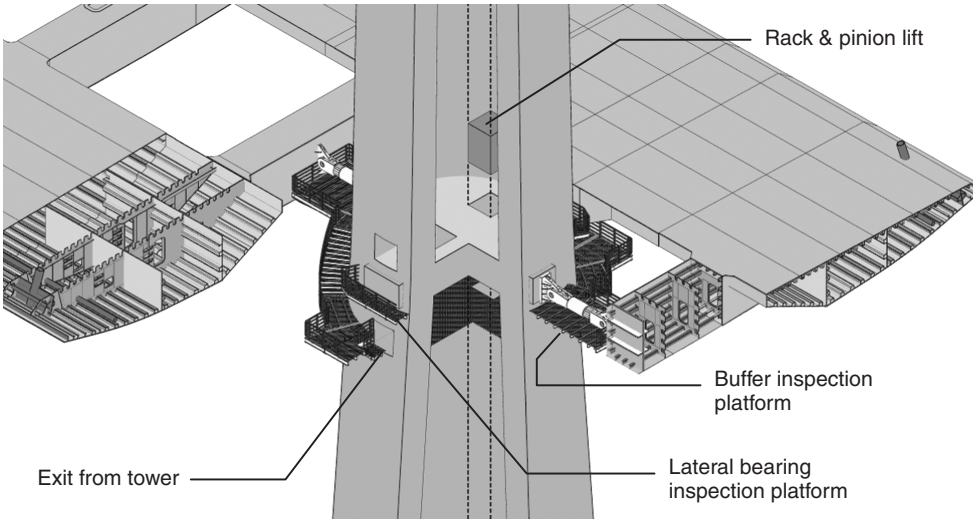


Figure 10. Access facilities at deck-tower junction.

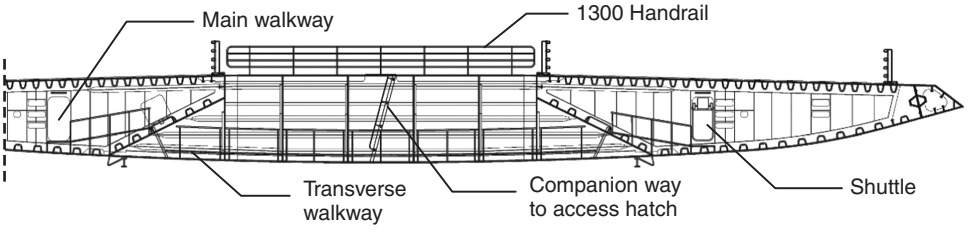


Figure 11. Deck interior access in main span at cross passages.

able to isolate sections of the track in order to work in safety. The shuttle is designed to carry two operatives together with 150 kg of equipment.

It is possible to access into every cross girder for inspection and maintenance. However, at typically 90 m intervals there is a designated cross-passage. The cross-passages provide easier access between the decks in combination with an exit hatch to the top of the cross girder. This is primarily provided as a means of emergency escape but will also have the benefit of allowing greater flexibility in accessing and ventilating the deck. The direction to the nearest cross passage is marked in every deck cell.

The service trays are conveniently located alongside the access walkways but only to the outside of the walkways thus preventing any obstruction to access into the cross passages.

A hatch is provided between the cross girders and the north longitudinal girder in order to force the circulation of dehumidified air along the entire length of each longitudinal girder.

5.2.3 *Access to upper stay anchorages*

The upper stay anchorages are located inside the anchor box, a fabricated steel box approximately 85 m tall. Inside the anchor box, a platform is provided at the level of every stay anchor to facilitate inspection and maintenance. Due to structural constraints it was not possible to allow large height penetrations into the box and the typical opening is only 1.2 m high which is not sufficient to access each anchorage platform directly from the lift. For general access personnel will take the lift to either just above or just below the anchorage box and then use an access stair to get to the desired location (see Figure 12). Safety covers close up the access opening on the lift side of the anchorage box to prevent collision between the lift and maintenance workers.

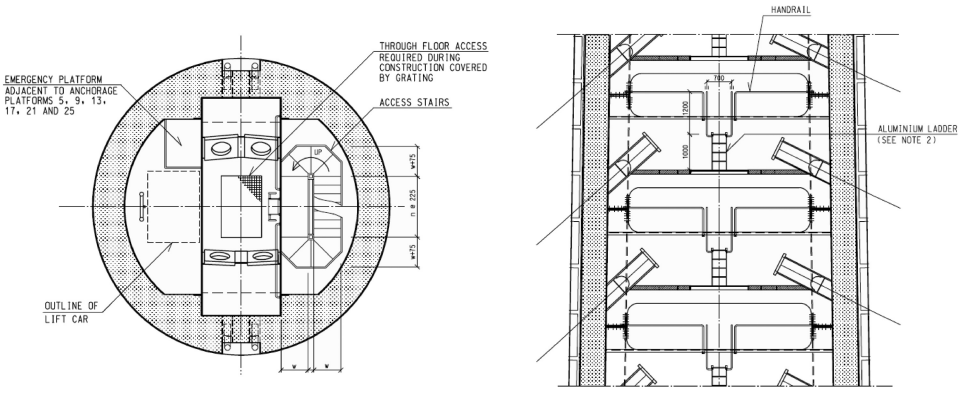


Figure 12. Access arrangements for upper tower anchor box.

Bearing in mind that there are 27 flights of stairs between the two lift stops, access to the anchorage platforms towards the middle of the anchor box could be considered fairly restricted. Fortunately relatively little maintenance is anticipated, the anchor box steelwork is provided with the full paint specification which will have an extremely long life inside the enclosed environment of the tower. The dead end anchors of the stay should require little other than inspection until the stays need replacing at the end of their design life which is expected to be 60 years. However, in the interest of maximum accessibility, the tower lift is specified with "abnormal" lift stops at every anchorage platform level to allow equipment to be transferred direct from the lift to the anchorage platform. This requires removing the safety covers that close the access hole in the anchor box and so, as a safety precaution, these stops can only be made by unlocking an additional control panel inside the lift.

5.3 *External access machines*

5.3.1 *Tower top maintenance unit*

The tower extends to nearly 300 m above ground level as a smooth cylinder. External maintenance of the tower is kept to a minimum by the use of a stainless steel skin to avoid repainting. However, it is still necessary to arrange access to the exterior of the upper tower to maintain the architectural lighting and periodic cleaning/general inspection of the steel may also be required occasionally. A tower top maintenance unit (Figure 13) is provided which is similar to building maintenance units typically used for cleaning windows of high-rise buildings. The particular challenges for Stonecutters Bridge are the confined space available at the top of the tower for the maintenance unit, the curved surface of the tower with varying diameter and the stay cables intersecting the tower at varying plan angles.

Two interchangeable cradles are provided with the unit. A 3 m wide cradle for general use and an 800 mm wide cradle for use in the narrow space between the stay cables where the architectural lighting is located.

The unit is designed to work up to a mean hourly wind speed of 15 m/s measured at the tower top. A series of anchor points will be provided on the tower at 20 m height intervals which can be used to restrain the suspension ropes to limit the amount of sway of the cradle.

The tower top maintenance unit is protected from the elements by housing it inside the tower top lighting feature which is a glazed circular chamber. Opening doors in the roof of the chamber allow deployment of the unit.

5.3.2 *Under-bridge gantry for steel deck*

Provision of an under-bridge gantry is routine for a steel box girder. For the case of Stonecutters Bridge there are some additional complications due to the void between the decks. As well as creating a

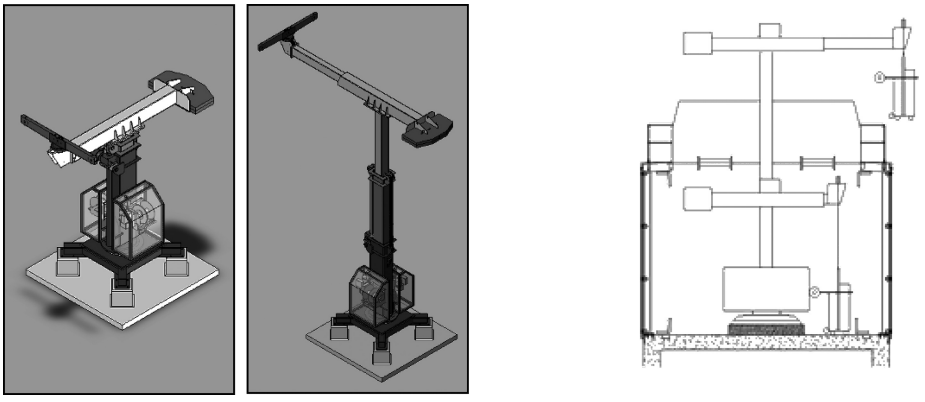


Figure 13. Tower top maintenance unit.
Source: Tractel.

more complex arrangement of surfaces that must be accessible the void also increases the total deck width to 53.30 m.

During the design it was considered whether this entire width should be covered by a single gantry or a pair of gantries. Since the steel deck extends a short distance into the side span it is advantageous to have a gantry which can pass the tower. The logical way to do this is to split the gantry into two, one beneath each deck. Once the gantry is designed to do this there is significant operational flexibility in designing the two gantries to be completely independent.

Both aerodynamic stability and keeping wind drag forces to a minimum were an important objective of the deck design and this includes the gantry rails. The gantry is supported from a stainless steel rail on the top of the leading edge of the deck and from an underslung beam on the lower surface which is located in the wake of a guide vane. This arrangement was found to provide an acceptable deck section for both flutter stability and vortex shedding vibrations.

Access to the internal inclined web of the longitudinal girders and the cross girder webs is achieved through a lifting platform.

In the conceptual design it was envisaged that the gantry might hinge down in order to pass the tower. However the suppliers proposed alternative of a telescopic gantry, which is illustrated in Figure 14, has been accepted.

The parking position of the gantry is adjacent to the exterior tower platform and access between the two is possible thus allowing the gantry to be used without any disturbance to traffic on the bridge. However, given the relatively slow safe operating speed of the gantry there would be some time wasted in driving to and from the tower at the beginning and end of each shift. Therefore it is also possible to access the gantry from the deck level with the use of climbing harnesses for safety. This gives the bridge operator the flexibility to park a maintenance van temporarily in the hard shoulder to drop off/collect operatives at minimal disruption to the traffic.

5.3.3 *Deck gantry for concrete side spans*

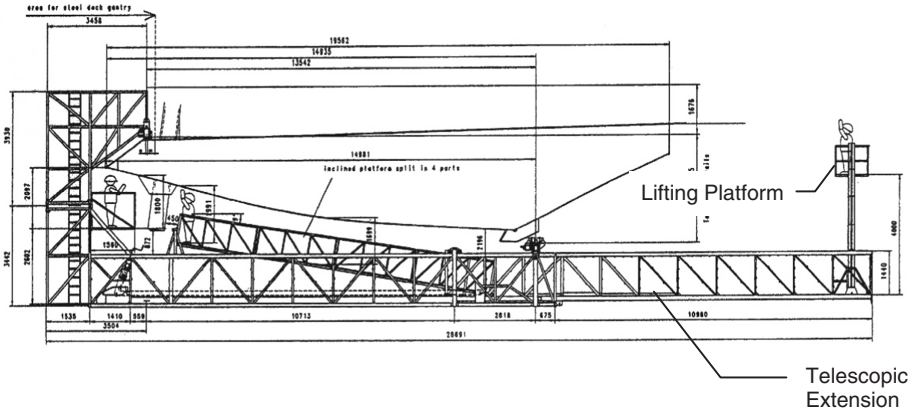
At the side span piers a monolithic crosshead drops down beneath the soffit of the deck. Due to this geometry it is not possible for the main span deck gantry to continue into the concrete side spans.

Whilst the plain concrete soffit of the girder has a minimal inspection requirement it will be necessary to inspect the lower anchorages of the stay cables periodically. Therefore, small gantries which run along the edge of the side spans are provided as illustrated in Figure 15.

It was originally envisaged that either one or two demountable gantries would be supplied which could be used in rotation. However, eventually a full complement of four gantries (both edges of both side spans) was specified to avoid the relatively complex operation of demounting the gantry from 70 m elevation.

Access to/from the gantry is from deck level as described for the steel deck gantry.

(a) Gantry in Ordinary Working Position



(b) Gantry with Extension Retracted to Pass the Tower

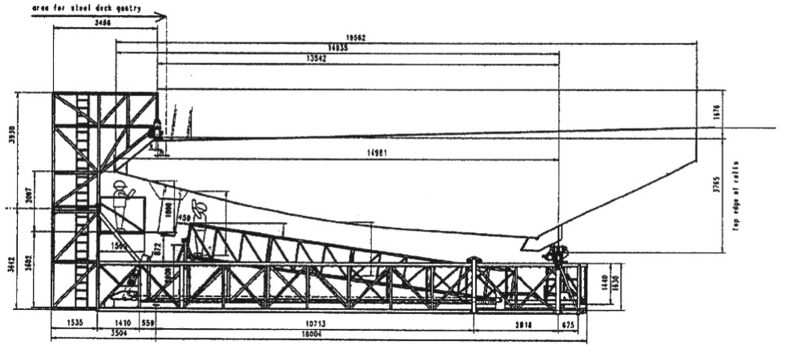


Figure 14. Under-bridge gantry for steel deck.
Source: MOOG Asia Limited.

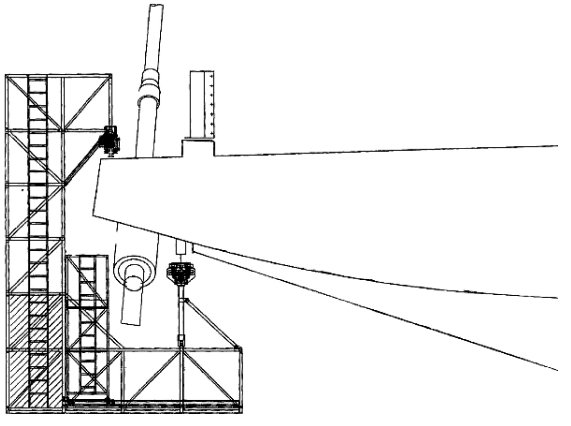


Figure 15. Deck gantry for concrete side spans.
Source: MOOG Asia Limited.

Figure 16. Ting Kau Bridge stay cable gantry.
Source: Tractel.

5.3.4 *Stay cable gantry*

The National Cooperative Highway Research Programme (NCHRP, 2005) distributed a questionnaire to all state and provincial DOTs in the United States and Canada. The same questionnaire was also sent to all members of the Post-Tensioning Institute's (PTI) Cable-Stayed Bridge Committee, as well as major U.S. and Mexican stay cable suppliers and testing companies. Many survey respondents "emphasized the desirability of finding effective ways to inspect cables visually."

A successful system for achieving this close visual inspection is in operation on Ting Kau Bridge. An inspection gantry is provided which runs along the full length of the cables. The gantry comprises a cradle suspended from a hoist unit which runs along a tensioned guide rope installed above the cable (see Figure 16).

Two similar gantries will also be supplied for Stonecutters Bridge, the main difference being that the gantries will each have a pair of cradles instead of the single sided arrangement used on Ting Kau Bridge. This allows both sides of the cable to be inspected in a single pass. The tensioned guide rope is temporary and its installation is facilitated with the tower top maintenance unit. Since the stay cables cross the carriageway, installation and removal of the guide rope must be carried out at night with temporary closure of one carriageway. However, inspection can be carried out in the daytime above the carriageway.

The gantry is designed to operate in wind speeds up to 15 m/s. An anemometer is provided which gives audible and visual warning signals if the wind speed exceeds 10 m/s.

5.4 *Design & procurement*

The conceptual design of the access facilities was developed in consultation with the relevant government departments. In particular, comments were sought from the divisions of Highways Department responsible for administering the TMCA so that the experience of maintaining the cable supported structures there could be fed back into the design of Stonecutters Bridge.

Stonecutters Bridge is constructed under a traditional remeasured contract with a full detailed design prepared by the Engineer and a Bill of Quantities tender submitted by the Contractor. However some elements of the bridge, including the access facilities, are procured as contractor designed items. The performance specification of the facilities are described in detail in the Particular Specification which also makes reference to a sequence of drawings which show the location and layout of the facilities.

5.4.1 *Fixed access facilities*

For the fixed access facilities, a full detailed design by the Engineer could have been practical. However, it was judged that a contractor's design could offer greater economy by allowing the use of proprietary systems.

The specification for the fixed access facilities is that the contractor's design shall be in accordance with BS5395-3 "Code of practice for the design of industrial type stairs, permanent ladders and walkways." However, high elevation external locations are classified as High Exposure and the requirements of BS 5935-3 for protective barriers are enhanced. To ensure adequate psychological and physical protection the top handrail is at 1300 mm with a toe plate, three intermediate rails and a mesh in-fill.

5.4.2 *Motorized access machines*

In the case of the motorized access machines, detailed design can only be practically carried out by the supplier. The development of the performance specification was carried out in consultation with suppliers in order to achieve practical and safe solutions. However, the specification was deliberately left open enough to allow competitive tendering for the supply subcontracts.

Further development of the design of the access machines by the contractor's suppliers is based on a number of submissions, each of which must be approved by the Engineer. The submissions are itemized below:

- Scheme Design
- Detailed Design
- Fabrication
- Testing
- Operation & Maintenance Manual

In addition a training programme is required for the operation personnel which includes both theoretical and practical instruction and active participation during the commissioning of the facilities.

6 WASHMS

A Wind And Structural Health Monitoring System (WASHMS) will be provided for the bridge to provide real time data and also to allow investigation of the structure to be undertaken after an extreme event such as a typhoon (hurricane) or an earthquake.

Typhoon loading in particular has been significant for the design of the bridge. Hong Kong's most powerful typhoon on record, Wanda, occurred on the morning of 1 September 1962. The maximum 10-minute mean wind speed recorded at the Observatory during Typhoon Wanda was 78 knots (144 km per hour), and the maximum gust 140 knots (259 km per hour).

The bridge is designed for a 10-minute mean wind speed of 53 m/s at deck level (190 km per hour) with a sophisticated dynamic wind loading model to define the gust effects. A wind buffeting analysis was carried out during the design to determine the dynamic response of the structure. The wind loading proved to be governing for many of the bridge elements.

Whilst it is hoped that the bridge may never experience a typhoon of this magnitude it is almost certain that lesser events will strike during its lifetime (typhoons are frequent in Hong Kong, even the word is said to be derived from the Cantonese "tai fung" meaning great wind). These events will present the opportunity to gather valuable data. By measuring and analyzing the wind during the event and the simultaneous response of the bridge it will be possible to confirm and improve upon the mathematical models that are used for contemporary wind buffeting design of long span bridges.

On the other hand, the observed seismicity in the region is relatively low, similar to that observed in the Eastern North America and about 50 times less than that in highly seismic areas such as California, Japan, Taiwan or the Philippines. A seismic hazard analysis showed that the return period for a large magnitude, $M = 7$ or greater event, within 100 km of Hong Kong, is greater than 3,500 years. For a moderate size event, with magnitude $M = 6$ or greater, within 100 km of Hong Kong, the

return period reduces to between 400 to 800 years (Free et al., 2004). However, the bridge is still instrumented to record the response during seismic events to allow rapid post-earthquake assessment.

The real-time data can be used to assist with inspection and maintenance by immediately highlighting anomalies that could indicate a fault (for example oil-pressure out of range on hydraulic buffers) or else by tracking long term changes in bridge behavior (for example gradual increase of effective friction coefficient on bearings).

6.1 *System architecture*

The WASHMS system is designed with four basic levels corresponding to different operational functions:

1. Data Collection Level
   System collects data from sensors and forwards to pre-processing.
2. Data Pre-Processing and Transmission
   Data Acquisition Units (DAU's) distributed through the bridge pre-process the data prior to transmission to central processor (signal conditioning and conversion of analogue data to digital). As well as the fixed DAU's, there is also a system of Portable Data Acquisition Systems which can be used in conjunction with removable accelerometers for specific field vibration measurements.
3. Data Processing and Analysis Level
   Collection, processing, analysis, display, archiving and storage of all data by a centralized Data Processing and Control System.
4. Structural Health Evaluation Level
   Analysis and interpretation of measured data, comparison with criteria for inspection and maintenance, display archive and storage of analyzed or interpreted results, production of structural health evaluation reports.

The range of sensors provided at the Data Collection Level is comprehensive (see Table 2). Analysis and interpretation of the data will require specialist software which is being developed to be compatible with the existing TMCA-WASHMS.

Table 2. WASHMS Data collection devices.

Device	Notes
Anenometers	at tower top and deck level
Seisometers	triaxial accelerometers at each tower base
Air temperature	shaded and unshaded
Structural temperature	steel in deck and tower, concrete in deck, tower and piers, stay cables
Surfacing temperature	outside running lanes
Relative humidity	outside and inside deck and towers
Weather station	barometric pressure and rain gauge
Corrosion measurement	ingress rate of chlorides etc. into concrete
Traffic measurement	weigh in motion system (WIM)
Digital video	routine traffic videos and specific pictures of HGVs and overloaded vehicles detected by WIM system
Accelerometers	triaxial on deck and tower – both fixed and removable
Stay cable accelerometers	on all cables to calculate vibration frequency and hence cable force
Hydraulic buffers	oil pressure, stroke position and oil temperature
Lateral bearings	force, stroke position and effective friction on sliding partners
GPS	displacement measurement of main span deck and tower tops (10 mm–20 mm accuracy with base reference station)
Tiltmeters	displacement measurements – on towers
Strain gauges	to measure local fatigue stresses and calculate member forces (includes embedded gauges in concrete members)

7 CONCLUSION

The appearance of bridges is becoming increasingly important for bridge owners and the conceptual design is often selected through a design competition. The increased focus on innovative and aesthetically pleasing structures has led to beautiful bridges but also to structures, which are technically more challenging to operate and maintain. Stonecutters Bridge is an example of such a bridge, which makes a dramatic architectural statement whilst at the same time building beyond the current world record span.

The development of the design has sought to make the operation of this complex structure as simple as possible whilst maintaining those aesthetic features of the competition winning design that the client held sacrosanct.

ACKNOWLEDGEMENTS

This paper is submitted with the permission of the Highways Department, the Government of the Hong Kong Special Administrative Region.

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Chapter 2

Second bridge over the Orinoco – design, construction and operation

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ABSTRACT: A new bridge built over the Orinoco river illustrates unique design features, e. g. combined rail-roadway traffic, two back to back cable-stayed bridges, independent ship impact protection, a delta pier as fixed point and an inspection and a special maintenance program compared to Venezuela's typical operation of infrastructure.

1 INTRODUCTION

With 2560 km length the Orinoco River is the third biggest river in South America after the Amazon and the Paraná. The average discharge of 30.000 m^3/sec ranks him 5th in the world. Whereas other rivers, e. g. the Rhine has over 100 bridges, the Orinoco had only one fixed crossing at Ciudad Bolivar. The new bridge 200 km downstream is part of a 170 km long combined road and railway connection between Ciudad Guayana and a deepsea port at the north coast of Venezuela. It will tap the resources of oil-slate between the Orinoco and the Caribbean. It connects the provinces Anzoatzgui in the south and Monagas north of the river. The intention is to continue the road south into Brazil. The whole contract includes 4 km of bridges and 165 km of roadway with three interchanges as connections to the existing net of highways.

The railway line will not be activated during the first years as the railway net has to be extended first to reach this crossing. Later, this connection allows to reduce dredging of the lower Orinoco, downwards from Ciudad Guayana as goods can be transferred to the deepsea ports.

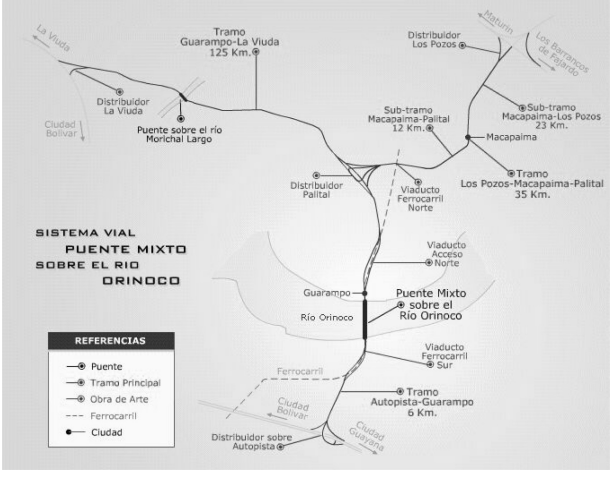

Figure 1. Plan.

2 RIVER CROSSING

2.1 *Concept*

Initial feasibility studies for a crossing in the area of Ciudad Guayana date back to 1996 with detailed examinations of various alternative locations. 30 years later contractors could qualify for a DBOT project to provide an offer, based on these preliminary studies. This tendering was suspended and an export credit between Brazil and Venezuela allowed in 2000 to start the project again with a contract awarded to the Brazilian firm Norberto Odebrecht S.A., Sao Paulo, Brazil. A new location about 20 km upstream from Ciudad Guayana has been selected.

The 3156 m long bridge is straight in plan and reaches the highest point in elevation with slopes of 0.25% at the south and 1% at the north side. The bridge is supposed to carry 4 lanes as well as 1 railway track in the center between the two carriageways. At this location, with an island in the center of the river two clearances of 260 × 41 m had to be provided for navigation channels north and south of the island. The river traffic consists mainly of barges for the transport of bauxite. The substructure of the bridge is not designed for potential impact of those barges as is anticipated to protect the foundations later by separate protection structures.

The south-north running bridge could be founded in the south on rock. On the north side the rock is overlain by sand layers up to 90 m which are partially prone to scour. The water level of the bridge varies regularly between 0 and +12.5 m with high water in August and low levels in March.

2.2 *Structure*

The length of the bridge is divided in

- the south approach with 4 sections of 300 and 360 m, a total of 1320 m
- the main bridge, a back to back cable-stayed bridge with 1200 m and
- the north approach composed of 2 sections of 300 and 336 m.

Approaches as well as the main bridge have similar cross-sections, composed of a single cell steel box under the railway line with cantilevering cross girders supported by diagonals at 3 m distance and an edge girder to support the roadway. The deck consists of a concrete slab in full composite

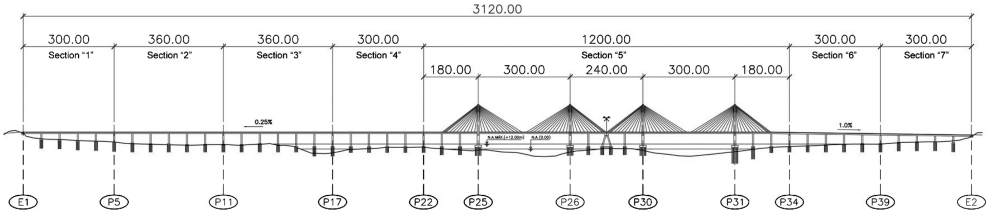

Figure 2. Layout.

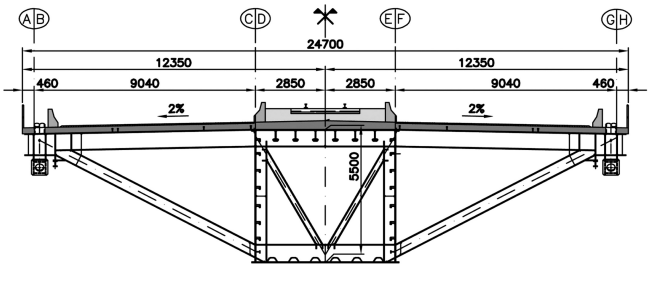

Figure 3. Cross-section.

action with the steel box and steel grid of cross girders and the edge girders. For the suspended deck the edge girders are strengthened to carry the cable forces and distribute the vertical component to 2 cross girders. Most likely this configuration is one of the biggest composite bridge for combined rail and roadway traffic.

The south approach consists of 4 sections with five to six spans, in total 22 spans of 60 m. At the north side 11 spans of regularly 60 m and one of 36 m are divided in two portions of 300 and 336 m.

As mentioned before the main bridge consists of 2 cable-stayed bridges with spans of $3 \times 60 - 300 - 4 \times 60 - 300 - 3 \times 60$ m. Except for both end spans a length of 1080 m is suspended by cables at 12 m distance. The $2 \times 8 \times 11 = 176$ cables are arranged in a semi-fan shape. Breaking and earthquake loads are directed to a delta shaped pier in the center where both cable-stayed bridges are touching each other. The transfer of forces at this fixed point takes advantage of the highest rock elevation at the island and the possibility to transfer as well the uplift forces from the main spans via a monolithical connection between super and substructure. The steel box is connected via 24 tendons of 37 strands and the shear transferred by 44 m long 150 mm deep shear keys.

For the approach structures a different concept was chosen to deal with seismic and breaking loads. All bearings are elastomeric bearings with guiding elements for service conditions. Between approach sections lock-up devices are installed to distribute longitudinal loads over many piers.

Under seismic loadings the guiding structures will break and allow the superstructure to move partially isolated with a long period and thus avoiding high seismic loadings. The joints between main bridge and approaches and at the abutments allow for the movements of the 3 independent sections of the bridge.

Except for the south side with raft foundations most piers are founded on piles with pile caps. Those have typically a diameter of 2 m and lengths between 13 and 60 m. The bearing capacity is developed in rock by tip loading. At the north side where scour and a deep sand layers require a different pile concept based on skin friction the diameter had been increased to 2.5 m for piles up to 83 m length. All piles are placed with permanent casing and the heavy reinforcement reaches up to 3%. The pile caps of the towers reach the size of $31 \times 52.5 \times 6$ m, those of the Δ-pier $30 \times 47 \times 4.5$ m. At high water level the pile caps of the towers are visible, all other pile caps remain submerged. At low water levels the pile caps and up to 7.5 m free pile length can be seen. The variable water level will be a challenge for the future protection structures. Piles as well as the pile caps must be protected against impact under all site conditions.

All regular piers with heights between 15 and 42 m have a constant section of 3×7 m with a wall thicknesses of 0.6 and 0.7 m.

The towers have the typical H-shape with slightly inclined legs and 2 cross-girders. With a total height of 120 m the legs are variable in one direction between 4 and 7.5 m and constant in the other direction. Cross beams are of hollow section as well with post tensioning in the webs. The top of the towers with the cable anchorage zone was designed as a composite section with a heavy steel case at the inside of the concrete box. This steel section carries all the bearing structures for the cable anchorages.

The Freyssinet cable system made up of individually protected strands within a PE-pipe have sizes between 42 and 72 strands. In the tower was placed the live end and at the deck the dead end. The isotensioning method allows to install the cables without the help of a crane and strand by strand. The longer cables are equipped with internal hydraulic dampers made up by a bladder filled with oil at the lower anchorage. In addition the two longest cables of each harp receive special external dampers similar to shock absorbers of a car.

2.3 *Design basics*

The basic codes applied are (AASHTO-LRFD 1998) and (AREMA 2002). Those were complimented by the recommendations for stay cable design of PTI, (COVENIN 2003-89) for the local wind loads and a local code for seismic actions which is structured similar to AASHTO.

Train loads were given as TREN COOPER E72 and the truck as HS20-44. Less than 10 trains will pass the bridge per day, however these are except for the unloaded ones equal to the design

load intensity. Exceptional load cases considered are 1 severed cable under full service loading and seismic acceleration up to 0.20 g.

All sections were sized according to permissible stresses and for the stability checks of the plates of the box section verifications according to German codes such as (DIN 4114), (DastRi012 1978) and (DIN 18800 1996) were provided.

In the fatigue analysis a service life of 80 years is considered. Not considered was that at least for 10 years the bridge will be in service without train service. The combination of rail and roadway traffic on one structure is not unusual in South America, however, the US codes AREMA and AASHTO do not reflect such a combination. Some of the European codes define these issues, however, not to much detail as it is an uncommon bridge type in Europe as well.

As the shape of the superstructure is carried on from the approaches the aerodynamic behavior is not optimal. Due to the relatively short span of 300 m and the huge stiffness of the section aerodynamic stability is fulfilled. However, vortex shedding could be a problem under certain circumstances and as a consequence the two longest cables could be excited by parametric excitation. Therefore, special damping devices were installed at those cables.

Generally weathering steel is used to avoid any additional corrosion protection. The concrete deck is supplied with 4 cm of asphalt without any sealing between concrete and asphalt.

Ship traffic in the river is low and consists of typical barge tow configurations of 3 × 5 barges. A future separation of the navigational channels in north and south side will increase the safety of the passage. The bridge will be protected in the future by independent protection structures.

3 PROCUREMENT AND CONSTRUCTION

Almost 6 of 12 months of a year, piling and pile cap construction had to be interrupted due to high water levels in the river. More than 50% of the pile caps are located in the river and 70% of the piles had to be executed in the water.

The mobilization of huge pontoons, which are generally used in coastal areas characterized this site. The execution of 370 piles with a total length of 13.100 m took 1½ years. By the help of GPS systems the piles could be placed with tolerances in the range of centimeters.

The bottom scaffolding of the pile caps consisted of lost precast panels and the pour of the cap was done in two or three stages. Temperature management was important due to generally high outside temperatures and the mass of concrete.

All piers and towers were built with gliding formwork. For the piers with constant section and without special embedded elements or connections this approach was very efficient. For the tower construction the gliding had to be interrupted two times to place the cross girders and to install the system again. Due to concentrations of rebar and the variable section the expected efficiency and quality could not be achieved. A non-technical but important aspect for the preference of the gliding system was the uninterrupted working process with the requirement for 24 h work. Otherwise the strong unions in Venezuela would not have accepted to work in 3 shifts without such a technical condition.

All steel came from Brazil. Preassembled portions of the segments were shipped from Belo Horizonte to Ciudad Guayana. In several shops in Ciudad Guayana the 12 m long segments were assembled and transported to the site at 16 km distance. In total 263 of these segments, 12 × 23 m in plan, had to be transported by flat beds over a partially closed highway.

Behind both abutments an assembling yard was prepared to assemble in 3 lines approach sections of 300 and 360 m length, as well as a portion of 210 m for the main bridge. Each bridge section was provided with steel noses at the front and at the end and was slid by means of teflon plates into its final position. The other partner of the sliding system was just the greased bottom flange of the steel box.

The center of the main bridge (120 m to both sides at the delta pier) was erected by the help of auxiliary trusses and partially by sliding to its final position.

Both main spans were erected by the free cantilevering method with 4 derricks in use.

Figure 4. Erection of backspans.

Figure 5. Free cantilevering erection.

To speed up erection always two segments of 12 m were connected to a 24 m lift avoiding 50% welding during the cantilevering phase. Pouring of the deck, however, is done in 12 m segments as the corresponding cables have to be installed after 12 m.

Construction will terminate in August with span closures in June 2006.

4 OPERATION

4.1 *General*

Traditionally in this country maintenance is not an important issue. Quite often nothing is done until major defects appear and then the repair work will still take a long time to come.

Sometimes the structures are completely run down until the safety of the users are at risk. A very prominent case was recently the failure of a major arch bridge on the highway between Caracas city and the airport. Over many years it was known that the abutments were moving. Once the repair had started it was too late and the breakdown of the structure interrupted for months one of the most important highways in the country.

In this contract, a design build contract, the construction company has to guarantee for the structure over a long period of time. So at least for this period it is not only the owners interest to maintain the structure well but also the interest of the contractor to see the client treating his investment according to the rules of operation. May be it is not more than hope that politics have changed regarding inspection and maintenance and it could be that this project becomes the test for such a novelty approach in Venezuela.

Most likely the supervision of the traffic and a camera controlled area will be provided for the general safety of the users and for the whole investment.

4.2 *Basic inspection and maintenance concept*

An Inspection and Maintenance Manual with detailed instructions is provided by the contractor/designer and handed over to the client. This will be adjusted and further developed to the specific needs of the structure and the behavior over time. The Manual is influenced by (DIN 1076 1999) "Infrastructure for roads and highways, surveillance and inspection", the standards of inspection (RI-EBW-PRÜF 1998) "Guidelines for standardized registration, processing and analysis of the results of inspection in accordance with DIN 1076, 1998", special inspection and maintenance instructions from suppliers of cables, bearing, joints etc. and the designer's long-standing experience with cable-stayed and railway bridges.

Main topics of the manual are

- Documentation of the structure (as built and handing over situation)
- Inspection intervals, detail of inspection
- Inspection planning and evaluation of results regarding
  - structural safety
  - users safety
  - durability
  - rating and expected future degradation
- Activities following inspection results
- Maintenance, routine and occasional
- Rehabilitation
- Extraordinary situations

The idea is to provide for the client guidebook to establish reasonable procedures to control the structures in every aspect, to register the situation over the lifetime and to be able to initiate the necessary maintenance activities. An optimum resource allocation and a high level of reliability over many years should be the final result.

4.3 *Possible complimentary systems under consideration*

An extensive permanent monitoring program is under consideration. The driving issue is to control the area, the traffic and the whole new red of roads by cameras and via an operation center. Attached to this system of pure operational and safety control in regard to the users a permanent

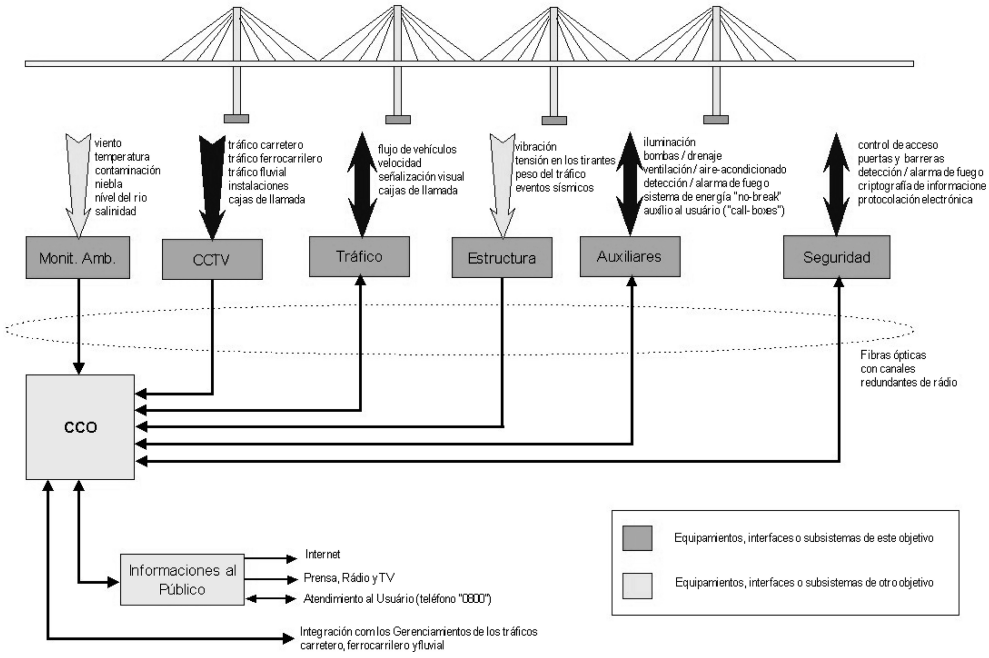

Figure 6. Interfaces of the integrated supervision and control of the structure.

structure control could be provided and registered and supervised in the operation center. The architecture of such an integrated system is the following:

CCO	Centro de Control Operacional (operation control center)
SRT	Subsistema de Recaudación de Tasas (toll facilities)
SMA	Subsistema de Monitoreo medio Ambiental (recollection of ambient datas, temperature, wind, …)
SME	Subsistema de Monitoreo de la Estructura (recollection and processing of datas from sensors in the structure)
SMV	Subsistema de Monitoreo Visual (control by cameras)
SSCTC	Subsistema de Supervisión y Control del Tráfico ferro Carrilero (control of railway traffic)
SSCTF	Subsistema de Supervisión y Control del Tráfico Fluvial (control of river traffic)
SSCU	Subsistema de Supervisión y Control de Utilidades (control of utilities)
SSFP	Subsistema de Seguridad Física y Patrimonial (general control of all structures and installations)
STDFV	Subsistema de Transmisión de Datos, Fonía y Vídeo (transmission system to CCO)

If the permanent operation control will be implemented the permanent structural control would not be a major issue. The personnel of the operation center would supervise as well the features of the structural control (SME). An evaluation of the datas accumulated by the SME system, however, will need to be done by others. Today it is no difficulty to provide via internet those data to

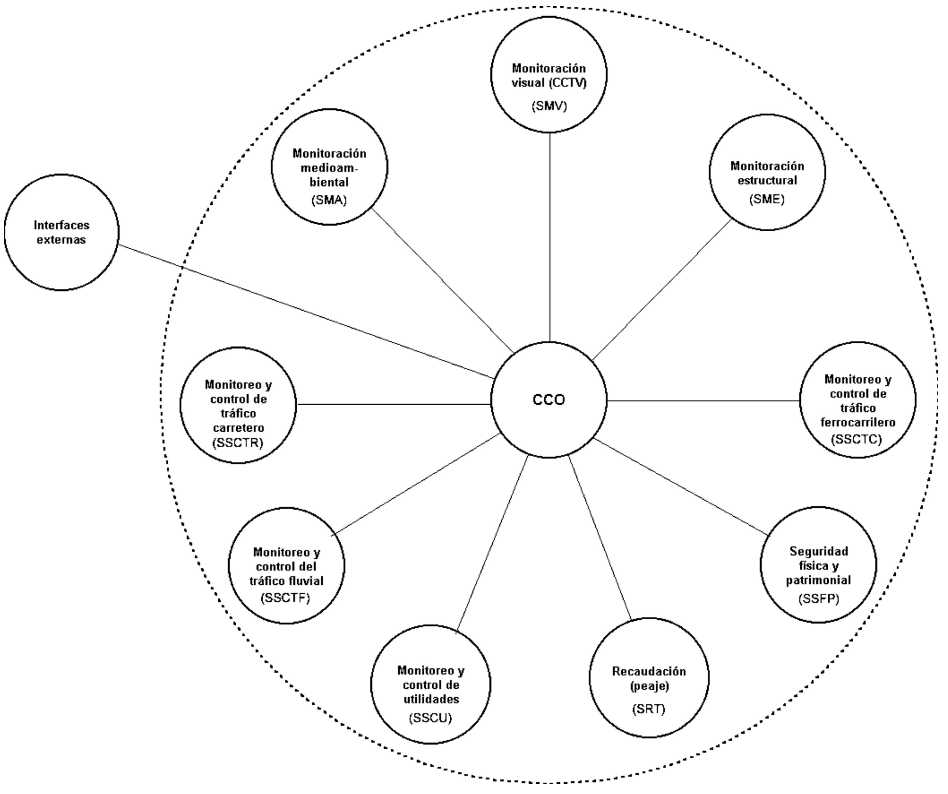

Figure 7. Subsystems of the integrated supervision and control system of the structure.

the engineer's office where these can be processed. The personal on site in the operation center will deal with maintaining the system of control and supervising the traffic.

At this stage, however, it is impossible to tell which way the owner decides to go.

5 CONCLUSION

An important link has been built over the Orinoco which will improve the possibilities for development of the whole region. This construction project created many jobs in the area and fostered export of steel from Brazil.

To guarantee the preservation of this important infrastructure over 80 years of service, with the highest reliability, owner and builder have to implement a system of maintenance. The first steps had been done. Hopefully this will become a positive example, to show how to manage other infrastructures in the country as well.

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Main contractor:	Constructora Norberto Odebrecht
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Joints, bearings, cables, post-tensioning:	Freyssinet

Consultant for detailed design:	Consorcio Brave (Lustgarten & Asociados for the main bridge; Figueiredo Ferraz for the approach structures)
Consultant and official checking engineer for Consorcio Brave:	Leonhardt, Andrä und Partner GmbH, Beratende Ingenieure VBI
Erection engineering for CNO:	Leonhardt, Andrä und Partner GmbH, Beratende Ingenieure VBI

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Chapter 3

A construction solution for self anchored suspension bridges

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ABSTRACT: There is a need to develop a cost-effective construction method for building a self-anchored suspension bridge, since this is becoming the new structure type for long span crossings. The challenge is for engineers and contractors to develop such a system that will be cost effective and very constructible. Currently, the methods considered for this structure type are to either build the bridge on falsework, and then erect the main cables and self anchor them, or to build the bridge as a cable-stayed bridge and removing the stays once the main cables are in place, and to self anchor it. This alternative provides a third option which limits the necessity to build two complete bridges prior to erecting and self anchoring the suspension cables.

1 INTRODUCTION

The San Francisco Oakland Bay Bridge, Figure 1, designed as a self-anchored suspension bridge, offers a beautiful solution to the new crossing of the San Francisco Bay between Yerba Buena Island and the Oakland East Bay Skyway. This project has been bid successfully, with construction now beginning.

Also, the Konohana Bridge in Japan, Figure 2, and the Yeong Jong Bridge in Korea were built as self-anchored suspension bridges. It seems that the big challenge with this type of structure is how to build it economically. A cost effective solution to this challenge is sought by the engineering community. A quest for innovation in construction methods should ultimately bring the correct answer.

The two most prominent options that have been considered as a construction method are:

1. To construct a temporary cable-stayed solution that would be used to construct the superstructure using temporary stays, followed by the erection of the suspension cables, which would be self-anchored to the already constructed superstructure. The temporary stay system would then be removed. The temporary stays and the suspension cables would use the same tower.

Figure 1. San Francisco Oakland Bay Bridge.

Figure 2. Konohana Bridge.

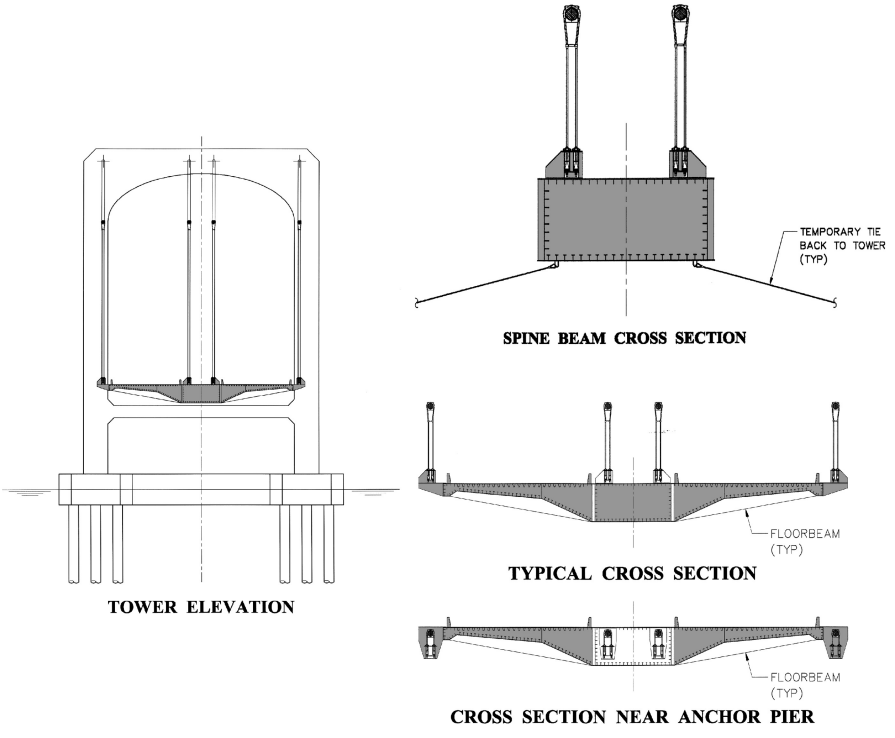

Figure 3. Cross sections.

2. To construct the entire superstructure on falsework, then to erect the suspension cables, followed by connecting the cables to the superstructure, self-anchoring such.

We suggest a third option be considered that has many advantages, including important economic advantages. The cross sections of this option are shown in Figure 3. The main features for this third option are as follows:

1. Start with a four suspension cable cross section. Two suspension cables will work but four cables seem more compatible with the concept. This offers the opportunity to improve the factor of

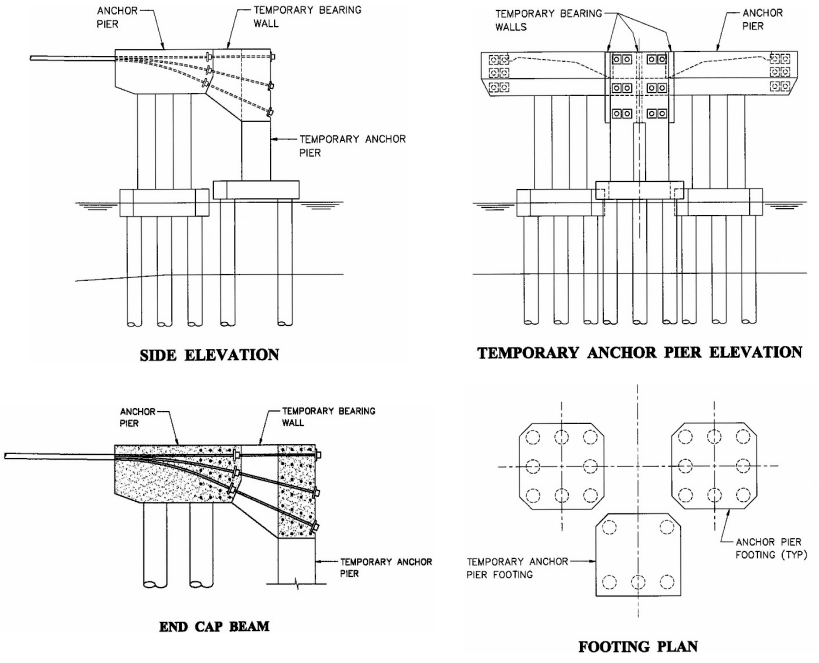

Figure 4. Temporary anchorage details.

safety in the cables and possibly the use of four smaller cables would reduce the overall cost of the cable portion of the structure. Also, the four cables offer the positive feature of better redundancy.

2. Using a supplemental temporary fixed anchor system (details shown in Figure 4) at each end of the bridge, construct the two interior cables of the four cable system. This temporary anchor system will line up with the two interior cables.
3. Initially, construct the center portion of the superstructure, which is approximately one-third of the superstructure, using the two temporarily anchored interior cables. This one-third of the superstructure would consist of a central spine beam. It would be possible to use a reduced factor of safety for the fixed anchor system based on the temporary conditions, reducing the cost of the temporary anchor system significantly. In addition, the small amount of superstructure supported by this fixed anchor system further greatly reduces the requirements once the spine beam is supported by the two interior cables self anchor the two interior cables and remove the temporary anchor system.
4. The final two exterior cables would be constructed with self-anchored connections at each end of the bridge. The self-anchored connection would tie into a large transversely post-tensioned concrete cap end beam constructed at each end of the bridge on top of the end pier.
5. Cantilever floorbeams would extend from the spine beam and the exterior cable suspenders would connect to the tips of the floorbeams. All four cables would connect at some point along the centerline of the floorbeams using suspender cables.
6. The remaining deck, assumed to be orthotropic steel decking, would be connected to the floorbeams and the spine beams. Finalizing the roadway surface, barrier, lighting, and other amenities would complete the structure.

By eliminating the need to construct two complete support systems, it is clear that economics should be accomplished. The completed structure would be the self-anchored suspension bridge with clean and open aesthetics.

Figure 5 shows an example with two 1,000-ft spans, which we have used to illustrate the concept that we believe is viable for construction purposes.

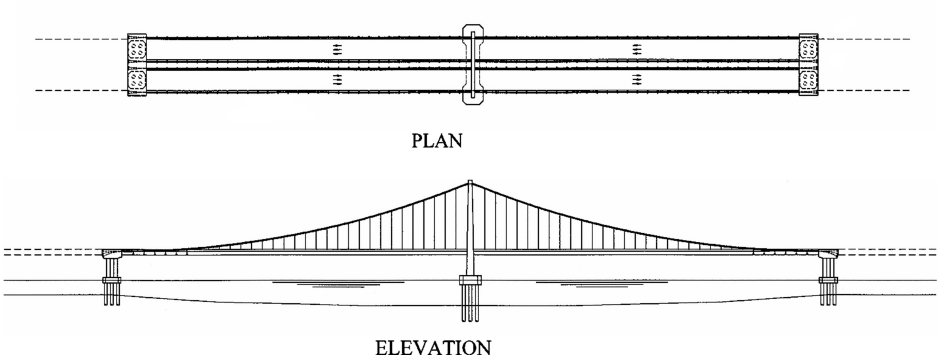

Figure 5. Plan and elevation.

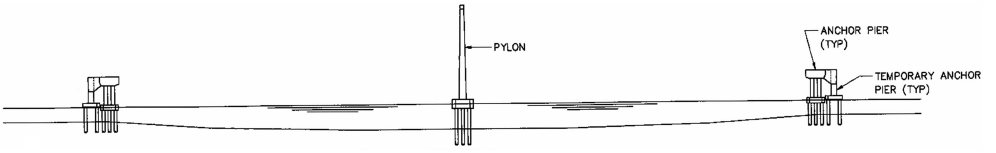

Figure 6. Construction – stage 1.

2 CONSTRUCTION SEQUENCE

The construction sequence is the heart of this presentation. This is the construction sequence:

2.1 *Construct pylon, end piers, temporary supplemental anchor piers (attached to end piers at top) – figure 6*

The construction methods for the pylon, the end piers and the supplemental anchor piers would use traditional methods of construction. It is anticipated that the foundations be founded upon large diameter drilled shafts, probably 8′-0″ in diameter or greater. These shafts are advantageous since they offer tremendous lateral strength due to the large moment of inertia generated by the diameter raised to the fourth power. It is anticipated that the end piers would have substantial lateral strength in and of themselves and they would be detailed as such. The end piers then would be connected by upper braces to a supplemental anchor pier, which would provide substantial combined lateral resistance, adequate to accommodate approximately 25% to 35% of the total dead load anchor force with a factor of safety somewhere in the order of 2.0. This factor of safety can be this low for temporary conditions. As mentioned, we anticipate construction of temporary concrete walls between the temporary supplemental pier and the end piers in order to cause both piers to act together in lateral strength. Also, the temporary supplemental anchor piers will probably utilize tension piles on the back pile row in order to provide a greater stability against overturning. Post-tensioned anchors in the back piles would offer considerable tensile restraint.

Consideration might be given to using something other than concrete for the temporary supplemental anchor pier. However, at this point, due to its mass, it seems reasonable to use concrete allowing for a suitable demolition method to remove the concrete.

2.2 *Erect two interior cables connecting to the supplemental anchor piers. Use seven-wire 0.6″ ϕ strands to make up the cable. Set anchors in temporary cap beam. Split the cable into six sub-cables for anchoring purposes – figure 7*

The next step would be to install the two interior cables using seven wire 0.6″ ϕ strands for the cables. Each strand would be laid out in its appropriate position and the entire group of strands

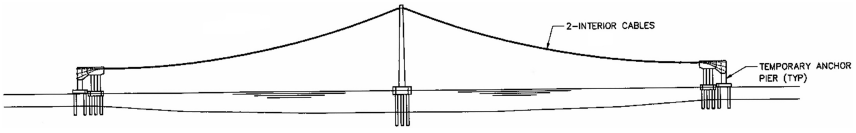

Figure 7. Construction – stage 2.

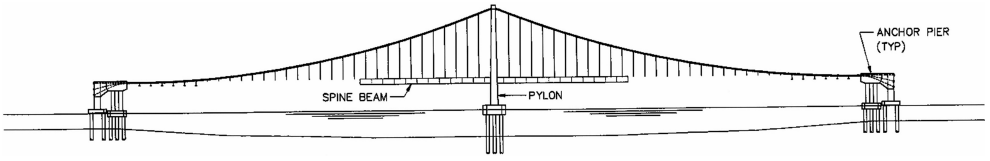

Figure 8. Construction – stage 3.

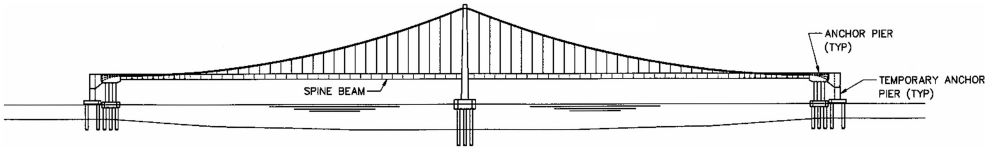

Figure 9. Construction – stage 4.

would be compacted together into one cable. This allows use of traditional post-tensioning anchor technnology, which should be more compatible with the self-anchoring procedure. The cable in the very large end cap beam area can be split into sub-cables for anchoring purposes. The number of sub-cables could vary according to the size of the anchors available. However, for this example, we are suggesting six sub-cables. Each of these cables would be anchored in the temporary cap beam with anchors prepared for future set in the permanent cap beam, but not set until the time when the cable force must be transferred into the permanent cap beam.

2.3 *Erect superstructure spine beam on two interior cables. Erection direction is from center pylon each way towards end piers – figure 8*

The superstructure as proposed is essentially a central spine beam with intermediate floorbeams that extend out as cantilevers from the spine beam. The floor beams could be box sections, with a dimension of 10-ft in the longitudinal direction. This would be set into 30-ft suspender panels. Therefore, the floorbeams would also be set on 30-ft centers. In this stage, we expect just the spine beam to be erected. The bulk of the longitudinal strength for the structure would be in the central spine beam. It is anticipated that the superstructure erection for the spine beam would be done in a manner that starts at the central pylon and progresses out each way from the tower, towards each end pier.

2.4 *Transfer all interior cable loads to permanent end cap by setting permanent cap anchors and by slowly releasing the cable free from the temporary end cap in a reasonable sequence. Rejack force into all suspenders – figure 9*

After the central spine beam is entirely complete and in place, and the forces adjusted in to the suspenders, the cable loads are to be transferred from the temporary end cap to the permanent end cap by setting the permanent cap anchors and slowly releasing the strands from the temporary cap. This should be done in a sequence to reduce eccentricity and to provide a provision for gradual release of the force into the permanent end cap beam.

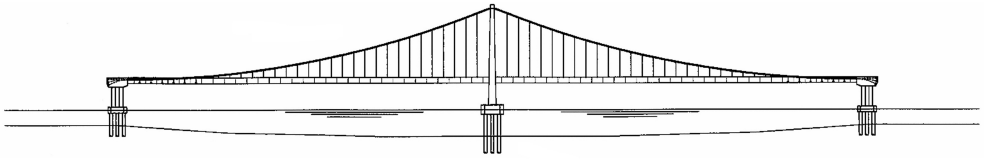

Figure 10. Construction – stage 5.

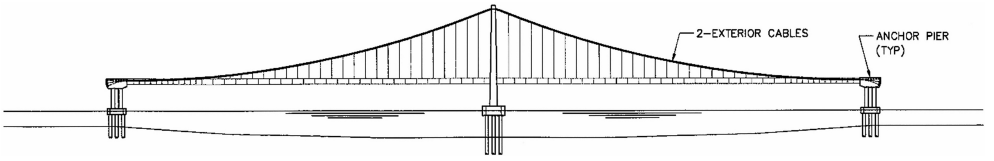

Figure 11. Construction – stage 6.

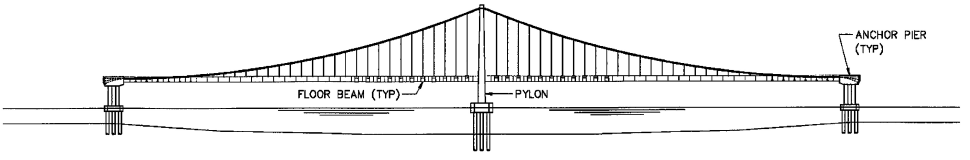

Figure 12. Construction – stage 7.

2.5 *Remove temporary supplemental anchor pier – figure 10*

To remove the supplemental anchor pier, some initial mechanism should be installed into the pier, which might enable it to easily be taken apart in pieces. Some appropriate layering in the detailing might allow for such a procedure.

2.6 *Erect exterior cables self anchored to the stiff concrete end cap beam – figure 11*

The exterior cables are then erected and self anchored in the stiff concrete end cap beam. The erection of these cables and the details for these cables would be identical to the interior cables.

2.7 *Erect exterior floor beams – connect to exterior cables with suspenders – figure 12*

At the same time as the exterior cables are being erected, the exterior floorbeams could be erected and connected to the spine beam with a bolted connection. The floorbeams should be erected in a consistent manner to balance the load on the central spine beam so that excessive torsion is avoided. Once the floorbeams are in place, the exterior cables would be connected with the suspenders and initial forces jacked into the suspenders.

2.8 *Construct remaining superstructure between floorbeams. Connect to spine beam – figure 13*

The superstructure between the floor beams would be constructed such that the load is transferred directly into the floorbeams, but would also be rigidly connected by bolted connections to the spine beam. Therefore, the intermediate deck sections would truly be supported on three sides.

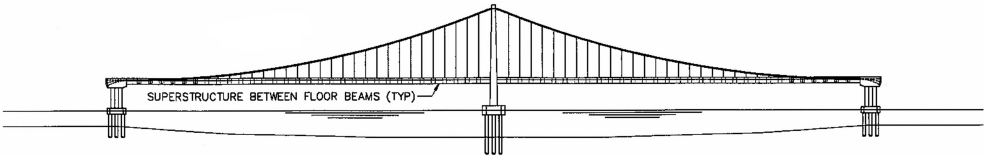

Figure 13. Construction – stage 8.

3 CONCLUSION

In order to make some rough calculations as to the potential savings of using the two supplemental temporary anchor pier and cap beam systems, we assumed an initial base cost for the sample bridge of $300 million ($US). The alternate construction methods would increase that cost by the following approximate amounts:

- In actuality, this 1,000-ft span is probably too long a span for a cable-stay possibility, the tower is short, which creates a flat angle for this alternative. This further increases the demand for the stay forces. It appears that the cable stayed alternative for such a span arrangement as suggested here, would increase the cost by something in the order of $120 million.
- For the falsework system, considering the necessity of deep water support foundation over the full length of the 2,000-ft crossing, and also assuming that there is a need to span the necessary navigational areas for the major structural elements, we estimate that it would take $80 million to build the foundation and associated structural work to accommodate such an effort.
- For installing the two supplemental piers, foundations, and cap beams, and for removing such, it appears that the approximate cost would be about $40 million. This includes the tension anchor system within the back shafts.

Certainly some innovation in any one of the techniques could alter the prices considerably. However, this presents a serious initial thought as to what might be considered as a way of making the self anchored system price competitive.

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Chapter 4

Mitigation and monitoring of cable vibrations for cable stayed bridges

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ABSTRACT: As cable stayed bridges become more common, designers and owners must determine how to monitor and mitigate cable vibration. This paper will discuss one case study in detail in which dampers were installed on a bridge already in service. Beginning with documentation of the cable vibration, the chosen solution will be discussed and monitoring techniques for verifying the effectiveness of the solution will be presented.

1 INTRODUCTION

1.1 *Bridge description*

1.1.1 *General*

The East Huntington Bridge is a 3,787-foot long bridge that carries West Virginia Route 106 across the Ohio River from East Huntington, West Virginia to Proctorville, Ohio. The cable stayed superstructure was designed by Arvid Grant and Associates, Inc. It was constructed from 1982 to 1985, and was opened to traffic in August, 1985. The States of West Virginia and Ohio are joint owners of the bridge and share inspection and maintenance responsibilities in accordance with their Agreement.

The central portion of the river crossing consists of an asymmetrical concrete cable-stayed two span bridge, as shown in Figures 1 through 4, with a concrete tower supporting the roadway and deck structure with 31 pairs of cables. The lengths of the cable-stayed spans are 900′-0″ and 608′-0″. The concrete tower extends approximately 284 feet above the roadway, which consists of two 12′-0″ traffic lanes and 3′-0″ shoulders.

1.1.2 *Cable system*

The bridge possesses two planes of cable stays; each plane contains 31 cables. Thus, the bridge has a total of 62 cable stays. There are no intermediate supports or connections to any cable, and they are held in position by their tension load.

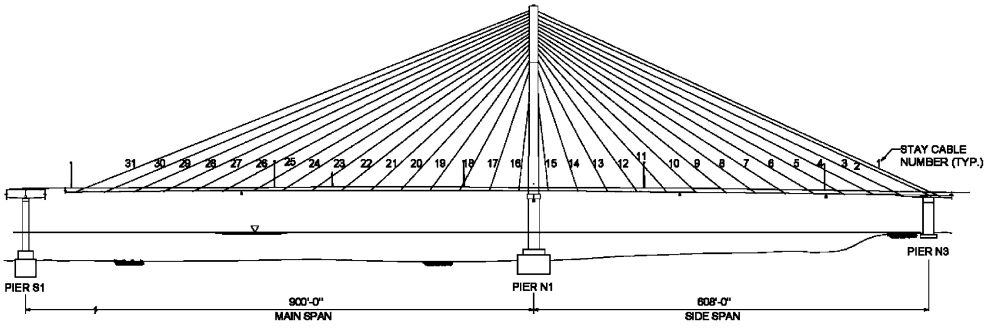

Figure 1. East elevation of main bridge.

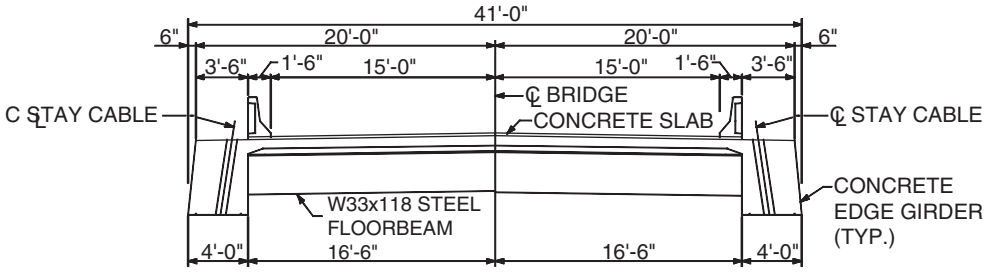

Figure 2. Cross section of main bridge.

Figure 3. Upstream (east) elevation of the main bridge.

Figure 4. Downstream (west) elevation of the main bridge.

In a typical cross-section of the cable, as shown in Figure 5, the central portion is a bundle of 1/4-inch diameter steel wires which carries the load. This bundle of wires is wound with a helix of steel wire strand and is fitted into a plastic tubing with walls approximately 1/4-inch thick. The diameter of the plastic tubing ranges from approximately 4 inches to 7 inches. Following installation of the cables on the bridge, the space between the steel wires and the plastic tubing was filled with cement grout. The steel wires are held by HiAm sockets which anchor the cable to the structure, both in the tower and at deck level.

The concrete tower has the shape of an inverted "Y" with hollow sloping legs. The cables are attached to the vertical portion of the tower above the legs. This upper vertical portion has an H-shaped cross-section with a web or core wall six feet thick and four small outstanding flanges which support concrete fascia panels. Steel anchor pipes are welded to steel bearing plates at their upper ends and are embedded in the core wall. The cable socket rests on the bearing plate, and the steel anchor pipe guides the cable through the core wall. The pipes project beyond the core almost to the fascia panels, which have openings through which the cables pass. These details are shown in Figure 6.

1.2 *Cable movement*

The cables are attached to the bridge at their ends and are free-standing, with some sag, over considerable lengths. The length of the longest cable is approximately 760 feet. Normal movement of

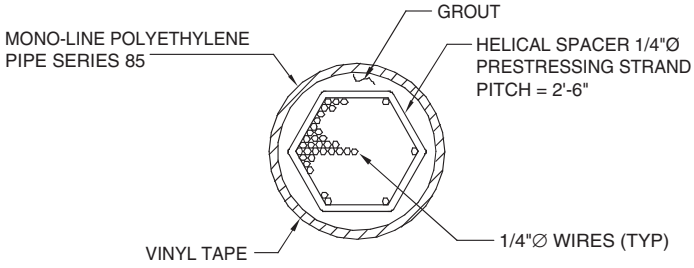

Figure 5. Typical cross section of cable.

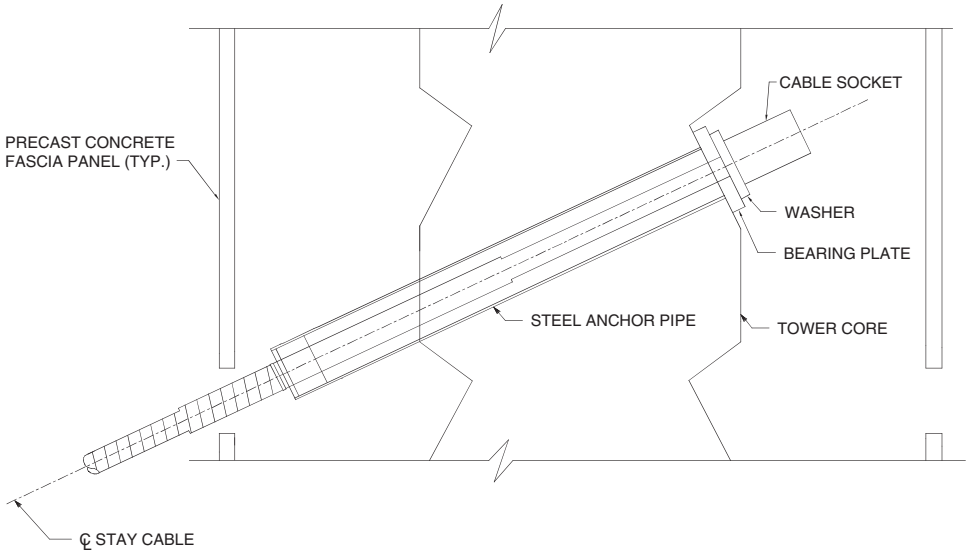

Figure 6. Cable anchorage details at tower head.

the bridge due to traffic causes some very small displacement of the cables. A momentary heavy gust of wind may also cause some movement of the cables. However, a more serious and threatening type of cable movement has occurred on this and other similar cable stayed bridges.

During periods of moderate, but steady winds and light rain or drizzle, the movement of one or more cables may develop and increase to alarming proportions. One such occurrence was witnessed on the East Huntington Bridge. The cable movement was such that it prompted the following description by a passing motorist:

"One of the cables on the East Huntington Bridge was shaking like it was not attached to anything."

This phenomenon has been termed rain-wind induced cable vibrations. In addition to the danger to the motoring public from the general distraction and unnerving sight of such a condition, large and violent cable movements risk the danger of damage to the cable itself, as it may strike some other object and damage its protective covering. The excessive movement may also cause unexpected stresses in the cables or may crack the cable's grout, thus rendering the steel wires susceptible to moisture infiltration and corrosion. Furthermore, the cable anchorages may be damaged when large cable vibrations occur.

As the use of cable stayed bridges has increased recently, the number of these types of episodes and experiences with rain-wind induced excessive vibrations has also increased. As design experience with cable stayed bridges increases, the overwhelming opinion concerning rain-wind induced vibrations is that they should be mitigated.

2 VIBRATIONS

2.1 *Cable vibration*

A tensioned cable, such as a guitar string or a cable stay, vibrates readily after an applied displacement or impulse load, and most often this vibration is a combination of different types and forms of waves. The two basic types of waves in a cable are "moving waves" and "standing waves."

The moving wave in a long taut cable is produced by a rapid whip-like movement at one end which happens too quickly for the entire cable to follow into equilibrium position. As a result, a short wave is produced in a generally straight cable and this wave moves quickly along the cable and is reflected back from the far end. The period is the time it takes the wave to move to the far end and be reflected back. Eventually the waves decay and subside due to energy absorption, usually the result of a combination of air and structural friction.

For the most part, interest in bridge cable vibrations are most concerned with the other type of vibration – the standing wave. Standing waves can most easily build up or accumulate energy and amount of movement. When vibrating due to a standing wave, virtually every portion of the entire length of the cable is in motion, moving back and forth past its normal position in a regular and repeating pattern, what, for most practical purposes, can be treated as a sine-wave. Whereas a swing and a pendulum have a simple and predictable single pattern of vibration, standing waves can cause a cable to vibrate in a number of different patterns or modes, and each mode has its specific characteristic shape and period:

- The first, or fundamental, mode occurs when the entire cable swings back and forth in a single curve with the maximum displacement at the center. This mode has the longest period.
- The second mode is a pattern shaped like an "S" with the midpoint of the cable stationery and a single loop on both sides – when one half of the cable is at maximum displacement upwards, the opposite half is at maximum displacement downwards. The period of this mode is one-half the period of the first mode.
- The third mode is a pattern with three equal and independent loops in the length of the cable. The period of this mode is one-third of the period of the first mode.

And the list of modes continues. Each mode has a unique shape and a specific frequency and characteristics.

To complicate matters, the cable vibration at any instant may be any combination of the different modes, and each mode in that combination may have a different amount of movement or energy. For example, at one time a cable may have this combination of vibrations:

– mode 1, at 2000 ft-lbf/sec energy, with
– mode 2, at 1000 ft-lbf/sec energy, and
– mode 3, at 1400 ft-lbf/sec energy,

and at a different time, the following combination of vibrations:

– mode 1, at 1200 ft-lbf/sec energy, with
– mode 2, at 2200 ft-lbf/sec energy, and
– mode 3, at 700 ft-lbf/sec energy.

Therefore, an infinite number of combinations of modes and their energies are possible for a cable's vibration due to standing waves. One major challenge in measuring and analyzing cable vibration is due to this fact that a number of modes occur at the same time. The analysis must be able to identify and isolate each mode and track its behavior.

2.2 *Rain-wind induced cable vibrations*

It stands to reason that a steady wind blowing on cable stays will impose a steady force on, and thus a constant deflection of, the cables. Similarly, a single, sudden gust of heavy wind will cause some momentary deflection or vibration in the cables. However, contrary to reason, the general experience is that cable stay vibrations build up and grow to the most violent movements under the combined effect of rain and wind, with both being light to moderate in their intensities.

Note: There are various phenomena, not all of which are well understood, which may also be responsible for the cable vibrations. Vortex shedding is one such phenomenon. A moving column of air traveling along the cable is another possibility. Indeed, in some instances, these cable vibrations are present when there is no precipitation.

The fact that uniform wind causes vibration to build up in a cable must mean that a steady wind does not always move evenly and uniformly past the cable. Since air and wind can only be seen under special and controlled conditions, it is generally not possible to see the unevenness.

Imagine a clothesline, with some sag, in a light rain. As the water strikes the line, some of it adheres to the line due to water's cohesion and surface tension. As the water accumulates, a water rivulet will flow down the line to the low point until some critical quantity has accumulated, causing a drop of water to fall from the line. It is generally accepted that it is this formation of a water rivulet on a cable stay that initiates the aerodynamic instability of the stay. This instability is considered to be a unique phenomenon, different from the instability caused by vortex shedding or wake galloping.

Certain combinations of wind and rain produce water rivulets on the upper surface of the stay. For this to occur, a water rivulet on the stay is forced to the stay's upper surface by wind blowing both in the proper direction and at the proper speed. These wind direction and speed combinations create a force on the rivulet which overcomes both gravity and friction on the stay's surface, and pushes the rivulet to the upper surface of the stay. The presence of the rivulet in this particular position then detrimentally changes the stay's cross section subject to the flow of the wind and creates lift on the stay. This lift then causes aerodynamic instability of the stay, characterized by low frequency, large amplitude rain-wind induced oscillations. These cable vibrations are discussed in more detail by (Yokoyama et al), (Hikami and Shiraishi), the (Research Committee for Cable Aerodynamics of Cable-Stayed Bridges) and (Poston).

2.3 *Controlling cable movement*

A number of different methods have been considered and used on bridges to control aerodynamic instability problems in cable stays, and these include:

- Use of neoprene cushioning pads between the cables and the ends of the anchor pipes. These have been used in a number of bridges, and were installed on the East Huntington Bridge in the

tower anchor pipes, and also in the anchor pipes at deck level. The damping ability of this system is more limited than others, but in some instances these appear to be able to prevent the cable vibration by absorbing energy as the cable moves. These neoprene dampers, however, are intended to suppress high frequency, low amplitude oscillations.

- Cross-tying the cables together. In this method, steel wire ropes are used at approximately right angles to cross-tie the cables to each other. This method serves to reduce the effective cable length, and as a result, increases the natural frequency of the cable. With a higher natural frequency, a higher wind velocity is required to initiate instability of the cable. Additionally, cross-tying increases the damping of the cable stays, reducing cable oscillations.
- Modifying the cable shape to change the aerodynamic conditions. In some cases the plastic tube surrounding and protecting the cable is not a smooth circular shape, but is fabricated to contain protrusions or ridges along the length of the cable. These ridges disrupt the formation and flow of water rivulets on the cable in an effort to eliminate rain-wind induced oscillations.
- Installation of energy absorbers on the cables to restrain the oscillations of the cables. These dampers are placed so that, as the cable moves, it causes movement of the damper and activates it. The damper resists the movement and absorbs energy from the cable during each movement, thus decreasing the cable's movement. This approach essentially increases the naturally occurring damping of the cable stays.

For the East Huntington Bridge, a field study extending over a period of several years indicated that a linear, hydraulic damper satisfactorily reduced the movement of a test cable. Based upon that study, dampers of appropriate design were installed on the remaining stay cables in the Fall of 2000. Each East (upstream) and West (downstream) cable, with the exception of Cable Nos. 14, 15, 16, and 17, was retrofitted with a single external damper.

2.4 *Cable dampers*

The cable dampers are linear, fluid viscous mechanical devices, similar to automobile shock absorbers, attached to the cables in the protected tower head behind the fascia panel. The dampers are double-acting, i.e., the dampers absorb linear motion in two directions – both through elongation and shortening of the damper. As a cable moves, it causes the damper to move, and the damper's resistance to the cable's movement absorbs the cable's energy.

2.4.1 *Damper operation*

The dampers are mounted to the cables in the tower head as shown in Figures 7 through 9. One end of the damper is anchored to the stationary steel anchor pipe with a fabricated steel bracket assembly, and the other end is attached to the cable with a fabricated steel cable collar.

When a damper is installed on a bridge cable, each movement of the cable will move the piston rod in or out of the damper body. However, each movement will be resisted by the damper; that is, each time the cable moves, the damper will work to resist its movement. The damper's forces are always against the cable's movement, because the forces are caused and timed by the cable's movement itself.

So, over a period of time, while the weather conditions may be creating small impulses to cause the cable's movement to build up, the damper is creating opposing impulses to cause the cable's movement to stop. While the dampers are effective for motion along only their longitudinal axis, which is aligned in the vertical plane of the cable, the field studies performed for the design of the dampers revealed that the predominate motion of the cables is in the vertical plane. Additionally, the field studies revealed that damping of cable motion in the vertical plane also disrupts cable movement in other planes.

2.4.2 *Damper movement*

The location of the steel anchor pipe in the tower head provides a convenient and sheltered place on which to mount the cable damper assemblies; however, this places the dampers relatively close

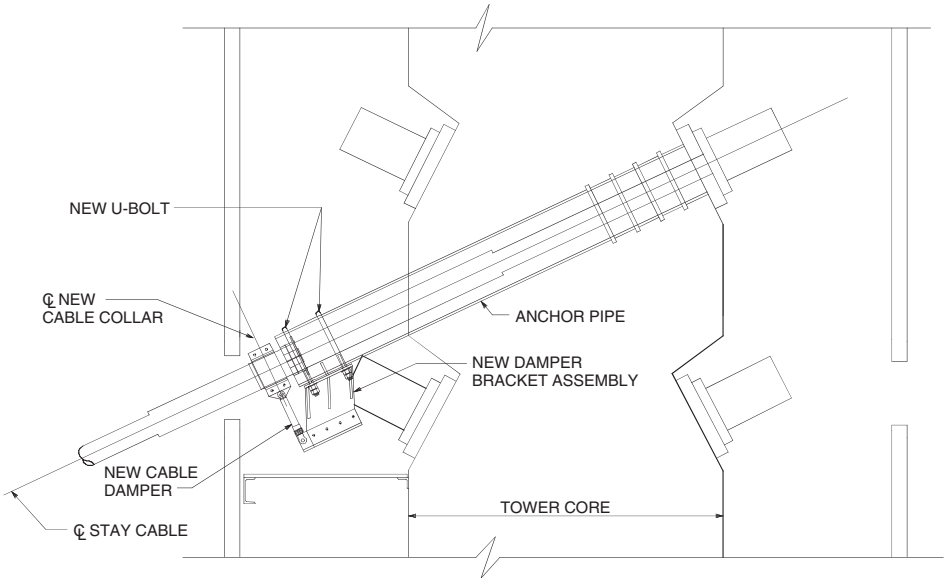

Figure 7. Detail of cable damper in tower head.

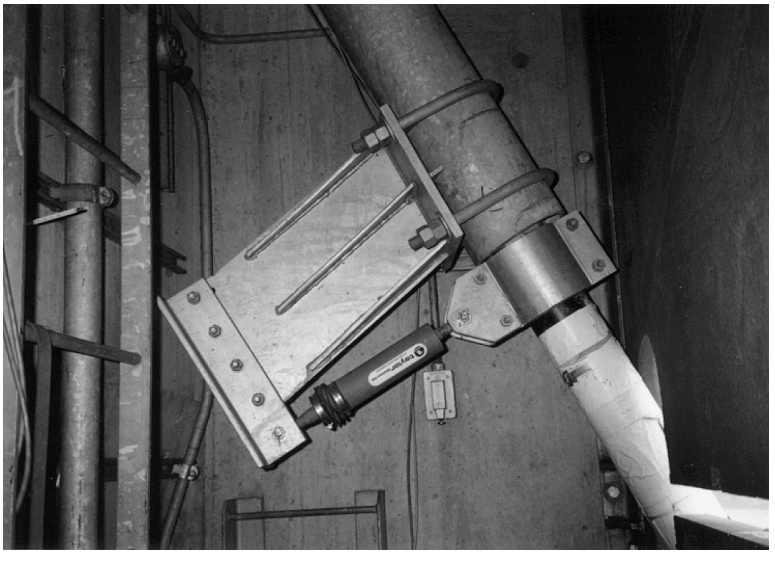

Figure 8. Typical type A cable damper.

to the end of the cables. Even though the neoprene cushioning pads were removed from the ends of the anchor pipes in the tower, the cable's movement at this location is not very large.

For one of the long sloping stays, the total cable length is approximately 760 feet. The cable damper is attached approximately 9.8 feet from the bearing socket, which does not move with the cable vibration. When this cable vibrates in the first mode (a single bow curve) with a deflection of three inches at the center (rather large) the deflection at the damper would tend to be less than 1/8-inch. The intent of the dampers is to control the cable movement so that the deflection does not become excessive. This means that the amount of cable movement at the damper is quite small.

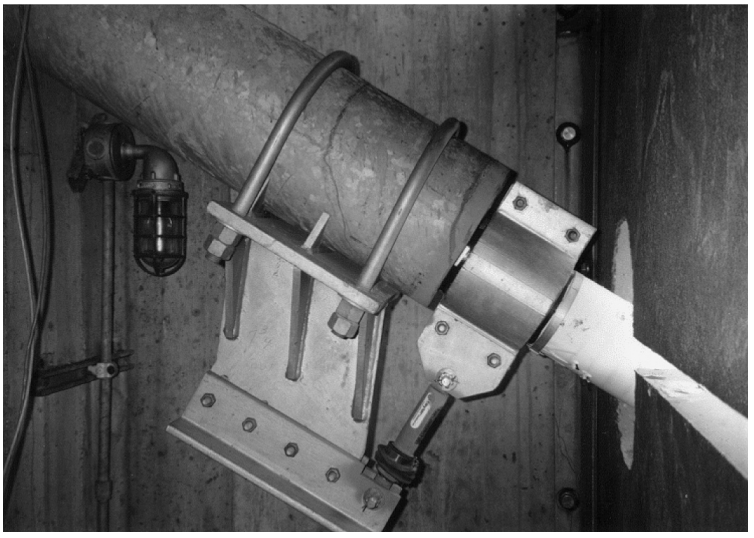

Figure 9. Typical type B cable damper.

Therefore, for the dampers to be fully effective, it is essential that the entire damper assembly and connections have very small clearances in the order of thousands of an inch. The design plans for the dampers required very tight tolerances on all parts in order to eliminate "loose play" within the damping system. While the cable movement at the damper activates the unit such that it absorbs the cable's energy, the small amount of cable movement at the damper illustrates the need for a tight assembly.

3 MAINTENANCE AND TESTING OF CABLE DAMPERS

The dampers are structural-mechanical units utilized to reduce the large amplitude rain-wind induced cable oscillations. They are intended to be a permanent part of the bridge and cable system, and as such, will require periodic inspection, maintenance and confirmation of their proper operation. The "operating portion" of the dampers is on the interior, and is sealed from access, which prevents direct visual examination. There are maintenance steps, however, that can be taken to ensure proper performance of the damper assembly. The damper maintenance will require several distinct phases of work:

1. Site inspection of each damper assembly,
2. Evaluation of each damper's effectiveness and, if necessary,
3. Replacement of defective dampers.

3.1 *Site inspection of damper assemblies*

Each damper assembly on each cable should be inspected periodically. This includes inspection of the damper itself, the cable collar and the bracket assembly attached to the steel anchor pipe. Examination of the structural elements should follow the usual bridge inspection procedures for structural inspections. Additionally, the damper assembly inspection should examine for the following:

- Tightness of the Assembly
- Length of the Damper

- Alignment of the Assembly
- Visual Assessment of Damper

A detailed inspection of the installed damper assemblies was completed in October, 2002 immediately after installation, using prepared inspection data sheets. A record of the inspection measurements can be used as a benchmark for measurements of future inspections.

3.2 *Inspection cycle*

Currently, the East Huntington Bridge is inspected annually to varying degrees. An in-depth Periodic Inspection of the entire structure is performed biennially. Between the biennial inspections, "walk-through" Interim Bridge Inspections are performed on the main cable stayed spans. Additionally, the tension in the cable stays has been evaluated by the vibration method in approximately four-year intervals.

The detailed examination of the dampers, complete with measurements, should occur every four years, coinciding with a Periodic Inspection and the evaluation of the tension in the cable stays. The data sheets for the dampers should be completed and compared to the previous sheets in order to assess any changes in the damper assemblies.

3.3 *Evaluation of damper's effectiveness*

Each damper's operating condition should be evaluated on a regular basis to confirm that it is providing the intended cable protection. The operating condition can be measured and evaluated by field testing of each cable's vibration characteristics.

This paper includes procedures for field testing each cable's vibration characteristics, analyzing the test data, and recording the vibration characteristics of each cable. This field measurement method is an indirect assessment of the cable dampers, but is much more efficient than removing each damper and returning it to the manufacturer for measurement of its damping characteristics. The method is based on the fact that each damper changes its cable's vibration characteristics. When the damper no longer changes the cable's vibration characteristic by a satisfactory amount, it is not adequately performing its intended function.

The field measurement method presented is an efficient procedure for determining and confirming the operating characteristics of the dampers on a periodic basis. This method consists of forcing each cable to vibrate. These movements are measured with accelerometers, and a data acquisition system is used to record the accelerations and to examine the pattern of the cable's vibration. The vibration data is then processed mathematically using a Fast Fourier Transformation. This processing ultimately produces the damping exponent and percentage of critical damping of each cable. The dampers can then be evaluated by comparing each cable's calculated percentage of critical damping to two baseline conditions: (1) without a damper installed, and (2) just after the damper was installed.

The procedure for field testing and data reduction will be described in this section. It is important that the same procedures and methods be used in subsequent field tests in order to minimize differences in the data acquisition and processing. The procedure described herein was used to determine the initial damping characteristics, and in order to provide a consistent basis for comparison, future procedures should follow similar steps.

3.3.1 *Field test procedure*

Taking these measurements at the bridge site requires a crew of two or more people, one or two pairs of accelerometers and wire leads attached to mounting frames, a computer, a power source, and some pull rope. The tests have been run with the bridge closed to avoid interference from traffic vibration. Working at night with the bridge closed is generally most convenient. Each cable is tested individually, and the field test procedure for the entire bridge is typically scheduled for a minimum of two nights. The second night is for any contingencies; to date vibration testing on this bridge has always been completed in one night.

The principal direction of movement of the cables is in the vertical plane, but two accelerometers have been used on the cable – one to measure the vibration in the vertical plane, and the other to measure any horizontal vibration. The accelerometers are about the size of a wrist watch and it is convenient to mount them and their wire leads securely to a light frame or wooden block which must be firmly attached to the cable.

Output from an accelerometer is a continuous analog signal, and the strength of the signal varies with the acceleration. This signal is excellent for viewing on a screen or graph, but since the results will be mathematically analyzed, it is necessary to store the results in numerical form. The data acquisition system "samples" the continuous signal at a predetermined rate (typically 200 times a second), and evaluates the strength of the signal numerically, so that digital results are recorded and stored.

With the accelerometers and wiring complete and in place for a cable, the computer screen should be viewed to determine that the circuits are functioning satisfactorily. The cable is then set into motion by a person pulling on the rope working at deck level. The best results are obtained under the following conditions:

- a soft rope is maneuvered to a relatively high point on the cable; and
- the rope is given a series of tugs carefully timed to coordinate with the natural frequency of the cable.

  Note: The cable should be monitored carefully during this excitation with the rope to ensure that the vibrations do not increase beyond a safe amplitude. On this bridge, the excitation of the cables should not produce a mid-point half-peak amplitude exceeding approximately six inches, i.e., the cable movement should be limited to one-foot of peak-to-peak movement at the cable midpoint.
- The cable is then allowed to vibrate freely without additional energy input (no tugging on the rope).

The readings from the accelerometers are recorded for about two minutes, as shown in Figure 10, and during this interval, the data is observed to verify that the readings appear to be normal. A minimum of two additional sets of readings are then taken from the same cable, and if the vibrations on the computer screen appear to be satisfactory on all three sets of readings, the field measurement of that cable is complete. The operations are then moved to the next cable. If any data sets are not satisfactory, the procedure must be repeated until three sets of satisfactory data are obtained.

Earlier discussions noted that any cable may vibrate in more than one mode. In its simplest terms, the number of the mode is the same as the number of loops in the cable – mode 1 is a single loop

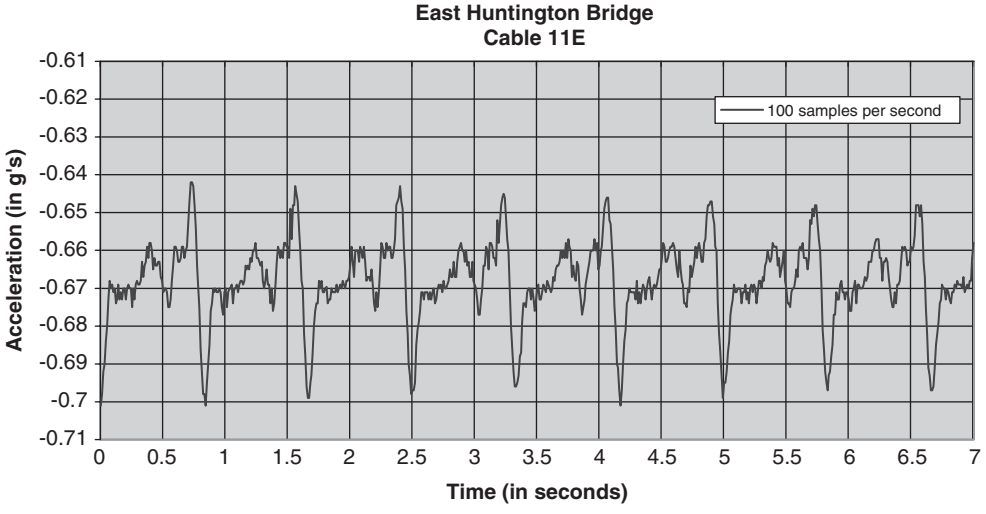

Figure 10. Sample accelerometer data for cable 11 – east.

between end supports; mode 2 has two equal loops (the mid-point is stationary). In obtaining field data for calculating the damping of each cable, every effort is made to excite the cable into the first mode due to the fact that this is the most easily established and controlled mode.

In controlling cable vibration, the dampers have different degrees of effectiveness against each mode of vibration; that is, the cable damper may tend to reduce one mode more quickly than another. The damper design was not intended to be optimal for the first mode, but rather to be effective for a range of vibration modes. Therefore, the measurements of the cable damping do not necessarily determine the most damping that is being provided for any mode, but rather, they determine the damping for the first mode. Given that this mode is most easily and reliably established in the field, the acquired data provides the most accurate means for comparison of the dampers' effectiveness.

After the field testing and measurement is complete, the recorded data is further processed and analyzed. This phase of the procedure takes somewhat longer than the field data acquisition.

3.3.2 *Analysis of field results*

Proper analysis of the field vibration measurements will yield the coefficient of decay for evaluation of the dampers. The coefficient of decay is based on comparing the energy of vibration in the cable when it is first set into motion to its energy at later periods of time.

The complications to t his process include the following:

- When set in motion, a cable will generally vibrate in a number of modes simultaneously and each mode may have a different energy level;
- Each frequency for the cable has its own coefficient of decay;
- The accelerometer measurements are taken at one location on the cable and the numerical recordings are a combined result of all these different vibrations at this one location on the cable.

The primary aim of the analysis is to determine the coefficient of decay for the fundamental, or first mode frequency, as this appears to be the most reliable indicator for this bridge.

The basic procedure for working with this data is to perform a Fast Fourier Transform analysis. This analysis identifies the principal frequencies present, and the relative motion or energy associated with each frequency, as shown in Figure 11.

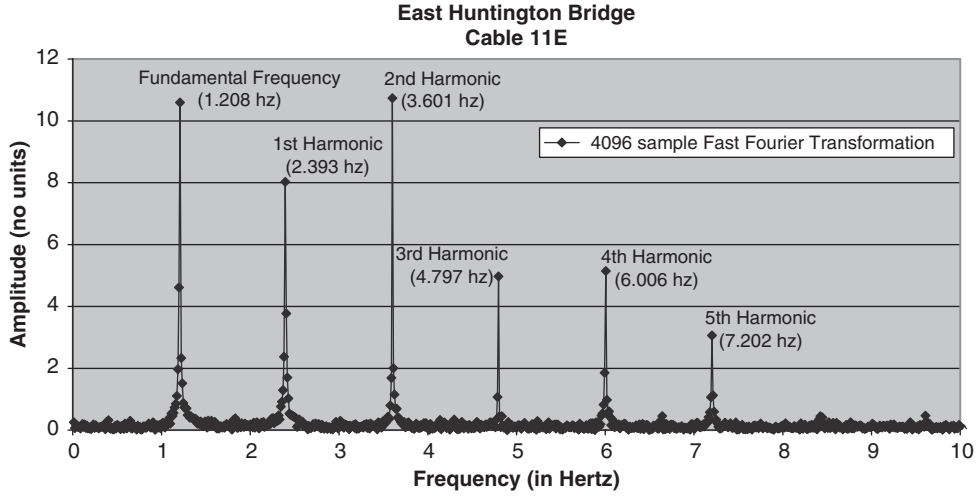

Figure 11. Frequencies for cable 11 – east.

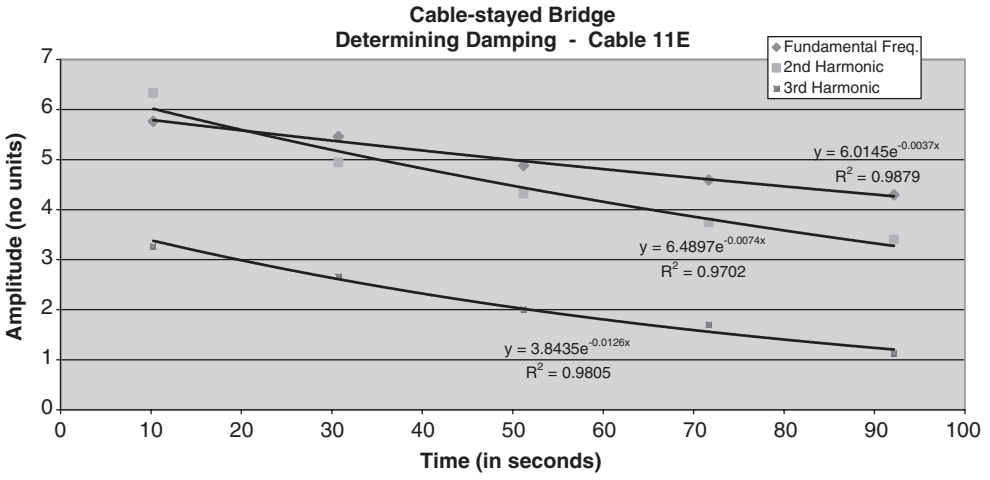

Figure 12. Determining damping coefficients for cable 11 east.

It is necessary to make a transform analysis at a minimum of five different times during a cable's recorded movement:

- the first immediately following the initial disturbance, when the amount of cable motion should be the greatest;
- the final movement near the end of recognizable movement from the initial disturbance; and,
- a minimum of three intermediate analyses that do not overlap with the two times listed above.

The first step in making the analyses for a cable is to examine each set of readings. This is done by displaying the numerical results for each cable. Avoiding any sets of data which have visually obvious anomalies may help in eliminating confusing results caused by an error due to a loose wire connection or other unexpected cause. Three trials of readings should be analyzed for each cable, but comparisons for decay always should be made between values from the same trial, see Figure 12. Damping exponents should be used from only those trials that have a minimum Correlation Coefficient (R^2 value) of 0.8, and preferably 0.9, for a minimum of five data points for the cable. The final result for the damping exponent of the cable is taken as the average of the damping coefficients for the three test trials for that cable. Figure 12 shows the damping coefficients for several frequencies for one trial of a cable.

3.3.3 *Reference cable vibration data*

Reference data has been obtained for the damping exponent and the percentage of critical damping of each stay cable, both without dampers installed and just after the dampers were installed. This information provides the means for comparison of future test results.

Figure 13 shows the change in the damping exponent from the no damper state to after the dampers are installed.

3.3.4 *Identification of unsatisfactory dampers*

If the damper ceases to provide a certain level of damping in the cable, it should be replaced. When the percentage of critical damping in the cable falls below the average of the before-damper and after-damper values (i.e., the damper is 50% effective), the damper is no longer providing satisfactory performance. The minimum acceptable percentage of critical damping in each cable has been calculated accordingly. Should the test data indicate that a percentage of critical damping falls below the listed minimum value, it is recommended that the subject cable be retested to confirm the results. The retest should be performed in the same manner as the original test. The percentage of critical damping of the retest (an average of 3 trials) and the original unsatisfactory test (an average

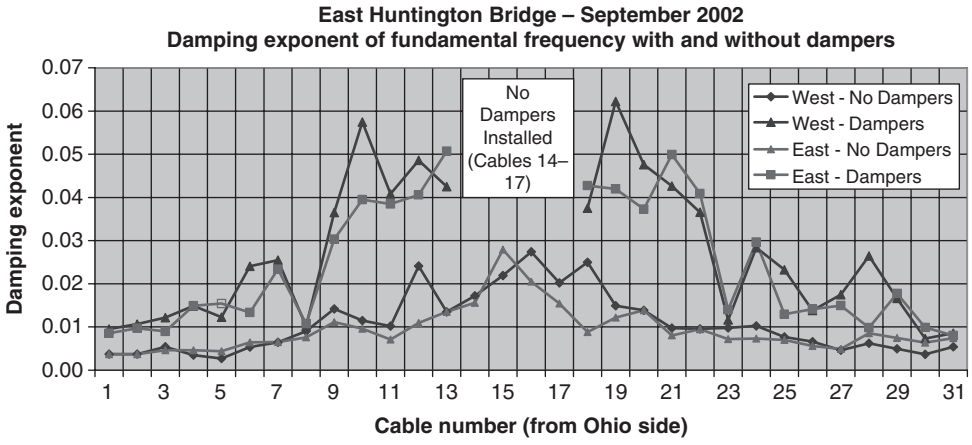

Figure 13. Change in damping exponent of fundamental frequency due to damper installation.

of 3 trials) should be averaged and then compared to the limit. If this average result falls below the limit, the damper should be replaced. However, the damper is considered to be satisfactory if the average result exceeds the limit.

In several instances, the field tested damping value of a cable in either the before-damper or after-damper condition was higher than expected when compared to the damping of the same cable on the opposite side of the roadway (e.g. the damping of cable 6 west vs. 6 east in the after-damper condition). In these cases, engineering judgment was exercised to define a reasonable minimum acceptable percentage of critical damping. This occurred at cables 6 west, 10 west, 12 west, 19 west and 28 west.

3.4 *Replacement of defective dampers*

Dampers which have been identified as providing unsatisfactory protection to the cable, as described in the previous section, must be replaced. Replacement in-kind is recommended, as it is important to match the original damping properties and a number of detailing dimensions for the damper assembly.

The hydraulic damping characteristics of any replacement dampers must match the characteristics of the original dampers. This includes both the damping constant, and the capability to carry the design force. These properties of the installed dampers are provided in Table 1. The "damper constant" is a measure of the stiffness of the damper. For a damper with a damper constant of 2000 LBS/(FT/SEC), the piston rod can be moved the full 2-inch stroke length in about 6.7 seconds with a steady 50 pound force. A 100 pound force would take only half as long to move the piston rod the same distance. A damper with a damping constant of 20,000 LBS/(FT/SEC) would require approximately 67 seconds for a 50 pound force to move the piston rod 2-inches.

The damper bracket assembly was designed and fabricated to specific dimensions based upon the length and end connection details of the original dampers. The connection angles of the bracket assembly are connected to the bracket web with high-strength bolts through slotted holes in the bracket web. These slotted holes allow for adjustment of the length of the damper. The piston rod of replacement dampers should be set at mid-stroke when installed, and the mid-stroke length must be such that it fits into the existing connection details.

3.5 *Special notes*

The results of the field tests performed on the cables in September 2002, following the installation of the hydraulic dampers, indicate that the dampers did indeed increase the level of damping in all of the cables. In several instances, specifically, cables 8 west, 23 west, 28 east and 31 east, the increase

Table 1. Installed cable damper parameters.

Cable damper parameters						
Cable	Damper type	Damper model	Damper part number	Damper constant (lbs/ft/s)	Damping force (lb)	Damping velocity (in/s)
1 (E &W)	Type A	2.5 × 2	67DP-16607-01	13500	1077	0.96
2 (E &W)	Type A	2.5 × 2	67DP-16606-01	15500	1457	1.13
3 (E &W)	Type A	2.5 × 2	67DP-16604-01	10000	1024	1.23
4 (E &W)	Type B	1.5 × 2D	67DP-16609-01	6000	518	1.04
5 (E &W)	Type B	1.5 × 2D	67DP-16609-01	6000	633	1.27
6 (E &W)	Type B	1.5 × 2D	67DP-16610-01	3250	382	1.41
7 (E &W)	Type B	1.5 × 2D	67DP-16610-01	3250	468	1.73
8 (E &W)	Type B	1.5 × 2D	67DP-16611-01	2250	378	2.02
9 (E &W)	Type B	1.5 × 2D	67DP-16612-01	1500	315	2.52
10 (E &W)	Type B	1.5 × 2D	67DP-16613-01	900	231	3.08
11 (E &W)	Type B	1.5 × 2D	67DP-16614-01	600	200	4.00
12 (E &W)	Type B	1.5 × 2D	67DP-16615-01	350	160	5.50
13 (E &W)	Type A	2.5 × 2	67DP-16603-01	350	228	7.82
14 (E &W)			No damper installed			
15 (E &W)			No damper installed			
16 (E &W)			No damper installed			
17 (E &W)			No damper installed			
18 (E &W)	Type A	2.5 × 2	67DP-16603-01	350	254	8.69
19 (E &W)	Type B	1.5 × 2D	67DP-16615-01	350	157	5.38
20 (E &W)	Type B	1.5 × 2D	67DP-16614-01	600	180	3.60
21 (E &W)	Type B	1.5 × 2D	67DP-16613-01	900	238	3.17
22 (E &W)	Type B	1.5 × 2D	67DP-16612-01	1500	317	2.54
23 (E &W)	Type B	1.5 × 2D	67DP-16611-01	2250	391	2.09
24 (E &W)	Type B	1.5 × 2D	67DP-16610-01	3250	465	1.72
25 (E &W)	Type B	1.5 × 2D	67DP-16610-01	3250	387	1.43
26 (E &W)	Type B	1.5 × 2D	67DP-16609-01	6000	705	1.41
27 (E &W)	Type B	1.5 × 2D	67DP-16608-01	7000	689	1.18
28 (E &W)	Type B	1.5 × 2D	67DP-16608-01	8250	701	1.02
29 (E &W)	Type A	2.5 × 2	67DP-16604-01	10000	718	0.86
30 (E &W)	Type A	2.5 × 2	67DP-16605-01	12250	897	0.88
31 (E &W)	Type A	2.5 × 2	67DP-16756-01	18600	1202	0.80

was not as large as anticipated when compared to the remainder of the bridge cables. However, the damper assemblies of these cables have been thoroughly inspected, and were found to be satisfactory.

It is noted again that the field test procedure does not necessarily evaluate the dampers in the mode of vibration for which they are most effective. The dampers were designed to be effective over a range of modes, rather than optimal for the first mode of vibration. Hence, the results of the first field test included in this paper do not necessarily indicate unsatisfactory performance of the dampers, but rather, provide a benchmark for future tests.

4 CONCLUSION

Based on the experience from the East Huntington Bridge, dampers are an excellent option for mitigation of vibration of cables in cable stay bridges. Dampers provide the following benefits as a mitigation technique:

- dampers can be easily retrofitted to most existing bridges or easily incorporated into new designs;
- dampers and the required mounting assembly do not impact the aesthetics of the bridge;

- dampers require very little maintenance; and,
- damper performance can be easily quantified in a few hours of on-site work.

In addition, cable tensions can be easily determined as a byproduct of collecting the accelerometer data and provide important information to the Owner about the health of the bridge.

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Chapter 5

Forth Road Bridge – maintenance challenges

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ABSTRACT: Forth Road Bridge is a suspension bridge with a main span of 1006 metres and side spans of some 408 metres. Approach viaducts to the north and south bring the total length of the structure to just under 2500 metres. The bridge was opened in 1964 and traffic has grown significantly from 4 million to 24 million vehicles using the bridge each year. All work carried out on the bridge has to be done under a regime of overnight or limited weekend carriageway or lane closures to minimise delays to users on this important strategic and commuter route.

Following a recent internal inspection of the main cables, significant corrosion was found leading to remedial works being planned. The inspection and remedial works are described elsewhere. This paper deals with the other major projects on the bridge.

Several large projects to strengthen and improve the structure, and replace major components have already been carried out. These have included strengthening the main towers to allow for increased live loading; constructing pier defences at the main towers to withstand shipping impact; replacement of the hangers and replacement of underdeck access gantries and runway beams. Several more significant projects are planned over the next 15 year period. These will include strengthening the suspended span deck truss; replacing the main suspended span and approach viaduct deck joints; replacing the bearings on the approach viaducts and parapet replacement. In addition, major maintenance projects such as waterproofing and resurfacing the carriageways and painting the suspended span truss, the approach viaduct box girders and the main towers will also be carried out.

1 INTRODUCTION

1.1 *Background and location*

The Forth Road Bridge is a long span suspension bridge and when opened in September 1964, was the fourth longest span bridge in the world and the first long span suspension bridge to be built outside of the United States. The bridge lies at a latitude of 56° north and at the time of construction was the most northerly long span suspension bridge in the world, Figure 1.

The bridge crosses the Firth of Forth some 15 km west of Edinburgh and is a vital link in Scotland's strategic road network. Over 24 million vehicles cross the bridge each year.

1.2 *Details of the construction*

The bridge has a main span of 1006 metres and the side spans are each 408 metres long, Figure 2. The orthotropic deck on the main span is made up of a 12.7 mm stiffened steel plate overlain with approximately 38 mm thick mastic asphalt, on a waterproofing layer. The deck to the side spans is a 203 mm thick reinforced concrete slab with a similar surfacing detail to the main span. The decks on both the main and side spans are supported on steel stringer beams that span between large steel cross girders spaced at 9.144 metre centres. These cross girders are supported by two longitudinal stiffening trusses, which in turn are supported by the main cables, Figure 3.

The main cables are 600 mm nominal diameter and are each made up of 11618 no. 4.98 mm diameter galvanized wires that transfer the loads from the bridge to the main and side towers and

Figure 1. Location.

Figure 2. Looking southwest.

also down to the north and south anchorages. The anchorages are tapered rock tunnels filled with concrete and post tensioned.

Linking the stiffening trusses to the main cables are 192 sets of wire rope hangers at 18.288 metre centres the hangers vary in length from 2.5 metres at mid span to 90 metres adjacent to the main towers.

The cables at the main towers, side towers and in the anchorage chambers are seated directly on cast steel saddles. These saddles on the main tower are fixed directly onto each tower leg whilst those at the side towers and anchorages are fixed to steel rocker boxes that are free to rotate in the direction of the bridge axis.

The main towers are of steel box construction rising some 156 metres above river level and are formed from three fabricated steel boxes that are joined by cover plates to provide a five cell structure in plan. The legs of each tower are connected by cross members at the top, and just below deck level, and by diagonal stiffened box bracing above and below the deck.

Figure 3. Suspended span stiffening truss.

The approach viaducts to the bridge are themselves large steel twin box girders with a reinforced concrete deck slab. The eleven span south viaduct is 438 metres long and the six span north viaduct is 253 metres long.

The design and construction of the bridge is described in the Proceedings of the Institution of Civil Engineers.

1.3 *Funding of the construction of the bridge*

The total cost of the bridge and its approach roads was some £19.75 million. This was met by local authorities contributing £0.5million and a grant from the Secretary of State for Scotland (the representative of the UK Government) of £4.65 million. The balance of £14.60 million was borrowed from the Secretary of State. Loan repayment and running expenses were to be met by toll charges. The last repayment on the loan was made in December 1993.

1.4 *Legislation*

The Forth Road Bridge Joint Board (FRBJB) was established by an Act of the Westminster Parliament in 1947 with powers to construct and maintain the Forth Road Bridge. The Scottish Parliament used powers contained in the Transport (Scotland) Act 2001 to replace the FRBJB with the Forth Estuary Transport Authority (FETA), and the FETA Order 2002 came into effect on 1 April 2002. On 21 March 2001, the Forth Road Bridge formally joined the Scottish Minister's list of buildings of special architectural or historic interest as a Category A listed structure. This listing in effect meant that any alterations other than immediate emergency work would require listed building consent from Historic Scotland, a government Agency.

1.5 *Role of FETA*

FETA is responsible for the management, maintenance and operation of the Forth Road Bridge and may develop, support and fund schemes that it considers appropriate in order to reduce road traffic congestion on the bridge or encourage an increase in the use of public transport across the Forth.

FETA, in partnership with local authorities on each side of the Forth, and the Scottish Executive, at all times endeavors to minimise the impact of bridge maintenance programmes on users of the crossing.

2 TRAFFIC MANAGEMENT

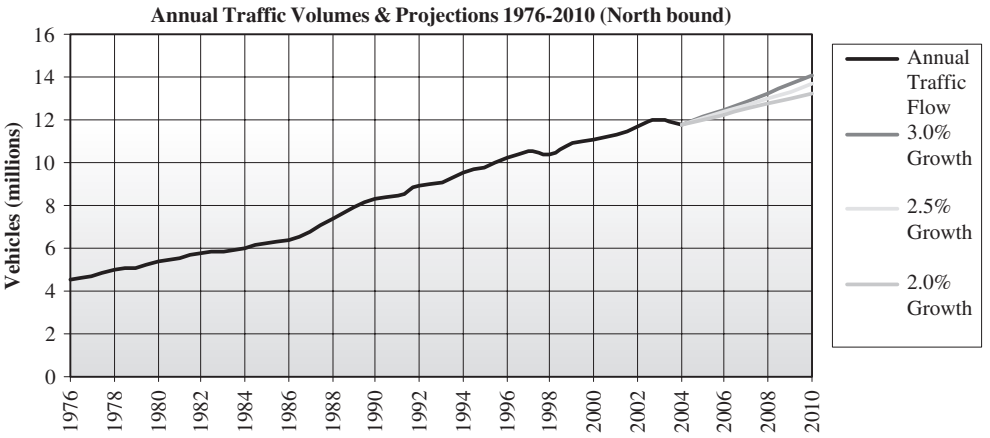
2.1 *Traffic growth*

One of the continuing problems faced by FETA is to maintain a bridge that carries over 24 million vehicles per annum or on average over 65,000 vehicles per day. The growth in traffic using the bridge has increased at a relatively steady rate and is shown in Graph 1 for northbound traffic. The split in traffic northbound and southbound is roughly 50/50, Figure 4.

As can be seen from the graph, if growth continues at the medium growth rate, by 2010 total annual traffic crossing the bridge will be around 28 million.

2.2 *Effects on works*

Because of the disruption to traffic that even a daytime lane closure causes most if not all inspection and maintenance work that requires lane or carriageway restrictions is now carried out overnight.



Graph 1. Growth in traffic.

Figure 4. Looking north – afternoon peak.

However, to meet with all the stringent requirements to ensure the safety of the workforce and the safety of drivers, traffic management can only be set out when traffic flow falls below a certain level. In addition, the carriageway must be clear in the morning before 06.00 hours to ensure that the morning peak flow is not affected. In essence, this often means that after travelling out on the bridge a contractor, or the bridge in house workforce, may only have between six and seven hours possession of the site. This makes carrying out work not only difficult but also expensive, and for some tasks and projects it is not possible to do the work required in these narrow overnight slots.

3 WEATHER AND WINTER MAINTENANCE

3.1 *Climate*

The Forth Road Bridge lies at the same latitude as Moscow. The prevailing wind is from the west (the Atlantic influence) but as the Forth flows into the North Sea there are times when this relatively cold body of water has a considerable effect on the weather locally. From April to September poor visibility, caused by a fog from the North Sea known locally as haar, can occur around the bridge despite the weather being fine and sunny just a short distance away from the Forth. This can have a considerably disruptive effect on an already relatively short painting season. In addition although the east coast of Scotland has a fairly low rainfall (some 640 mm as an annual average) the months of July and August, although warm can be very wet. It is also windy, especially during the winter months. These quirks of the weather combined with an exposed environment make maintaining and carrying out improvement works on the bridge a real challenge to the FETA staff and external contractors.

3.2 *Winter maintenance*

The de-icing material used on the bridge carriageways is glycol at a spread rate of 0.025 litres per sq. metre which is suitable for temperatures down to $-8°$ Celsius. Road salts are used by the adjacent roads authorities on the approach roads but stopped at a distance of 300 metres from the north approaches and at the toll plaza on the southside. The glycol is manufactured under the trade name Konsin and costs over £950 per tonne and around £30,000 worth of material is used each year. A combination of Ice Alert Monitors in the road surface and visual inspections trigger the application of the Konsin which is spread by FETA's operations staff. The Konsin will not prevent a heavy snow fall lying on the carriageways and a combination of ploughing, using FETA vehicles, and spreading washed grit is used to deal with compacted snow. Chlorides samples taken from the approach viaduct concrete deck slabs show low levels of chlorides in the concrete.

4 BRIDGE MAINTENANCE

4.1 *In house team*

The bridge has benefited from having both a dedicated maintenance workforce and engineering management team on site and both are considered essential to ensure that a proactive maintenance and improvement programme is funded and carried out. The total number of staff working at the bridge is over 100 and some 40 of those work on bridge maintenance. In general, FETA's in house workforce carry out all routine maintenance with the exception of servicing of specialist plant (e.g., the lifts in the main towers; cathodic protection to the pier defences; the security systems and air compressors). Minor alterations to steelwork and weld repairs are carried out in house and all temporary access and rigging is done by FETA's own rigging team. Two in house electricians carry out all but the most major electrical installations and two electronic technicians look after such diverse systems as the toll equipment, CCTV cameras, weigh in motion systems, ice alerts, the anemometer and the variable message signs. All maintenance painting of the structure is all currently carried

out by FETA staff. The workforce of four permanent painters is supplemented between April and October by between six and fifteen temporary painters.

One of the tasks that has been routinely carried out during the summer months is to wash down the steelwork of the suspended span truss with fresh water to remove surface salts and guano.

4.2 *Training*

Training of staff and ensuring that method statements and risk assessments are carried out prior to any task being undertaken, are seen as being key to ensuring that all work is carried out in as safe a manner as possible.

4.3 *Minor works*

As well as the hundreds of routine maintenance tasks carried out in house by FETA staff, the following are some of the small to medium sized projects that have been undertaken since the bridge opened:

- Replacement of a number of failed hanger holding down screwed rods
- Access walkway system to the soffit of the suspended span decks erected
- Toll collection equipment replaced
- Toll plaza capacity increased by adding new toll lanes
- New electrical ring main installed on the bridge
- Road lighting replaced
- CCTV system added
- Compressed air and water mains added
- Anchorage Chambers and Side Towers concrete walls crack sealing
- Anchorage Chambers roof waterproofing
- Toll plaza carriageway reconstruction
- Overhead sign gantries on Southern Approaches and roadside variable message system
- installed
- Central Reserve Crossovers formed
- Intruder and fire alarm systems added to viaduct box girders and main towers
- Viaduct Central Reserve barrier installed
- Administration Building concrete repairs and refurbishment
- Boat Shed constructed
- South Anchorage Building constructed
- Waterproofing Administration Building basements
- New Maintenance Building constructed
- Various plant rooms, stores and canteen refurbishments.

5 MAJOR IMPROVEMENT WORKS AND REPLACING MAJOR ELEMENTS

5.1 *Summary*

Since the bridge opened several major capital projects have been carried out to replace, strengthen or improve elements of the structure. These projects were necessary due to changes in traffic loading, design code changes and a risk assessment of shipping impact. These are listed below with the final project cost:

Strengthening of Viaduct Box Girders (Merrison Strengthening)	£1,145,485
Main Tower Wind Bracing strengthening	£2, 916,312
Main Tower strengthening	£12,742,999
Construction of Pier Defences	£9,914,357
Hanger replacement	£9,706,998

5.2 *Strengthening viaduct box girders*

The first of the major works undertaken was the strengthening of the viaduct box girders. The bridge was originally designed to sustain the loadings specified in BS 153: Specification for Steel Girder Bridges 1954 Part 3A: Loads. During the 1960s and early 70s the limitations of BS 153 with regard to the then current forms of steel construction became apparent. One of the areas of greatest concern was with welded stiffened plates in compression. The "Merrison Report" was published after a considerable amount of research and laid down guidelines for steel box girders as used on the Forth Road Bridge. During 1976, an assessment of the viaducts to the Interim Merrison Rules was carried out by designers of the bridge and a strengthening scheme put forward. The Cleveland Bridge and Engineering Company (Scotland) Ltd. carried out the work and the scheme was completed in 1978 at a cost of £1,145,485. This work allowed a temporary loading restriction on Abnormal Vehicles on the bridge to be removed. However, following advice from the Scottish Development Department, Babtie Shaw and Morton were appointed to carry out an independent appraisal of the strengthening scheme.

The Consulting Engineers reported that some additional strengthening to the box girder diaphragms would be required and the temporary weight restriction on heavy loads was again imposed. Detailed drawings were prepared and the additional strengthening was carried out by the Joint Board's own staff. The works were completed in financial year 1981/82 allowing the permanent removal of the temporary weight restriction.

5.3 *Increases in traffic loading*

Before going on to describe the other projects it is worth considering the background to the changes in loading that made the strengthening projects necessary.

Forth Road Bridge (and the Severn Bridge) were both designed for the same B.S. 153 vehicle loading. This loading was derived from a train of up to five 22 tonne and eight 10 tonne vehicles followed by lighter vehicles, and the clear space between the goods vehicles was assumed to be 5.4 metres.

In the UK during the 1960s and 70s the amount of goods moved by road measured in tonnes/km almost doubled and the number of goods vehicles with a gross weight over 28 tonnes rose from an insignificant number to 90,000.

In addition, the weight/length ratio of heavy vehicles increased with the introduction of containers. Furthermore, from observations of traffic jams it became obvious that vehicles spacings were in practice significantly less than had been assumed in BS 153. A design vehicular loading for four lanes derived from a statistical analysis of observed data on vehicle weight and length distribution, traffic mix and queue occurrence was found to be 20–160% higher than the original design loading for loaded lengths in the range 100–600 metres.

These trends continued and in 1983, 38 tonne articulated 5 axle vehicles (with the maximum drive axle weight of 10.5 tonnes), were allowed on UK roads and on the 1 January 1999, the allowable weight limit was increased to 40 tonnes on 5 axles (with the axle weight limit increased to 11.5 tonnes). Since 1994, 6 axle (max drive axle weight of 10.5 tonnes) 44 tonne vehicles have been allowed on UK roads but only when carrying containers for transfers to rail services or for delivery after being conveyed by rail. On 1 February 2001, 6 axle 44 tonne vehicles were allowed on all UK roads (with a max drive axle weight of 10.5 tonnes). By the end of 2003, over 50,000 lorries were registered at the higher weight limit and they accounted for over 45% of all heavy goods vehicles.

5.4 *Structural integrity appraisal 1983 to 1986*

In December 1983, following advice from the Scottish Office, and as a result of the concerns raised over increasing measured traffic loads, the FRBJB appointed W.A. Fairhurst and Partners to carry out a loading and structural integrity assessment on the bridge. This assessment included:

- An inspection of the parts of the bridge to be appraised
- Verification of the as built drawings by measurement and survey

and an appraisal of:

- Main towers including saddle assemblies
- Side towers including saddles and rocker boxes
- Anchorages and anchorage saddles

5.5 *Design live loading and bridge specific live loading*

The appraisal work was carried out in accordance with the then current British Standard, BS 5400: Part 2:1978. However, the live loadings that were to be considered were the loading developed at Severn Bridge by the Flint and Neill Partnership (the Severn Appraisal Load), and a load specific to Forth projected to the year 2010. In addition to normal traffic loads, 45 units of HB abnormal loading (180 tonnes) were also to be considered.

In order to determine a live load specific to Forth a traffic weight survey was carried out in March 1985 over a three week period. During the survey a significant sample of vehicles crossing the bridge were counted, classified by vehicle type and weighed. This was done using a reinstalled weighbridge and piezoelectric sensors.

It is interesting to note that based on several ad hoc surveys the inter vehicle gap used for the queue weight analysis was assumed to be 2.00 metres.

In order to deal with the statistical probability of the occurrence of loading greater than that being measured, it was necessary to determine mathematically the loading that had a probability of exceeding that measured within the remaining life of the bridge. This load was called the Characteristic Load.

For this work the Characteristic Load was derived for the full bridge life of 120 years rather than the remaining 99 years. The chances of the traffic on one of the bridge carriageways being as heavy as the Characteristic Load are 1 in 2,400 in any one year or 5% in 120 years. This load could only be achieved if that most unlikely succession of vehicles was constrained to queue across the bridge. The odds against a similar procession of vehicles on the other carriageway at the same time is over 5 million to one and the chance of the Characteristic Load occurring on both carriageways over significant lengths stretches the odds even further. Therefore, it was considered that a safe level of loading for the assessment of the bridge would be the application of the Characteristic Load to one carriageway while the other was subjected to the heaviest loading measured over the survey period.

In order to try to future proof the bridge against the increase in numbers and weight of heavy goods vehicles to the year 2010, the Characteristic Load at Forth was increased by 5%. This new load was named the Recommended Loading and was used at Forth as the bridge specific load in the assessment.

Table 1 shows all the loads considered and a comparison with the Severn Appraisal Load for the 1812 metre loaded length.

Table 1. Bridge traffic loads on full bridge – 1812 metres.

	Northbound		Southbound		
Loading	LH lane total load (tonnes)	RH lane total load (tonnes)	RH lane total load (tonnes)	LH lane total load (tonnes)	4 lanes i.e. both carriageways total load (tonnes)
BS 153 : 1954 Design	1080	360	360	1080	2880
March 85 Survey	1920	720	660	1530	4830
Recommended Forth Loading	2595	1040	695	1605	5935
Severn Appraisal Loading	3160	1580	1580	2105	8430

5.6 *Temperature effects*

The overall temperature effects considered were as set out in BS5400 with some modification when combined with live load. With regard to the towers it was felt that differential temperature effects could be considerable but no research had been undertaken on differential temperatures in vertical steel boxes. Therefore, thermocouples provided by the Transport Research Laboratory (TRL) were installed on Forth to produce continual measurements. The work carried out on the bridge had shown that there were considerable differences in the differential temperatures assumed in BS5400 and those measured on the bridge by TRL. This effect is caused by a low sun in mid winter heating up the south face of the tower leg while the north face remained at a low temperature in the shade. This results in a significant temperature differential of up to 40° Celsius and high stresses in the tower leg. However, as this occurs only in the middle of the day when the normal proportion of heavy goods vehicles to cars was considered to be low, the potential for the assessment live load to be reached is minimal.

5.7 *Wind loading*

Wind loading for the appraisal was based on BS 5400 clauses used in conjunction with the results of the aerodynamic investigation carried out for the design of the bridge by the National Physical Laboratory. Advice was sought from the Meteorological Office to determine the most appropriate wind speed.

5.8 *Findings of the appraisal*

The bridge was found to be, for the parts assessed, economically designed, properly constructed and well maintained for the loads assumed by the designers. However, due to increases in live load, and to wind and temperature loading changes, the most significant findings were that the main towers and cable saddles; the wind bracing and lateral thrust members to the main towers were significantly overstressed.

The appraisal noted that the combined effects of the increases in the traffic loads and wind forces on the main tower legs and cable saddles could be dealt with in the short term by local traffic management with a minimal effect on traffic flow. However, the overstressing of the main tower wind bracing and lateral thrust members was mainly due to an increase in wind load and therefore steps had to taken to strengthen these elements without delay.

5.9 *Strengthening of main tower wind bracing*

As a result of the appraisal the main tower cross bracing members and the deck level lateral thrust members were upgraded to withstand the higher wind loadings and in April 1988 the FRBJB approved a £2,629,000 tender from Cleveland Bridge & Engineering Ltd to carry out these strengthening works. The works were completed in 1992 at a final project cost of £2,916,312.

5.10 *Further appraisal work on the suspended span deck*

Following the initial appraisal it was recognised that the inspection and assessment work should be extended to cover the suspended span deck structure and Fairhurst were instructed to carry out following additional work:

- Verification of the suspended span deck structure and as built drawings following inspection
- Appraisal of the deck structure to BS 5400 using the Recommended Forth Live Loading

For the stiffening truss the critical loading lengths are between 150 and 250 metres and it was felt that the statistical approach at these loaded lengths was not appropriate. Therefore the Severn Appraisal load was used instead of the recommended loading. For the steel and concrete decks the critical load was the HB Abnormal vehicle.

The suspended span truss was found to be overstressed at the ultimate limit state under traffic loading. However, it was considered that a redistribution of load into the hangers would occur and the structure would continue to carry load. It was argued that for the traffic load case on a suspension bridge the serviceability limit state is critical and there was no overstress at serviceability in any of the members.

There was also an overstress due to the combined wind and traffic load case. However, controls in place on the bridge restrict high sided, heavier vehicles from crossing the bridge at wind speeds gusting at over 22 m/sec (50 mph). Therefore, this load combination was not considered further. No checks were made during the assessment on the main span deck for fatigue loading.

There was concern over the overstress found in the friction grip bolts in the connections. The bolts are waisted and although it is not normally permitted to take the shear strength of such bolts into account, after slip has occurred, a study was carried out into previous research and was combined with testing work. The conclusion was that shear could be permitted and the bolts were found to be acceptable.

Nothing in the analysis indicated that the loading of the whole bridge structure was limited by the strength of the stiffening truss and this was reported to the FRBJB by Fairhurst in 1994.

5.11 *Main tower strengthening*

In December 1989, following on from the cross bracing work, WA Fairhurst were instructed to undertake design proposals for the upgrading of the main towers and cable saddles.

Although the bridge elements had been assessed for the derived Recommended Loading the decision was taken to strengthen the towers and cable saddles to the current UK loading code for new bridges applicable in Scotland. This was Technical Memorandum SB6/88.

As well as increases in live load and wind loading the new code addressed differential temperature effects. In addition, the decision was taken to allow in the design of the strengthening for the possibility of a future fifth lane of traffic (light vehicles and cars only) running in the central reserve.

The scheme to upgrade the towers involved erecting new substantial steel columns within the existing tower legs and transferring a proportion of the dead load to the new columns, Figure 5. The assessment showed that a total axial load of 26,000 tonnes can occur in each tower and it was calculated that a maximum load of 6000 tonnes could be transferred to the new column in each tower leg. However, because of the reduction in vertical load, tensile stresses could develop in the

Figure 5. New tower steelwork.

north and south faces. Therefore, cover plates were designed to span across the joints in the original tower box joints to prevent them opening up. The load was distributed into the new column by a jacking force of 4000 tonnes just below the saddles at the tower tops and a further 2000 tonnes applied just below deck level. Also included in the work was the strengthening of the saddles.

Given the significance of these works an independent check (known in the UK as a Category III check) on the design of the upgrading works was carried out by Babtie Shaw and Morton.

Tender documents were prepared by WA Fairhurst and a £9,115,842.62 tender submitted by Grootcon (UK) Ltd was accepted by the Board in February 1994.

The works were carried out in demanding environment with the new 3.2 metre high tower leg sections being shipped out to the piers and loaded into the towers through enlarged accesses in the steelwork. Strand jacks located at the top of the tower within the box hauled the first unit up to allow the second unit to be slid in and bolted to the first. Then the two combined units were hauled up to allow the next unit to be similarly fixed and the procedure was repeated until all the units were installed. Final jacking of the load into the new columns was carried out overnight when live traffic loads and temperature effects were minimized, Figures 6 and 7.

Work on the tower tops on strengthening the saddles was carried out without significant disruption to traffic using scaffolding protection at the tower tops and directly above carriageway.

The works were substantially completed in June 1997 at a final out-turn cost of the project of £12,742,999.

In 1998 the project won the Scottish Saltire Society/Institution of Civil Engineers Award (Construction Commendation) and the prestigious British Construction Industry Civil Engineering Award.

5.12 *Pier defences*

In the early 1990s the Scottish Office was considering options for a second road bridge across the Firth of Forth at Queensferry. One of the studies carried out analysed the risk of marine vessels impacting on the main tower piers of the existing bridge.

Further studies were carried out by the Babtie Group on behalf of the FRBJB to determine the effects of a ship collision, and they advised that whilst the mass concrete piers could withstand the

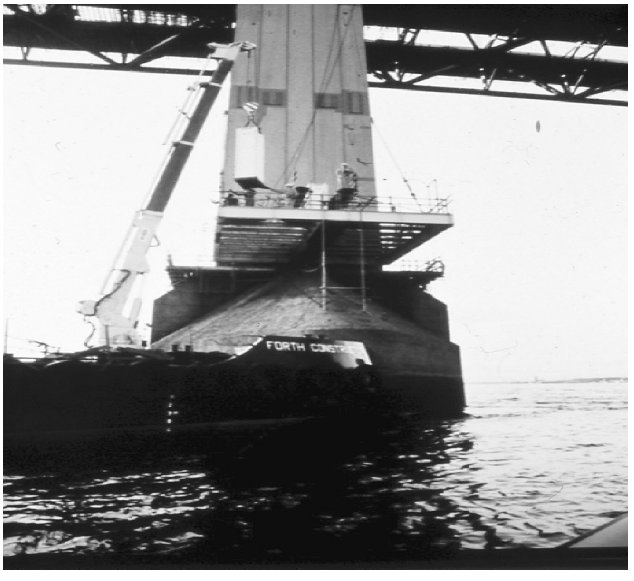


Figure 6. Loading in the new tower section at pierhead.

Figure 7. Inside the main towers installing the temporary strand jacks.

impact, it was possible for the overhanging bow of certain vessels to impact on a highly stressed tower leg before striking the pier.

In December 1995, the FRBJB approved the principal of a scheme drawn up by the Babtie Group to provide protection to the main towers in the form of rock filled cellular cofferdams at an estimated budget cost of just over £5 million.

Following detail design work and a tendering exercise, a tender submitted by Balfour Beatty for £6,863,891 million was accepted in August 1996.

The works commenced in October 1996 and comprised steel sheet piles driven to rock or to set in sand and gravel, forming large circular cofferdams on the navigation channel side of each tower. The cells were filled with rock and capped with a reinforced concrete slab. Cathodic protection was applied to the cofferdams, Figure 8. As the nesting site of the rare Roseate Tern was adjacent to the north pier no piling work could take place at the north side in July or August. In addition, the contractor was restricted by the noise level requirements imposed by the adjacent local authorities.

Work on site proved to be difficult in the harsh environment of the Forth and the works ran both over time and budget. The presence of a sunken vessel immediately adjacent to the north pier caused considerable delays until it could be removed. The works were finally completed in February 1998 at a final out-turn cost of £9,914,357.

5.13 *Hanger replacement*

In the original construction the hangers were nominally vertical and each hanger consisted of a pair of vertical steel wire ropes looped over the suspension cable and attached to the top chord of the steel deck truss by pairs of mutual sockets, Figure 9. In the main span the ropes were 44 mm diameter, 6 × 19 construction with a wire strand core. In the side spans the ropes were 52 mm diameter, 6 × 27 construction, again with a wire strand core

The most heavily loaded hanger in the main span carries some 176 tonnes and in the side spans the load is 224 tonnes. The hangers were designed to have a maximum breaking strength of 2.5 times the working load.

On 1 September 1995, one of the two ropes making up the hanger at mid span on the west cable was found to have frayed and traffic restriction were put in place until the erection of temporary

Figure 8. Constructing the cofferdams.

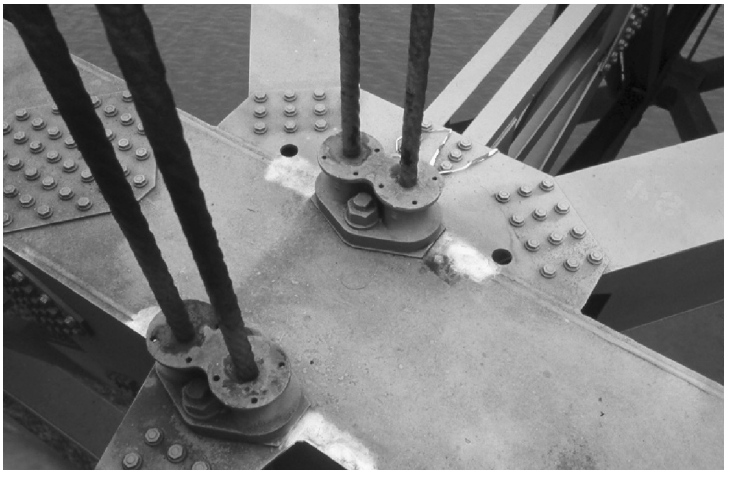


Figure 9. Original hanger socket detail.

load bearing steelwork was installed. The hanger was then removed for testing. On the basis of the first tests a further three hanger sets were replaced and removed for testing. Results indicated that five of the ropes making up the hangers had lost some 25% of their original strength and one rope had lost 57% of its original strength. The hanger assemblies tested were from mid span where the maximum lateral movement due to wind combines with the longitudinal movement of the bridge and the relative movement of the deck and main cables to produce the most severe rotational movement on the short hangers.

It was apparent from an examination of the specimens that the weakness in the ropes lay close to the sockets where a number of corroded and broken wires were found, Figure 10.

W A Fairhurst calculated the factor of safety as 1.32 based on the worst test result and the worst loading condition. Given that the worst loading condition was based on the yet to be introduced 40 tonne vehicles, it was decided that loading restrictions were not required. However operational procedures were put in place to prevent traffic jams occurring. In an attempt to try to find a non destructive test for the remaining hangers contact was established with the DMT Institute of

Figure 10. Hanger corrosion.

Conveying and Transport in Bochum, Germany. DMT had extensive experience of the testing of ropes and was originally the testing laboratory for the German Coal Industry. DMT carried out in situ NDT testing on 51 hanger assemblies and found damage in nine, all near mid span.

To correlate the results of the inspection one further hanger (98NE) was removed and taken to Bochum for inspection and testing. This hanger had exhibited three breaks in the outer wires. The results of two tensile tests were consistent with the original testing indicating an average loss of strength of 21%. Unfortunately, on stripping the rope for examination it was discovered that there were extensive wire breaks between the inner core and the outer lays that had not been picked up by the in situ NDT.

DMT considered that these wire fractures were not uncommon in rope of this construction where the central core is shorter than the strands wrapping around it. When an axial load is applied to the rope the outer strands move across the stiffer inner core and cause fretting between the wires in contact leading to notching and breaking, especially when combined with corrosion and rotational movements.

A check on other major suspension bridges provided information that the 33-year life of hanger ropes on Forth was not exceptional. The Severn Bridge ropes were replaced at 22 years of age and the Golden Gate ropes were changed at 40 years life.

Given the above the FRBJB decided in January 1997 to approve expenditure of £9 million to replace all the hangers on the bridge and WA Fairhurst were asked to prepare the scheme, Figure 11.

The hanger replacement work also included:

- Replacement of the cable band bolts
- Replacement of the hanger sockets and hanger screwed rods
- Replacement of the handstrands and handstrand upstands
- Replacing the surfacing on the northbound carriageway of the main span

As resurfacing of the northbound carriageway was due to be carried out in the summer of 1998 this work was incorporated into the hanger replacement contract to simplify the management of both projects.

The design and detail of the hanger socket was changed from the original, again each new hanger was a pair of ropes looped over the existing cable band but in the new detail each pair had a single socket at the ends and were fixed to a new deck detail with a single horizontal pin, Figures 12 and 13.

Figure 11. Replacing hangers.

Figure 12. Jacking down new sockets.

This was an improvement on the old detail as it allowed for any future replacement of one of the pair of ropes in the event of damage. In the original detail if one rope was damaged the pair had to be removed. One other change in detail was the use of resin sockets instead of white metal or zinc.

In March 1998, the FRBJB accepted a tender of £7,802,212.14 from Monberg & Thorsen A/S and the works started on site in May 1998.

Again the key element was to complete the works with minimal risk to the health and safety of the bridge users and the workforce, and to do so with the minimum of traffic disruption. A lot of the works were done at night with a carriageway closed. However a significant proportion of the works were carried out with traffic running, Figure 11. It is probably accurate to say that with changes

Figure 13. New socket detail.

in attitude and approach to health and safety legislation in the UK over the last 5 to 10 years, it would be a further challenge to see how a similar scheme could be carried out in the future without a continuous closure of a carriageway.

The Hanger Replacement Scheme is described in detail in a paper by Alastair Andrew. The works were substantially completed in January 2000 at final project cost of £9,706,998.

It should be noted that the hangers on the Forth Road Bridge are likely to require to be replaced at least twice more in the bridge's theoretical design life of 120 years.

6 RECURRING MAJOR WORKS

6.1 *Background*

In addition to the large capital projects described above, the painting and resurfacing of the bridge are two high cost items of recurring work, that for the purposes of budgeting FETA include as items of capital expenditure.

6.2 *Painting of the bridge*

In 1964 the FRBJB was advised that major maintenance painting was not required on the bridge for a period of 15 years. However, in 1978 a report to the Board highlighted known problems in relation to the condition of the paint systems on the bridge and emphasised the need for action that in some areas was considered to be urgent. It was noted that the cost of maintenance was going to be very much higher than previously considered.

Up until 1980/81 maintenance painting on the bridge had been carried out using a number of different paint systems and was limited to relatively small areas of blast cleaning and overcoating.

Following the introduction of a new permanent access system to the truss in 1980/81, major maintenance painting was undertaken on the suspended structure between 1980 and 1993. This major maintenance included the removal of the existing paint system using blast cleaning and the application of new paint systems based on zinc phosphates; chlorinated and acrylated rubbers, and epoxy esters.

From 1994 until 1999 areas of existing sound paint were washed down and overcoated using a chlorinated rubber system as required.

In 1999 an in-house review of the method of painting the truss was undertaken. This review was required because paint manufacturers had intimated that the production of chlorinated rubber paints and other paints with a high volatile organic compound (VOC) content were being phased

out and the UK Highways Agency were removing such paints from their specifications. The phasing out of these paints followed the 1990 Environmental Protection Act (EPA) which restricted the amount of VOCs that paint manufacturers could emit during production.

The review took into account the Scottish Environmental Protection Agency (SEPA) Joint Guidance Note PPG23 "Maintenance of Structures over Water – Pollution Prevention Guidelines" and SEPA's objective that over a period of time there should be a significant reduction in the volume of spent abrasive and paint entering the Forth.

Trials were carried out late in 1999 to determine whether or not the existing aged paint system could be over-coated. A modern two pack epoxy system was selected to be applied on top of the existing oleo-resinous/chlorinated rubber systems. Unfortunately, the results of pull-off testing of this option were poor, with recurring cohesive failure of the underlying layers.

Initial concerns about using a completely different paint system, with very different properties, on top of an aged multi-layered system proved to be well founded. The option remaining was to consider the removal of all the existing aged paint systems and the application of a new protective system on the grit blasted steel or remaining metal spray.

After considering advice from consultant Steel Protection Consultancy (SPC) and discussions with the Highways Agency and other bridge owners, the decision was taken to organise site trials based on the removal of the existing paint systems and the application of a modern two pack epoxy system. A paint system registered and widely used in the UK by the Highways Agency and other Overseeing Authorities was selected as the preferred system for the trials.

During the painting season of year 2000, trials were undertaken on the truss adjacent to the north main tower. Containment was put in place to prevent grit and paint spray reaching the carriageways and cycletrack/footways of the bridge. The erection of this containment was very labour intensive and thus expensive. The containment was successful in preventing the blast medium and paint overspray affecting bridge users, however a number of complaints of alleged paint damage to cars and property from residents and others in North Queensferry were received. After a long investigation by the Board's insurers and forensic scientists, the Board denied liability for the damage, as it was considered that all reasonable practical steps had been taken to avoid damage to third parties. However, a number of claims were settled on a strictly "without prejudice" to liability basis.

It had become apparent that the two pack epoxy paints were remaining active for some time after spraying and had the potential to cause damage to property a considerable distance away. This contrasted with the use of chlorinated rubber paints which when used even with a minimum level of containment, only caused damage locally if overspray occurred. In addition, chlorinated rubber paints can easily be removed quite satisfactorily from car bodywork and glass by the use of solvents. The same is not true for two pack epoxies, that unfortunately, are likely to cause pitting damage to car bodywork. Therefore, although the two-pack epoxy system appeared to be working well in terms of adhesion and applicability, problems of overspray and the provision of effective containment required a major re-evaluation of the painting programme on the bridge.

SPC was appointed to carry out a survey of the condition of the existing paint on the main and side spans, which was completed in February 2001. Following the survey, SPC recommended that approximately 50% of the zinc had to be removed, leaving sound metal spray on the flat areas, i.e. the main webs where porosity had not proved to be a problem. It was also recommended that an epoxy system be applied to the sound substrate and metal coat which would last 25 years after which time it could be overcoated to give a further 25 years without the need to blast clean.

As a result of this survey, a further series of large-scale trials on the bridge suspended span was organised. The purpose of the trials was to:

- Trial epoxy systems from different paint manufacturers
- Examine in more detail the problems of containment and removal of spent blast
- Examine different methods of application

The importance of a high standard of containment was essential to prevent a recurrence of the 2000 trial and an American containment standard The Society for Protective Coatings Guide No. 6 was adopted as there was no comparable British equivalent.

The trials were successfully carried out in September and October of 2001 and confirmed that even in the limited trial area the form and size of the steel truss making up the underdeck of the suspended spans of the bridge makes access and containment for painting work very difficult. The eight systems applied in 2001 continue to be monitored closely by SPC.

It was clear that the new standards of containment necessary for epoxy paint systems, requires a radical change to the methods traditionally used to paint the bridge. Trials on site; desk studies and studies of containment systems on other bridges in the UK and abroad, have been undertaken to determine the most suitable form of access, containment and paint system for Forth Road Bridge.

These studies into access and containment initially focused on the provision of a large, discrete, effectively contained underdeck access platform specifically designed for Forth Road Bridge. This platform was perceived to have a number of advantages over traditional scaffolding in terms of health and safety; minimising of contact points and speed of movement.

Following the feasibility studies and site trials, which highlighted the complexity and uniqueness of the platform, it was considered that the Authority's best interests would not be served by procuring, managing and operating such a platform to paint the suspended span truss.

Further it was recommended that tendering by the private sector would be the most suitable format to assess the commercial risks, and determine the most cost effective method of painting the suspended span truss, including the provision of access and containment required to meet all relevant health and safety, and environmental standards.

Estimating the overall cost of a contract to repaint the truss is difficult because there are so few precedents. An estimated cost has been produced by the Authority and external estimates have been obtained from SPC and Balfour Beatty Ltd who are currently project managing painting works on the adjacent Forth (Rail) Bridge.

These estimates must be considered to be first order costs but all three are indicating that the total cost of painting the 202,000 plus square metres of the suspended span truss by the private sector including access, containment, blasting and the protective systems required, would be in the order of £65 million and could be achieved within a 12 to 15 year programme.

Given the cost of this project SPC were instructed to carry out a further review of options available. SPC examined two options, a "Do Nothing" option and a "Spot Blast and Overcoat" option:

6.2.1 *Do nothing option*

There would be no initial financial commitment for this option. However, the report stressed that a far more difficult situation would be stored up in the future. It is anticipated that the coatings at the present time could last approximately 10 to 12 years before delamination by inter-coat adhesion loss and blistering of the zinc coating would occur. The main concern though would be that under the zinc film corrosion of the steel substrate, that is already occurring, would increase in intensity with loss of localised section of steel where the porosity of the zinc metal spray is already causing a problem.

The heavy pitting of the steel which would result from the do nothing option would mean that however careful the re-blasting and repainting would be in the future, the high standards and long life which are achievable, if painting is not delayed, would not be able to be met. The future re-blasting process would only be likely to achieve a 10–15 year life in the corroded areas and these would then become a constant maintenance problem in the future. In addition if steel corrosion becomes significant, section replacement may become necessary.

It is important to note that most of the corroded areas are visually concealed at the present time, but were exposed during the blasting trials and survey and many of these are associated with welds and flange joints which are critical parts of the structure.

6.2.2 *Spot-blast option*

The second option detailed in the report is to commence a programme of spot blasting localised areas of breakdown and patch coat these localised areas. These patches would then continue to be protected, but other areas either remote from those already patched or in adjacent areas, would

continue to appear year on year thereby setting up a costly, and technically inferior, annual maintenance spot blasting programme which would continue indefinitely for the life of the structure.

The spot blasting concept could be supplemented by the application of one or two coats of single pack material which is not as highly stressed as the two pack system that we are recommending to be applied to the sound metal and bare steel surfaces. This option would extend the life of the overthick coatings by perhaps 7–10 years, but there is the risk of flaking of the older coats and new paint from the original in the form of delamination and premature blistering, with associated hidden rusting of the steel. This hidden rusting could become significant in depth and area without the observer being able to see this through the over thick coatings. One of the main disadvantages of the spot blast option (with or without overcoating) is that the same level of access and containment is required as would be provided for carrying out the full blasting and repainting work.

Given that the access and containment elements of the work account for a large proportion of the costs it was not considered prudent to provide less than the full specification.

The report from SPC concluded that most cost effective long-term solution particularly from a technical standpoint was to adopt the recommendation to remove the existing paint by blast cleaning and apply a new paint system.

6.3 *Resurfacing of the suspended span decks*

The original road surfacing laid on the 12.7 mm thick steel deck plates of the main span of the bridge was 38 mm thick mastic asphalt over a 3 mm layer of rubber bitumen compound on Bostik 1255, Figure 14. The original and replacement surfacing requires to be impervious, durable, skid resistant and of minimum weight. The mastic asphalt used is grade T50 Grade S to BS 1447:1988, "Specification for mastic asphalt (limestone fine aggregates) for roads, footways and pavings in building". Coated 14 mm stone chippings are rolled into the surface to provide the required skid resistance. The mastic is also required to adhere well to the steel plates as this gives the composite action that reduces the magnitude of the stresses in the steel deck and increases the fatigue life of the deck welds.

All of the mastic-asphalt laid on the Forth Road Bridge over the years including the recent works in 2004 has been hand laid with mixing temperatures of up to 230 degrees centigrade. Unfortunately, there are few contractors left with the necessary skills to lay mastic-asphalt as modern bridges tend to have thicker bituminous surfacing layers which are more suited to withstand the wear from present day commercial vehicles.

Due to the increased volume of traffic and increased weight of heavy goods vehicles over the years, the surfacing on the bridge is failing sooner and having to be replaced at more frequent intervals. In addition, it is considered that surfacing life is also being shortened due to the detrimental effect of the introduction of the high pressure single tyre or "super single" on heavy goods vehicles which replaces the twin wheels on trailer axles. This is described in the Storebaelt paper

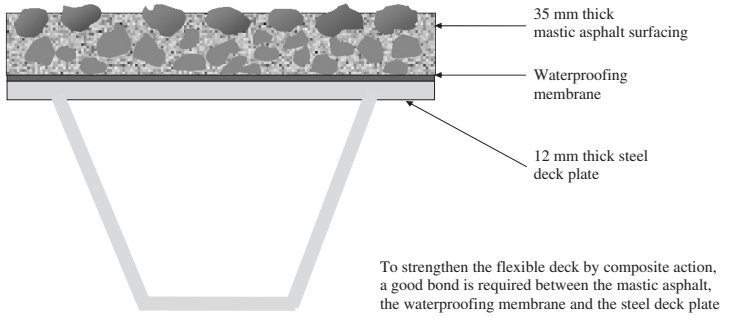


Figure 14. Typical section of them main span through the steel deck.

on the East Bridge, Denmark. The East Bridge has an orthotropic steel deck similar to the main span on the Forth.

The northbound main span was first resurfaced in 1988, after 24 years service, using epoxy asphalt. This material did not perform well in some panels and was replaced with mastic asphalt in 1990, 91 and 92. By 1998, re-surfacing was again required, this time after only 10 years service, and was completed in the summer of 1998. It is programmed to be carried out again in summer 2007. A spray applied waterproofing membrane was first used on the deck in 1988. Eliminator, a methyl methacrylate resin based waterproofing was specified and this waterproofing was repaired and over sprayed in 1998. One of the challenges in resurfacing is to remove the existing mastic to leave the waterproofing suitable for applying the replacement mastic.

Similarly, the main span of the southbound carriageway was first resurfaced in 1991 after 26 years service. This surface also failed 10 years later in 2001. Research had been carried out in 1998 to identify alternative, less disruptive resurfacing techniques, but trials failed to produce a practical alternative to mastic asphalt. In an attempt to reduce traffic disruption and find an alternative to complete removal of the existing material, the top 25 mm of mastic asphalt on the southbound carriageway was replaced during overnight working in 2001, leaving a nominal 10 to 12 mm biscuit of the surfacing material laid in 1991, on top of Eliminator. However, unfortunately, the resulting two layer mastic surfacing could not withstand the traffic loading and in June 2003 extensive cracking was noted in the overlay. Investigations into the failure were carried out using overnight possessions during September 2003 and these revealed that some of the overlay had separated from the base layer. From a structural perspective the desired composite action with the steel deck had been lost, thus reducing the fatigue life of the deck welds and there was a risk of failure of the waterproofing leading to the possibility of corrosion in the deck plate.

Further overnight trials were then undertaken to determine the most appropriate method of removing the existing bituminous surfacing from the main and side spans without causing damage to the existing bridge deck waterproofing membrane.

It was concluded that the bituminous surfacing on the main and side spans could not be removed from the Eliminator waterproofing membrane without severely damaging the membrane and prejudicing its integrity.

It became clear that the southbound main span carriageway would need to be resurfaced again and the following options were considered in November 2003:

- Option 1: Remove the surfacing and waterproofing membrane entirely on the main and side spans and replace with new Eliminator and mastic asphalt surfacing (estimated cost £3.5 m). On the steel main span deck, it was considered this option would provide approximately 8–10 years useful life of surfacing. However, it was expected be a slow, time-consuming process to remove and replace all the existing Eliminator. Spray-applying Eliminator is weather dependent, requiring dry, wind-free conditions. On the concrete side spans the same basic comments applied except that the life expectancy of the surfacing might be greater as there is less stress applied to it from the flexing of the stiffer deck panels.
- Option 2: Remove the surfacing by planning and water-jetting to leave an asphalt biscuit only a few millimetres thick and resurface using mastic asphalt (estimated cost £2.8 m). This second option, was considered for both the main span and side spans as practical and, in the short term, the least disruptive. However, it was emphasised that the long term life of any laminated surfacing which depended on an aged mastic (or Asphapol on the side spans) biscuit was unquantifiable due to the lack of its previous use. It had been already proven that leaving an existing layer of around 10 to 12 mm did not work and caused premature failure and there was no way of knowing how much of the existing material could be left. It was considered that leaving even a very thin layer increased the risk of premature failure, and that the surfacing life would not exceed 3–5 years on the main span, possibly longer on the side-spans.

Given the above, the Authority took the decision to proceed with Option 1. The disruptive nature of these works was of course recognised and the Authority undertook a massive consultation exercise with stakeholders and the general public to both explain the reasons for work and that it would

need to repeated on a 8/10 year cycle on each carriageway. Opinion was also sought on whether to carry out the works over an estimated 16 weekends or over a period of 33 continuous days.

The overwhelming response was to carry out the work during weekends when traffic is lighter and commuters are not greatly affected. The tourist industry was concerned over restrictions in July and August and for that reason works were not carried out during those two months. A large publicity campaign with nationwide signing was also planned to ask users to either avoid the bridge or use public transport.

Following a tendering process, Tarmac Ltd were awarded the contract to carry out the waterproofing and resurfacing of the southbound carriageway on the suspended spans of the bridge. The contract was prepared and supervised on site by W.A. Fairhurst. The total surface area of the carriageways is over 13,300 square metres and it worth noting that all the mastic for this considerable area was laid by hand.

The contractor was given possession of the northbound carriageway from Friday night at 21.00 hours until Monday morning at 04.00 hours. FETA operations staff carried out all the traffic management and liaison with the adjacent roads authorities, police and other stakeholders.

Works began in April 2004 on the main span adjacent to the north main tower. The method of removal developed by the contractor during earlier trials was to plane out approximately 25 mm of mastic and use ultra-high pressure jetting machines operating at 2000 bar pressure to remove the remaining mastic layer and waterproof membrane.

During the trials it was estimated that this jetting operation would take four hours per carriageway panel. The whole programme of sixteen weekend possessions depended on this time not being exceeded.

For reasons not yet fully understood it took twenty-four hours for the water jetting equipment, operating at 2500 bar to remove the remaining mastic and waterproof membrane from the first panel.

During the second weekend, attempts were made to plane off more of the mastic but it still took twelve hours to clear off the remaining material from one panel. Unfortunately, in attempting to remove more mastic, the planer damaged the steel deck plate and five hours were lost in welding repairs. It was later found that the heat from these repairs had damaged the paint coating on the underside of the deck.

In total, four main span panels were stripped down to steel allowing weld tests to be carried out. All tests proved satisfactory and no evidence was found of failure of the waterproofing to protect the deck steel from corrosion, Figure 15. However, clearly the inability to remove the waterproofing within the estimated time was having a serious effect on the programme and methods being employed were increasing the risk of permanent damage to the steel deck.

In able to gather as much information as possible, the equipment was moved to a side span panel where it was found easier to remove the mastic and waterproofing from the concrete slab deck than the trials had indicated, Figure 16.

Following discussions with the contractor Tarmac and consultant W.A. Fairhurst, it was decided to progress the side spans with Option 1, i.e. full replacement of waterproofing and surfacing. It was considered that as the waterproofing on the main span steel deck could not be removed without excessive effort and time, and an unacceptable risk of damage, it was concluded that some of the 1991 mastic would have to be left adhering to the waterproofing.

During the third weekend, six full panels on the north side span were completely waterproofed and resurfaced and further trials were undertaken on the main span.

As a result of this work, the developed method of working on the main span was to carefully plane out as much surfacing as possible down to the waterproofing, apply 500 bar pressure jetting to uncover any loose material in the remaining mastic "biscuit", rake out the old central expansion joint and overlay with new mastic asphalt.

Whilst no guarantee could be given on the life expectancy of this modified overlay treatment it was reasonable to expect between 5–7 years. Differences over the 2001 method of working have added to the level of confidence. Daylight working allowed cracks to be seen and cut out from the "biscuit", the water-jetting produced a clean roughened surface to assist the bonding of the overlay and debonded biscuit adjacent to the central joint was removed.

Figure 15. Waterproofing main span deck.

Figure 16. Hand laying mastic.

Further, it can be said that at 500 bar pressure, the integrity and adherence of the remaining mastic had, to a degree, been tested.

Advertising and notification of the works through advance signing; the media (newspapers, TV and radio) and by leafleting customers through the tolls had an impact and traffic volume during the first weekend was 33% less than the average over the preceding six weekends. However, despite that reduction the maximum delay was recorded at forty minutes and by the second weekend delays were up to one hour with a 28% reduction in traffic.

The scheme was successfully completed within the 16 weekend programme despite some very wet weather during the early part of the works. The full scheme cost for waterproofing and resurfacing the southbound main and side spans, including design, site supervision and traffic management was £3.683 million, a unit cost of £277 per sq. metre.

It should be stressed that the difficulty in removing waterproof membrane during restricted possessions of the deck introduces a major obstacle with unknown consequences for future resurfacing works.

7 ACCESS SYSTEMS

7.1 *General*

As in most long span suspension bridges there are areas where access for inspection and/or maintenance cannot be easily gained from a platform or vehicle parked on the carriageway or footway or from the ground or water below the deck. Therefore, some form of access platform or bridge gantry is required. These areas on Forth are the suspended span truss and soffit of the deck; the main cables; the main towers and the viaduct box girders, and deck soffit.

7.2 *Underdeck gantries*

Underbridge inspection platforms or gantries provide below deck access for inspection and maintenance of long span bridges. The use of such gantries has in the past been problematic in the UK with several accidents having occurred resulting in injury and fatalities. In an attempt to provide clear and authoritative guidance, The Institution of Structural Engineers produced a guide, "The operation and maintenance of bridge access gantries and runways", in 1996. The FRBJB in conjunction with other large bridge owners in the UK and the government was a sponsor of the preparation work and an active participant in the development of the guide.

7.3 *Suspended span gantry refurbishment*

In 1995/96 Charles Scott and Partners, Consulting Engineers, used the I Struct E guide to prepare a scheme to refurbish the two existing suspended span underdeck gantries that had been provided in 1985. The work involved strengthening the existing runway beams and providing a new support and trolley system to the gantries. The refurbished gantries are powered using air driven winches on wire ropes. These wire ropes are pulled out and anchored manually in front of the gantry before each move. The compressed air used is supplied from the ring main on the bridge. This work was substantially completed in 1998 at an out-turn cost of was £533,948. The I Struct E guide requires that gantries be subject to a performance load test and close inspection every six years. A special test frame was erected below the north viaduct to permit load testing.

However, removing the gantries for testing is a difficult and expensive procedure and the Authority have recently developed a technique for testing the gantries in situ.

7.4 *Viaduct gantry replacement*

Following inspections of the existing gantries on the north and south viaducts a decision was taken in 2001 to replace the gantries and runway beams, as it was not viable to modify or refurbish them.

Figure 17. Tower painting platform.

Again, using the I Struct E guide, Charles Scott and Partners designed a scheme to replace the 4.2 kilometres of runway beams and provide three gantries on each bridge viaduct. In November 2002, it was recommended that a tender of £2,473,073.37 be accepted from C. Spencer Ltd. for this work and this contract was substantially completed in 2004.

7.5 *Tower access painting platform*

Painting of the main towers of the bridge presents a particular difficulty given current traffic volumes and an access cradle was purchased for use by FETA's in house painting squad that would provide both containment and secure access for employees, Figure 17.

The cradle is designed to be suspended by wire ropes and hydraulically powered winches. In February 2002 a tender of £767,965.00 submitted by Alps UK Ltd to supply a platform and containment system manufactured by the Beeche Corporation (USA) was accepted.

7.6 *Tower painting dropped object canopy*

A risk analysis indicated that it was not acceptable to work above live traffic without providing some secondary containment to protect drivers, pedestrians and cyclists from the hazard of objects being accidentally dropped from the work platform. The only practical method of eliminating risk whilst painting the main towers was to provide a dropped object canopy above the deck level, Figure 18. The design and construction of this unique structure was carried out by W.A. Fairhurst and Pierse Contracting Ltd. The canopy has a clearance to traffic of about 6.00 metres. Immediately around the footprint of the towers it consists of a decking made up of a plywood and polystyrene sandwich on secondary steelwork on two large plate girders spanning both carriageways and supported on the main towers. The canopy extends outwith the tower to a distance of 18 metres each side (to the first hanger location) and consists of a high tensile net supported on steel portals spanning both the carriageway. As there are no supports in the central reserve the risk of collision and the possible collapse of the structure are overcome.

The total project cost of painting the towers likely to be £4.1 million giving a unit rate of £241 per square metre. Most of that cost is access and containment.

Figure 18. Dropped object canopy.

7.7 *Operating gantries and platforms*

All operation of FETA gantries and access equipment is carried out and supervised by FETA's staff and there is a rigorous training and refresher training programme in place and all supervisors, drivers, and users must have the appropriate training and certification. All access equipment has a separate operation and maintenance manual and all staff are made familiar with the requirements of both manuals.

7.8 *Further work*

The Institution of Structural Engineers has been asked by the UK's Highways Agency and the Health and Safety Executive to carry out a review of the existing I Struct E guide and this work is currently being progressed.

8 TOLL PLAZA

8.1 *Background*

More recently, FETA awarded a contract to replace the toll registration equipment that was installed in 1991. The existing equipment was described as "exposed to an increasing possibility of catastrophic failure" by consultant Hyder in its report to FETA in 2003.

8.2 *Current situation*

The project is due to be completed in July 2003 and includes for the installation of electronic toll equipment; ancillary back-office equipment; provision of new toll booths affording a higher level of protection from traffic and an improved working environment. A new canopy will provide both protection from the elements and a platform to support signs and services. An extension to the Authority's administration offices is included in the project. The estimated cost of the project is £7.0 million.

9 CURRENT AND FUTURE WORK

9.1 *Capital Plan*

FETA's 15 year Capital Plan for strengthening and improvement works is reviewed each year. The current total cost of work programme from 2006 to 2021 is over £118 million.

Some of the projects planned are described below.

9.2 *Main cable*

The main cables of a suspension bridge are the primary load carrying members. FETA's maintenance regime requires that the Forth Road Bridge is regularly inspected and whilst the external surfaces of the cable can be readily inspected this does not provide a definitive guide to the condition of the individual wires making up the cable.

The U.S.A. has many more suspension bridges of a greater age than Forth and the U.S. Transport Research Board of the National Academies commissioned a guide for main cable inspection under the direction of consultant Weidlinger Associates Inc of New York. This guide was to become known as NCHRP Report 534.

FETA kept abreast of the development of the guide and senior staff were present at workshops during its drafting. Although the guide was not published until 2005, FETA decided to adopt the draft guide recommendations that aerially spun cables on suspension bridges over 30 years old should be opened up for a first internal inspection.

This first internal inspection work has been completed and an assessment of the strength and remaining service life of the main cables and the monitoring and remedial works proposed is described in "Forth Road Bridge – First Internal Inspection, Strength Evaluation, Acoustic Monitoring and Dehumidification of the Main Cables".

Over the next three years FETA have committed a total of £13 million to install Acoustic Monitoring and Dehumidification on the main cables of the bridge.

9.3 *Anchorages*

The condition of the post tensioning strands in the Anchorages is an area of concern. There is no access to the base of the rock tunnels and the only inspection possible is to monitor for movement at the strand anchor bearing plate. A study to replace or augment the main cable is being undertaken by FETA and as part of the study methods of assessing the condition of the post tensioning will be sought.

9.4 *Bearings*

The main span deck is allowed to move freely in the longitudinal direction at both main towers, through a sliding lateral thrust bearing that has stainless steel bearing plates. The side spans are fixed at the side towers and also slide longitudinally at each main tower and are also restrained laterally by a sliding thrust bearing. These main lateral thrust bearings were refurbished during the main tower wind bracing strengthening. The bearings supporting the ends of the side spans and the viaduct bearings on top of each pier, and at the abutments, are the original bearings. On the shorter viaduct piers roller bearings are fitted whereas knife edge bearings are fitted on the taller piers and they are designed to allow flexure. These bearings have been inspected regularly but no major work has been carried out on them. A Principal Inspection is due to be carried out on all the bearings in 2007 and it is expected that they will all require to be replaced.

9.5 *Main suspended span deck joints (demags) and other carriageway joints*

The main carriageway joints at the main towers are rolling leaf type joints on curved girders called Demags after the German manufacturer. These joints are now showing signs of considerable wear and tear after over 41 years service and are due to be replaced in 2008/2009. Unfortunately, it will not be possible to carry out this work during overnight carriageway closures and longer duration continuous carriageway closures will be required.

Such carriageway closures are rare and the opportunity will be taken to replace the viaduct carriageway comb joints at the same time. These are also the original joints and have been repaired a number of times but are getting beyond economical repair.

9.6 *Parapets and barriers*

The bridge parapets and barriers were designed before an UK national standard was adopted and since then, the design requirements for these elements have significantly increased in line with heavier and faster modern vehicles. In February 2003, FETA appointed the Flint & Neill Partnership to carry out an assessment of the bridge parapets and barriers.

This assessment was to take the form of a strength assessment and geometric checks to the current UK assessment standards.

Following the assessment, shortfalls in both strength and geometric requirements were identified in a number of elements with a wide range in the degree of inadequacy. In order to quantify the shortfalls an effective rating has been calculated in accordance with the UK national standard. The rankings were generally medium to low when compared to other bridges although there is an argument for achieving a better standard on a crossing of such national importance as the Forth Road Bridge.

Flint & Neill recommend testing including off-site vehicle impact testing to determine the containment level of barriers. This will be of particular benefit on the suspended structure where aerodynamic performance and the strength of the underlying structure to carry the additional forces will require careful consideration. Tenders are currently being prepared for this testing work.

These issues as well as the Category A listed building status of the bridge all weigh heavily against barrier replacement which would be of considerable expense and cause serious traffic disruption over a long period of time. Other elements were recommended for upgrading or replacement where barriers will struggle to contain light vehicles. The outer barriers on the viaducts are of similar design to the barriers on the suspended spans. However, given that there is a footway/cycle track immediately adjacent to the carriageway on the viaducts, the consequences of a breach of these barriers was identified as having an increased risk to pedestrians and cyclists. There are other safety issues with respect to spacing of balusters and overall height on the pedestrian/cyclist parapets over the entire length of the crossing and appropriate warning signs have been displayed along the length of the footway/cycle tracks.

It must be emphasised that whilst the parapets and barriers fail to comply with current design standards, over 550 million vehicles have crossed the bridge since its opening and no cars have breached the barriers. Heavy goods vehicles overturning have damaged sections of barrier but this is not a load that bridge parapets are normally designed to withstand. The risk to bridge users is therefore minimal.

10 BRIDGE SPECIFIC ASSESSMENT LIVE LOADING (BSALL)

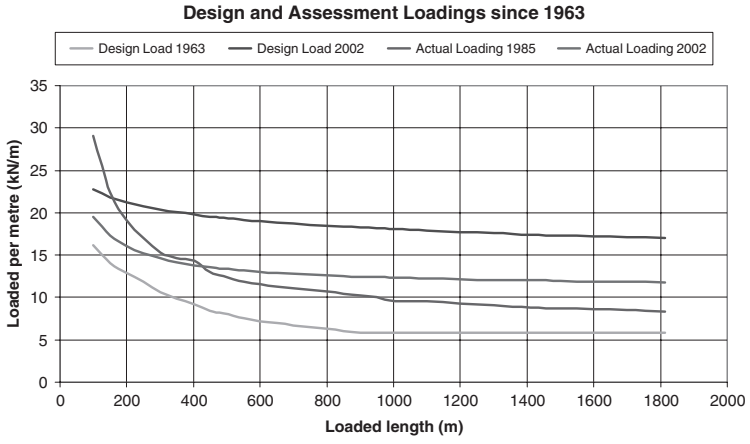
10.1 *Current and future work*

Following the assessment work carried out in the 1980s and 1990s, in 2001 the Authority installed a weigh in motion station in both carriageways at the north side of the bridge. This station classifies and dynamically weighs each vehicle crossing the bridge and allows a bridge specific live load to be determined more easily and accurately.

Using data from the station W.A. Fairhurst developed a BSALL for the bridge in 2002 and a comparison of the 2002 BSALL with the Original Design and the Recommended Load are shown in Graph 2.

As can be seen the loading derived from the 2002 survey has increased considerably from the 1985 work.

The 2002 BSALL was included in the assessment and audit of the main cable and will be used in conjunction with wind tunnel test being carried out at Glasgow University to reassess the suspended span truss.



Graph 2. Loading specific to Forth Road Bridge design and assessment loadings since 1963.

11 HEALTH AND SAFETY LEGISLATION CHANGES

11.1 *Background*

Since construction of the Forth Road Bridge in 1964, there have been fundamental and substantial changes in Health and Safety requirements in the UK. Following the implementation of the Health and Safety at Work Act in 1974 the focus of safety management shifted dramatically from prescriptive and very specific controls, to an overarching responsibility to reduce risk in a proactive manner. The 1974 Act has subsequently been supported by a growing number of regulations, many of them European driven and mostly occurring since 1985, that give explicit form to the duties within the Act. Given the nature and usage of the bridge it is becoming increasingly onerous to comply with the legislation.

11.2 *Impact on the bridge*

Health and safety changes have impacted in three main areas:

Firstly, new legislation and guidance have appeared in existing areas, and supersede previously acceptable systems and equipment. The I Struct E Guide for gantries is one such example of fresh guidance requiring original equipment to be upgraded.

Work practices have also changed. To access the suspension cables and stiffening truss now requires a risk assessment, employee medical assessment, safe system of work, fall arrest equipment, full body harness, rescue team, rescue boat, training and practised rescue procedure. Fitting a horizontal restraint wire alone cost £130,000 in 1997. Introduction of the "Working at Height" regulations came into force in 2005 and requires a further review of existing practice. For example, accesses within the anchorage chambers are currently being redesigned and ladders, hatches and edge protection to the main towers are being upgraded. The cost of the dropped object canopy described previously was increased by the requirement for a staircase for access where previously a ladder would have sufficed.

Increased traffic flow has brought its own Health and Safety problems. As discussed most maintenance activities can now only be carried out at night or at weekends during lane or carriageway closures. To enable these to be facilitated requires designated Traffic Management Operations. This requires operatives trained to a nationally accredited standard via the Lantra Section 12 scheme, using bought or hired specialist placement vehicles. In 2005 signage, vehicles and training for Traffic Management cost in excess of £100,000.

Secondly, the bridge was constructed in an era where many materials and processes were used that are considered unacceptable today.

The historical use of such products leaves problems of removal and disposal. Within the past few years there have been changes to the Control of Hazardous Substance Regulations, the Control of Lead at Work Regulations and Control of Asbestos at Work Regulations. Such products are universally expensive to remove due to their hazardous nature. For example during the cable inspection works dispensation was gained to renew lead paste. This came at a cost of containment, medical surveillance, training, protective equipment, decontamination units and specialist disposal.

The European Physical Agents Directive has impact on noise and vibration issues. Exposure limits are being cut dramatically and will affect methods of working. This translates directly into longer programme durations or more operatives, either of which increases costs. During the 2004 resurfacing contract, the contractor could not accelerate the programme due to worker availability on jackhammers, caused by reduced "trigger times" which under new measurements have fallen from an action level of 45 to 20 minutes, with a shift maximum of 85 minutes.

Thirdly, health and safety costs more because it now embraces more. FETA now has to consider health and safety issues, which were not previously recognised. Work related upper limb disorders, ergonomic considerations and manual handling now need to be considered.

12 CONCLUSION

12.1 *Acknowledgement*

Working to maintain and operate the Forth Road Bridge in a safe manner to allow users a safe and reliable crossing requires a large number of skilled people to work together effectively. This paper is dedicated to all those past and present who have been involved in this difficult but hugely rewarding task.

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Chapter 6

Inspection and monitoring of cable stayed structures – design, implementation and performance

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ABSTRACT: Recent improvements in stay cable systems have enabled the design and construction of longer and longer spans. Because these longer spans are more sensitive to loading (wind, seismic vibration, traffic), monitoring the loading and observing the structure's response has become an essential condition indicator. Also several older cable-stayed structures are showing signs of damage, often caused by wind-induced vibrations and corrosion. Inspection and monitoring can provide confidence in structural integrity and enable early detection of potential problems. In order to protect the substantial investment to design, construct, operate and maintain these bridges, owners must consider ways to mitigate the risk of loss and extend the functional life of the bridge. This paper describes a comprehensive, risk based, monitoring, inspection, and maintenance management system that has been successfully utilized on several signature bridges including the Rion Antirion in Greece, Penang Bridge in Malaysia, and France's Milleu and Seyssell.

1 INSPECTION AND MONITORING PROGRAM: DESIGN METHODOLOGY

Structures like bridges can be complex arrangements of several hundred structural components (beams, cables, panels, bearings …) made of a diversity of materials (steel, concrete, mortar, plastics …). Because each of these components and materials have their specific behavior and are subject to vulnerabilities of various origins, there does not exist a unique way to monitor a bridge. The design of a monitoring system shall consider the following aspects:

- monitoring objectives,
- number of measurement items necessary to understand the structure's behavior and follow its evolution,
- required durability and precision for each measurement items,
- required reliability: 24 hours a day all year round is the standard,
- data acquisition capabilities, data-processing capabilities, data analysis capabilities, data storage and archiving,
- data transmission protocols: remote automated measurements feeding data directly into the system, versus field measurements performed by staff on periodic basis,
- cost of monitoring system (including supply, installation and operation),
- comparison of alternative techniques,
- which authority is in charge of the operation and maintenance, what are their capabilities and expectations.

2 RISK ASSESSMENT

Prior to development of specific inspection and/or monitoring programs it is desirable to perform a comprehensive risk assessment of the structure(s) in question. This assessment will identify

				P	Occurrence	Nota
Material deterioration						
	Corrosion					
		Pt failure		2	*Every 10 years*	
		Concrete bursting		2	*Every 10 years*	
		Lost of steel cross section		2	*Every 10 years*	
		Vagabond current		2	*Every 10 years*	
	Earth					
		Land sliding		1	*Every 100 years*	
		Liquefaction		1	*Every 100 years*	
		Settlement		4	*Several time per year*	
		Scouring		2	*Every 10 years*	
	Other					
		Steel cold fragility		1	*Every 100 years*	
		Concrete alkali reaction		2	*Every 10 years*	
		Wear end ageing due to traffic		4	*Several time per year*	
		Wear end ageing of equipments		3	*Every year*	
Loads						
		Creep & Shrinkage		4	*Several time per year*	
		Truck loading (1)		2	*Every 10 years*	Over 13 t per wheel line as per calculation
		Wind static loading (1)		1	*Every 100 years*	Greater than 50 m/s = 10' average over 50 years = 60' over 120
		Wind dynamic loading (1)		3	*Every year*	Probability of dynamic event is 2 for the deck, 3 for for the stays and 1 for the pylon.
		Earth quake (1)		1	*Every 100 years*	Greater than the calculation one
		Tectonic movement		3	*Every year*	
		Traffic fatigue load		2	*Every 10 years*	
		Truck accident		2	*Every 10 years*	
		Ship collision (pier)		1	*Every 100 years*	
		Ship collision (deck)		1	*Every 100 years*	
		Aircraft collision		1	*Every 100 years*	
		Cable failure		2	*Every 10 years*	This event is considered by its consequences as an outer risk resulting from other causis such as corrosion or fire ...
		Snow		1	*Every 100 years*	Which create a significant overloading
Fire						
		Truck fire		2	*Every 10 years*	
		Boat fire		1	*Every 100 years*	Under the bridge
Other						
		Bad maintenance		3	*Every year*	
		Vandalism		2	*Every 10 years*	Terrorism is not considered
		Tsunami		1	*Every 100 years*	
		Influence of bad construction		2	*Every 10 years*	
		Lightning		2	*Every 10 years*	
(1) value to be defined						

Figure 1. Example of potential hazards listed with their occurrence probability level.

potential hazards for the specific design and environment of the bridge. The scope of this assessment must be sufficiently broad to insure that all hazards, environmental and structural elements unique to the structure are systematically considered in order to avoid overlooking less visible risks and to avoid the temptation of focusing solely on historical risks associated with a particular structural type.

In terms of risk assessment, several families of risks can be defined:

- Design risks: This category includes the ignorance of a problem, or the design mistakes, that leads to a structural insufficiency.
- Construction risks: This family involves the risks that are left after the construction, linked to deviation between the design expectations, and the field reality.
- Ageing risks: Materials ageing is inevitable; the process is however well known and is usually able to be controlled. An insufficient control could lead on the long term to structural insufficiencies.
- Operation risks: During the lifespan of the structures alteration to the theoretical behavior can appear, even if they are not directly related to structural aspects, but still causing structural modifications, or persons injuries, (i.e., water filling a foundation, ice on the pavement, …).
- Hazards risks: During its life a structure will be submitted to external events, that might be taken into account into the design, but that exceptional, and that still require specific reactions (i.e., earthquake, boat collision, fire, …).
- Maintenance risks: Maintenance of a structure may imply to modify its static conditions, and therefore create operation risks by itself. An insufficient knowledge of the risks related to an operation could lead to unwanted damages to the structure and to the workers, or users of the

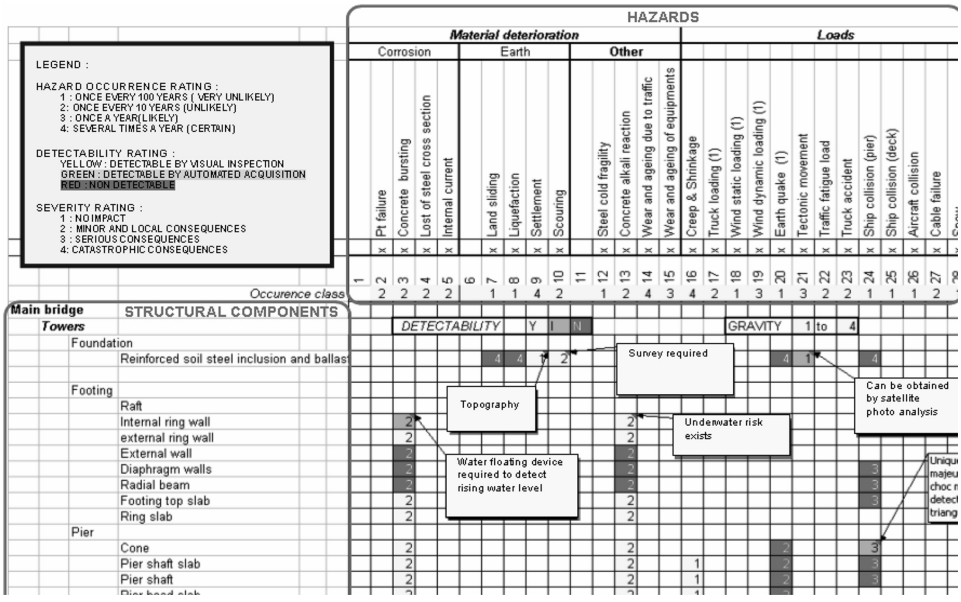

Figure 2. Example of cross-table showing the potential hazards, their occurrence probabilities and their expected impact on each structural component, the severity rating and the detectability rating.

structure. The risk assessment consists in estimating for each structural component the impact that each hazard may have.

A methodology derived from the manufacturing industry is applied in order to quantify the risks. The method consists in the following steps:

1. All structural components composing the structure are listed.
2. All potential hazards are listed, along with their expected occurrence probability (for example: every 100 year for an very large earthquake or every 10 years for a smaller earthquake, Figure 1.
3. Structural components and potential hazards are listed in a cross-table, Figure 2.
4. The impact of each hazard on each structural element is expressed with a severity rating (for example: no impact, minor impact or catastrophic impact) and a detectability rating (for example: can be detected by visual inspection, can be detected by sensory equipment, or can not be detected).

The resulting cross-table shown in Figure 2 contains all the information on potential hazards, their expected impact on each structural component and the techniques with which the hazards can be identified if they occur. This information is largely discussed by experts in order to ensure that it is as systematic and accurate as can be.

3 STRUCTURAL HEALTH MONITORING

The structural risk analysis concludes with determining which automated measurements shall be conducted on the bridge. Automated measurements generally consist in:

– weather stations for monitoring of wind direction, velocity, ambient temperature, humidity and air pressure,
– load cell on fuse restrainer,
– accelerometers for monitoring wind effects on deck,
– accelerometers for characterization of earthquake motion,

Figure 3. Stay cable displacement monitoring.

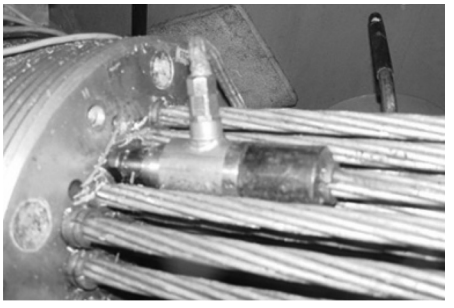

Figure 4. Stay cable force monitoring.

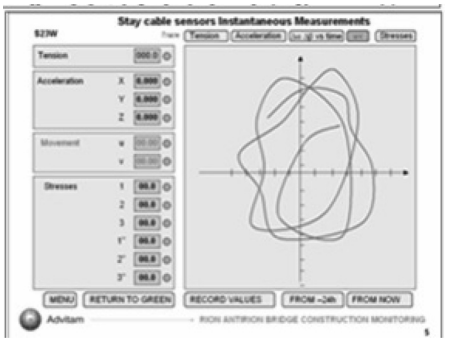

Figure 5. Data analysis software: Stay cable displacements.

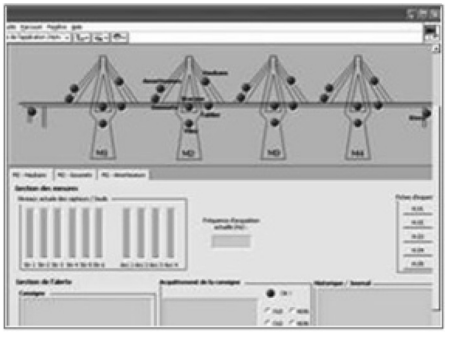

Figure 6. Data analysis software: Over-view of measurements points.

- accelerometers for monitoring earthquake effects on deck,
- accelerometers for monitoring earthquake effects on pylons,
- monitoring of expansion joint openings,
- detection of water inside pylon legs,
- road temperature,
- characterization of wind effects on stay cables:
  - accelerometers for monitoring vibrations induced by traffic and wind,
  - monostrand load cells for monitoring of tension in the stays,
  - displacement transducers for monitoring of the vibration amplitudes,
  - strain gauges for monitoring of internal stresses in gussets (connection between stay anchorage and deck).

Automated measurements items are generally implemented:

- to measure quantities in inaccessible locations (for example, internal stresses in the pylon concrete legs, or acceleration at mid-stay),
- to collect automatically and simultaneously large amounts of data that require synchronization (for example, comparison of pylon movements with earthquake ground motion, or stay cable forces with stay cable vibration amplitude),
- to collect real-time information (for example, temperature or wind speed).

4 INSTALLATION CONSIDERATIONS

On new structure, the monitoring system is preferably installed at an early stage in the construction of the bridge, so that it can monitor the behavior of the bridge during construction activities.

In particular, the early monitoring of stay cable vibrations can enable a direct verification of the damping ratio in the installed stay cables. This information can prove particularly useful later on during the installation of external damping devices.

On existing structures, installation of a monitoring system generally does not require traffic interruptions of lane closures.

5 VISUAL INSPECTIONS

For all other items that do not require automated acquisition, visual inspection items are defined and documented (inspection type, frequency, points to be measured …). These visual inspection points generally correspond to material degradation phenomena, like concrete cracking, paint chipping, steel corrosion or bearing sliding. These items do not require constant attention or cannot be easily monitored, and therefore it can be more cost-effective for these items to schedule periodic inspections rather than to install automated acquisition systems. Inspection drawings and checklists are 100% electronic: Advitam has developed and implemented Scanprint® software to handle information related to infrastructure inventory, collection of field inspection data and management of maintenance information, Figures 7, 8 & 9.

Figure 7. Software for structure inventory.

Figure 8. Collection of field inspection data with handheld computers.

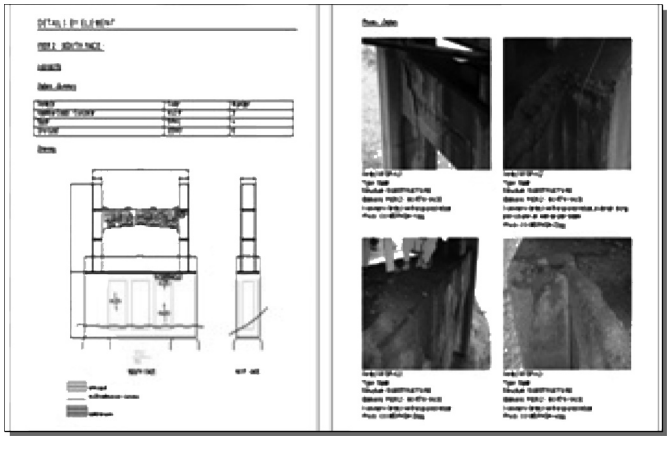

Figure 9. Automatically generated inspection report.

6 CASE STUDIES

6.1 *Rion-Antirion Bridge (Greece)*

Located 250 km west of Athens in Greece, the Rion-Antirion Bridge is the longest cable stayed bridge in the world with a continuous deck of 2,250 meters, Figure 10. For a bridge as complex and unique, it was essential not to re-use conventional designs but to invent a tailor-made system that would ensure that the monitoring would be fitted to the structure specificities and meet the operator's expectations. Advitam proposed an innovative approach in the design, development and implementation of a comprehensive monitoring system for that exceptional structure.

The system uses a combination of automated measurements and visual inspections to monitor the behavior of the structure during the construction phase and during the operation phase. A detailed analysis of the potential hazards threatening the structure served as the baseline for deciding which structural components should be monitored and which monitoring technique was preferable. The resulting design ensures that the system meets the operator's expectations, providing him with relevant and valuable information on the condition of the structure. Advanced software processing is used for data collection, archiving and analysis.

6.2 *Millau Viaduct (France)*

Opened to traffic in 2004, the Millau Viaduct is 2460 m long with roadways 260 m above the valley, Figure 11. Advitam installed a structural health monitoring system to control the forces in the deck and the stays during the launching sequences of the construction.

The monitoring system used during construction is then adapted to be used as a permanent monitoring system. It controls the fatigue characteristics in the stay-deck connections and monitor acoustically the stays.

6.3 *Penang Bridge (Malaysia)*

Built in 1985, the Penang Bridge stays are made of PT bars connected with couplers encased in a metallic tube injection filled with grout. A Soundprint® acoustic monitoring system has been installed to detect and locate wire breaks as (if) they occur anywhere along the length of the stays. Consequently, fatigue or corrosion related problems can be detected, Figure 12. A total number of

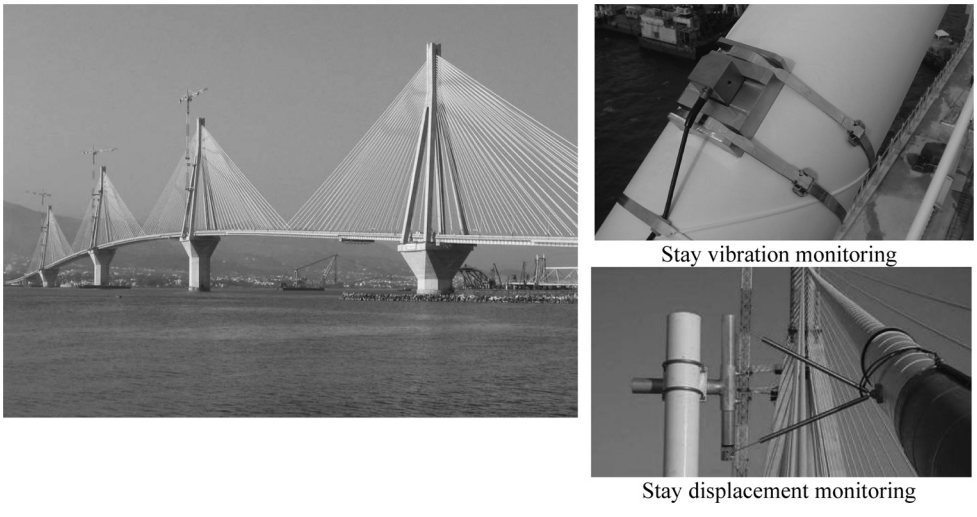

Figure 10. Overview of the bridge and details of stay monitoring.

Figure 11. Millau Viaduct.

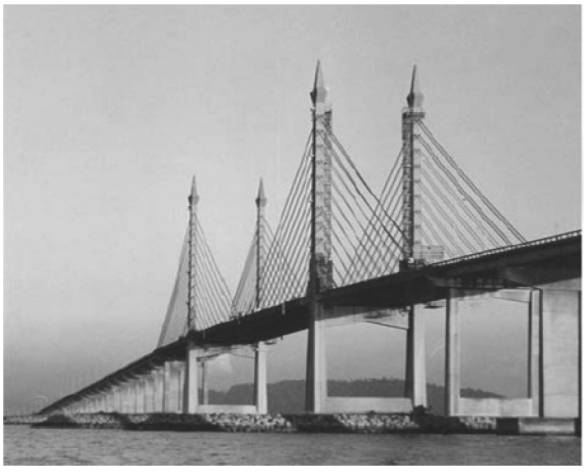

Figure 12a. Overview of Penang Bridge.

Figure 12b. Detail of acoustic sensor.

200 sensors have been installed to monitor 120 stays. The sensors are glued onto the external steel tube of the stays at each extremity.

The installation of the system was completed in 2 months and consisted of:

- The installation of sensors on the pylons.
- The installation of all wiring on the deck (more than 22 km of coaxial wires).
- The installation of the acquisition unit and the wireless communication system for real-time monitoring by internet access.
- Tests and settings of the acoustic filters.

6.4 *Seyssel Bridge (France)*

Built in 1986, the Seyssel Bridge is an asymmetric, inverted-Y pylon, cable stayed bridge. Advitam perfomed in 2003 an extensive investigation of the structure, including:

- tension verification in the stay: lift-off and vibration measurement,
- chemical analysis of concrete in pylon leg,

Figure 13. Aerial view of Seyssel Bridge.

– detailed visual inspection of anchorages and identification of corrosion,
– acoustic monitoring of the stays,
– geometric survey.

7 PERFORMANCES

In order to provide relevant information, structural health monitoring systems must deliver accurate data consistently. Systems shall be designed to overcome events such as lightning, strong storms and earthquakes. Also provisions must be made in case of malfunction and power-shortages.

Advitam's experience on both new and existing structures has shown that structural health monitoring systems, when designed properly, can sustain such events and even provide valuable information on the reaction of the structure to the event, i.e., the reaction of the structure to extreme loading.

Design life of a structure health monitoring system can vary depending on the application from 3 to 10 years. Just like any other equipment, health monitoring systems require some maintenance and replacement of minor components is expected.

Because structure health monitoring system have the capacity to acquire and transmit large quantities of information, it is easy to get lost under tons of data. Filtering of relevant data and data interpretation are essential. Software algorithms can be developed to identify patterns and filter data sets so that the interpretation of data can be more straightforward. Custom-made data acquisition software and data analysis software can be developed to secure archiving and allow seamless interpretation of data (Figs 5 & 6). Alert and alarms level are defined so that the system is capable of automatically notifying any anomaly in the measured data (for example, excessive stresses in a structural component, or unusual vibration) or any malfunction in the system (for example, power shortage or communication error).

8 CONCLUSION

Structural health monitoring systems can provide essential information of a bridge behavior in the short term (response to loading, reaction to exceptional events …) and in the long term (long term deformations, changes in tensions, stress redistributions …). Structural health monitoring systems can therefore become management tools, ensuring better decision making and helping reduce overall life cycle cost.

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Chapter 7

Repainting of Seto-Ohashi Bridge for increase of durability

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ABSTRACT: The Seto-Ohashi Bridge is located in the center route of three routes that connect Honshu, Japan's main island, and Shikoku. It consists of three suspension bridges, two cable-stayed bridges, and one truss bridge on the strait and some viaducts on the islands. They are highway-railway combined bridges whose overall length is approximately 10 kilometers. The bridge, mainly comprised of steel structures, is in a severe corrosive environment. Therefore, the heavy duty coating system is used to prevent corrosion. The bridge has served for 18 years since the opening, and allover repainting program, which makes LCC minimum, started with modified paint system for 1,800,000 m^2 of repainting area. In order to develop the repainting schedule, the thorough coating inspections to grasp the deteriorated condition and the prediction of the coating exhaustion were implemented. Then, the priority and the timing of repainting were determined based on the results. The careful quality control is conducted to obtain the high quality coating film in the field repainting work.

1 INTRODUCTION

The Honshu-Shikoku Bridges comprise the three routes that cross the Seto Inland Sea between Honshu and Shikoku. Location of the three routes is shown in Figure 1.

Seto-Chuo Expressway is a four-lane mortorway from Hayashima Town in Okayama Prefecture to Sakaide City in Kagawa Prefecture with total length of 37.3 km. The bridges on this route have

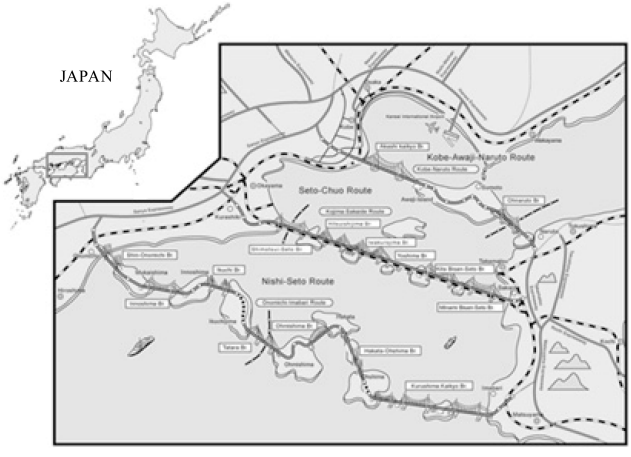


Figure 1. Outline of Honshu-Shikoku Bridges.

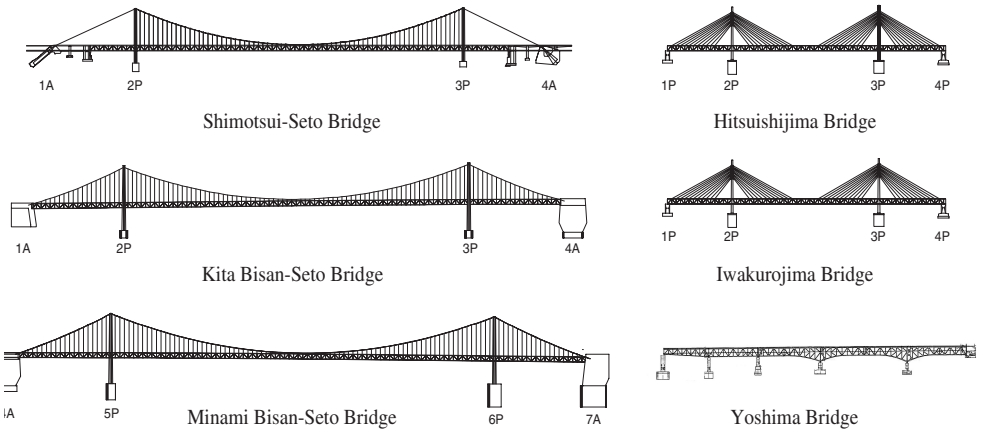


Figure 2. Side views of Seto-Ohashi Bridges.

Table 1. Dimensions of Seto-Ohashi Bridges.

Type	Bridge	Total length (m)	Center span length (m)
Suspension	Shimotsui-Seto	1,400	940
Suspension	Kita Bisan-Seto	1,538	990
Suspension	Minami Bisan-Seto	1,648	1,100
Cable-Stayed	Hitsuishijima	790	420
Cable-Stayed	Iwakurojima	790	420
Truss	Yoshima	850	245 (maximum)

Figure 3. Seto-Ohashi Bridge, completed in 1988.

deck structures for highway and railway. This route comprises three suspension bridges (Shimotsui-Seto, Kita Bisan-Seto and Minami Bisan-Seto Bridges), two cable-stayed bridges (Hitsuishijima and Iwakurojima Bridges), and one truss bridge (Yoshima Bridge) on the strait and some viaducts on the islands. Figure 2 shows side views and Table 1 shows dimensions of these bridges. These bridges are grouped together and called the Seto-Ohashi Bridge. The route was completed in April 1988. This paper describes the repainting of the Seto-Ohashi Bridge for increase of durability.

Table 2. Coating specification for external surface of steel structures (shop, spray).

Layer	Coating Specification	
1	Inorganic zinc-rich paint	75 μm
2	Mist Coat	–
3	Thick-coating type epoxy resin under coat	60 μm
4	Thick-coating type epoxy resin under coat	60 μm
5	Epoxy resin intermediate coat	30 μm
6	Polyurethane resin top coat	30 μm
	Total film thickness	255 μm

2 BASIC POLICY FOR MAINTENANCE

Features in maintenance of the Honshu-Shikoku Bridges are; 1) these bridges must be maintained for very long term (assumed more than 200 years) because reconstruction will be extremely difficult, 2) tight control is required in maintenance because these bridges are located in severe corrosive environment, 3) safe and uninterrupted smooth traffic must be always provided due to no alternative routes near by. Therefore, the basic policy in maintenance is "preventive maintenance"; enough inspection is conducted, and repair work is implemented before deterioration emerges or deterioration is at an early stage. Painting film control is also based on this policy.

3 PAINTING SPECIFICATION

3.1 *Painting specification at construction stage*

Total painting area on external surface of the Honshu-Shikoku Bridges is vast, and repainting work occupies large portion of the repair works. Therefore, cost reduction in repainting is very important issue.

Heavy-duty coating system is adopted, because the structures are exposed to severe corrosive environment. Inorganic zinc-rich paint used for the first layer contains metallic zinc powder in high concentration and has a strong corrosion protection performance due to sacrificial anode action of zinc. Epoxy resin paint which protects inorganic zinc-rich paint and has waterproof property is used for the third and forth layer. And, fluorocarbon resin paint or polyurethane resin paint which has weather resistant property is used for the top coat. While the fluorocarbon resin paint is used for newly built bridges such as the Akashi Kaikyo Bridge, Kurushima Bridges and so forth, the Seto-Ohashi Bridge used polyurethane resin paint. At construction stage, steel structures were painted mostly in shops up to the top layer and only joints were painted on-site. Painting specification for external surface of steel structures in shops is shown in Table 2.

3.2 *Repainting specification*

Repainting policy for the Honshu-Shikoku Bridges restricts repainting layers to deteriorated top and intermediate layers, keeping the inorganic zinc-rich paint and the under epoxy resin layers which protects the inorganic zinc rich paint in a sound state. This is because it requires on site blasting process, which is disadvantageous in terms of quality control, cost and environment, if the entire coat is repainted. Repainting specification is shown in Table 3. In repainting, only fluorocarbon resin paint, which is more durable than polyurethane resin paint, is specified as a top layer.

Table 3. Repainting specification for external surface of steel structures (on-site, brush).

Layer	Coating specification	
	Surface treatment	–
1	Epoxy resin intermediate coat	30 μm
2	Fluorocarbon resin top coat	25 μm
	Total film thickness	55 μm

Figure 4. Peeling of coating film on truss members.

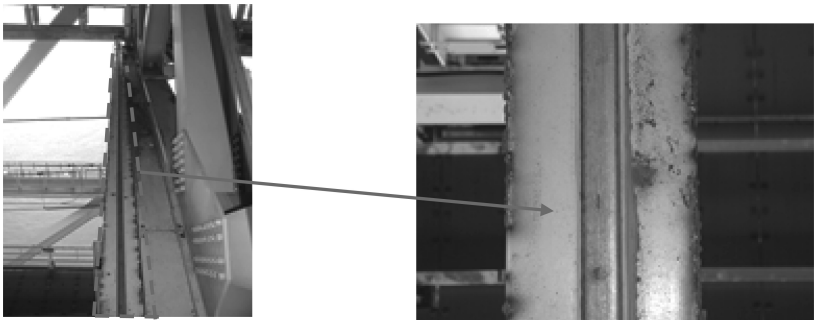


Figure 5. Deterioration on edge.

4 REPAINTING

4.1 *Present condition of coating film*

The main deteriorations of the coating film in the Seto-Ohashi Bridge are peeling and decrease of coating film thickness.

Almost all peelings occurred between the top coating and the intermediate coating (Figure 4). It is considered that the peeling was caused by inadequate removal of contaminations such as water, oil and salt and so on, or by painting work in high humidity in the shop.

The edge portion is liable to be damaged because of lack of coating thickness (Figure 5). Therefore, it was specified that the edge should be rounded with a radius of more than 2 mm in new bridge fabrication of Honshu-Shikoku Bridges after 1989.

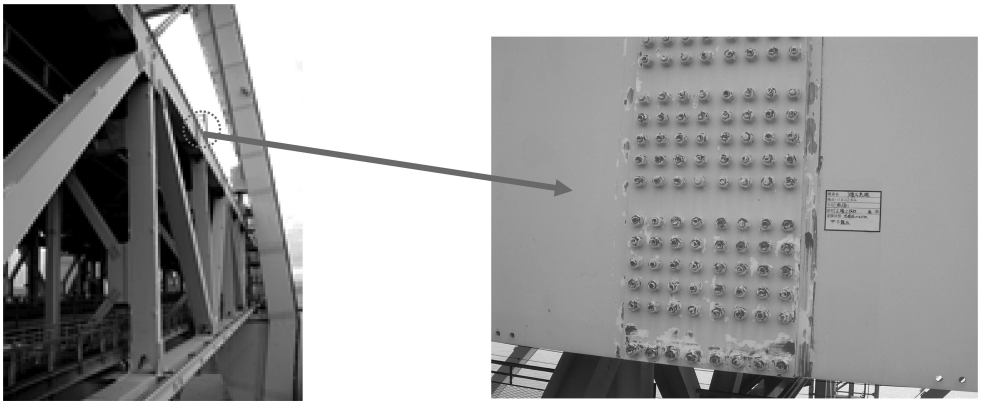


Figure 6. Decrease of coating film thickness on joint.

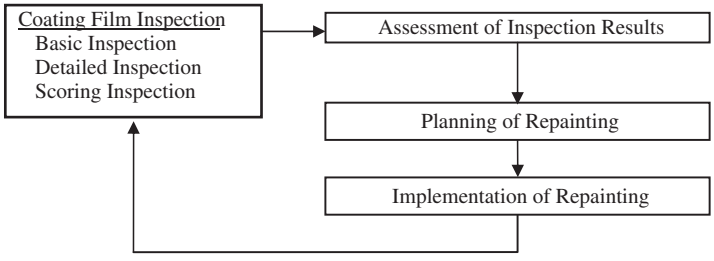


Figure 7. Flow of coating film control.

Decrease of coating film thickness occurred in many joints of truss members (Figure 6). The painting over the inorganic zinc-rich paint on joints was conducted in the field at the construction stage. The cause was supposed that the coating film thickness was apt to become uneven because the field painting was conducted with brush, or the conditions of the field painting work such as temperature and humidity were unfavorable.

4.2 *Flow of coating inspection*

The following three type inspections are conducted in order to grasp the coating film soundness, and the repainting schedule is planned according to the inspection results. Figure 7 shows the flow of coating film control.

4.3 *Results of coating film inspection*

4.3.1 *Basic inspection*

Basic Inspection is conducted at intervals of two years to record the position and area of damaged coating film in extensive portions by sight survey with a binocular. The coating film deterioration can be found early by this inspection.

The damaged areas measured in the basic inspection conducted up to 2005 are shown in Figure 8. The Hitsuishijima Bridge, the Kita Bisan-Seto Bridge and the Iwakurojima Bridge have large damaged areas. And the areas are concentrated on the local panels.

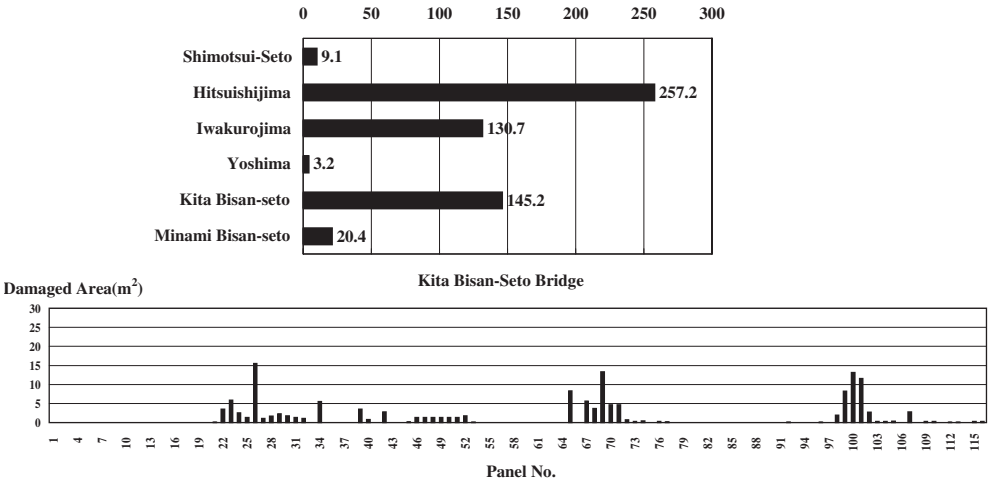


Figure 8. Results of basic inspection.

Figure 9. Inspection scene.

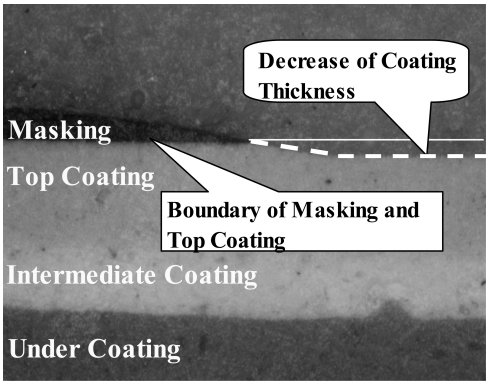


Figure 10. Coating film section by optic microscope.

4.3.2 *Detailed inspection*

Damage of the coating film can be found in the basic inspection, but detail of deterioration of the coating film cannot be checked. Therefore, fixed observation points are setup at each bridge, and the detail of deterioration is continuously checked in the periodic investigation on coat thickness, glossiness and adhesive strength. Inspection scene and sectional photograph of the coating film with optical microscope are shown in Figure 9 and Figure 10, respectively.

Exhaustion speed of the top coating is obtained by the periodical measurement of the coating film thickness, and the coating deterioration curve is drawn as shown in Figure 11. In the sea crossing bridges of the Honshu-Shikoku Bridges, timing of the initial exhaustion of the under coat is predicted, and starting and completion timing of the repainting is determined. This measure based on the basic policy of "preventive maintenance" enables to enhance the durability of steel bridges and to reduce the life cycle cost.

According to the results of detailed coating inspection, the overall repainting works of the Hitsuishijima Bridge and the Kita Bisan-Seto Bridge having high exhaustion speed of top coating is to start in 2006.

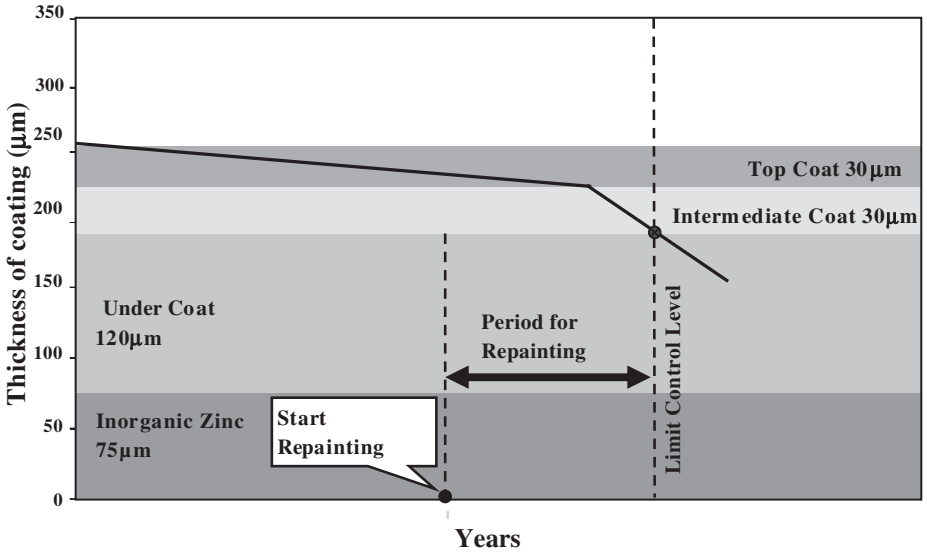


Figure 11. Coating deterioration curve.

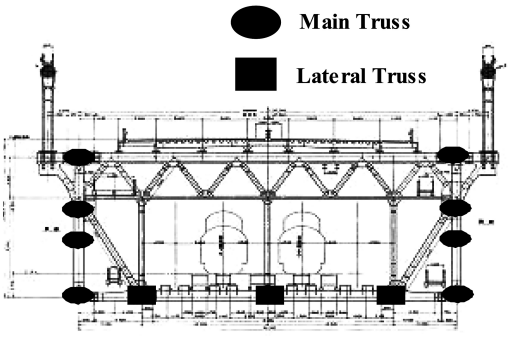


Figure 12. Survey points for coating.

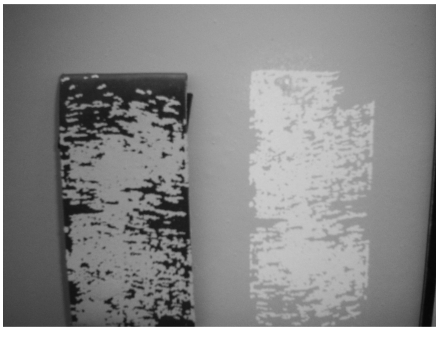


Figure 13. Peeling of coating film adhesion.

4.3.3 *Scoring inspection*

Scoring inspection is conducted by sight and touch survey prior to the repainting, and the scores are given to each bridges according to the coating film conditions (rust, exposure of inorganic zinc-rich paint, peeling, swelling, crack, chalking).

As above mentioned, the basic repainting polity is that coat exhaustion is limited to the top and intermediate layers, keeping the under layer and the inorganic zinc-rich paint in a good state. Therefore, the deterioration is not so much progressed yet. In order to supplement this inspection, the following "adhesion test" and "transparency test" were conducted.

Adhesion test using a packing tape was conducted at the 3,000 points shown in Figure 12 in every 2 panels in order to examine the failure of adhesive strength which cannot be checked by the sight inspection. The peeling of coating film is shown in Figure 13. Figure 14 shows the ratio of the number of peelings to the number of inspection points. Almost all peelings occurred between the top coating and the intermediate coating.

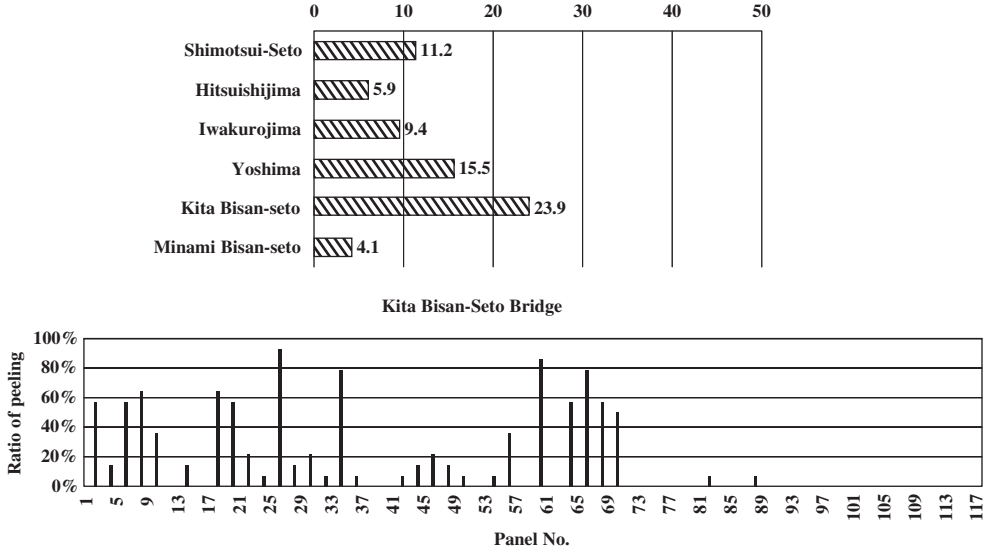
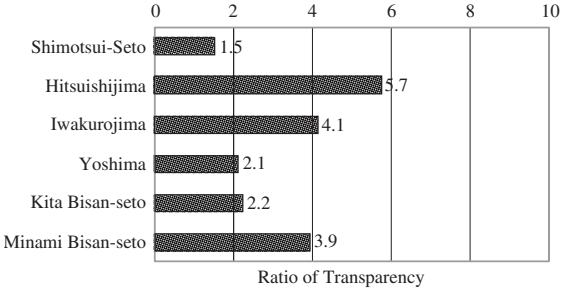


Figure 14. Results of adhesion test.



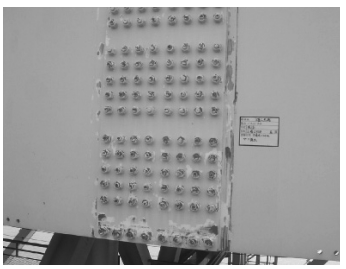


Figure 15. Results of transparency test.

Transparency test was conducted to check "transparency", the under layer become visible, due to the decrease of coating thickness. Figure 15 shows the ratio of transparency. Transparent portions were concentrated on the field painting areas such as joints.

4.4 *Priority of repainting based on coating inspection results*

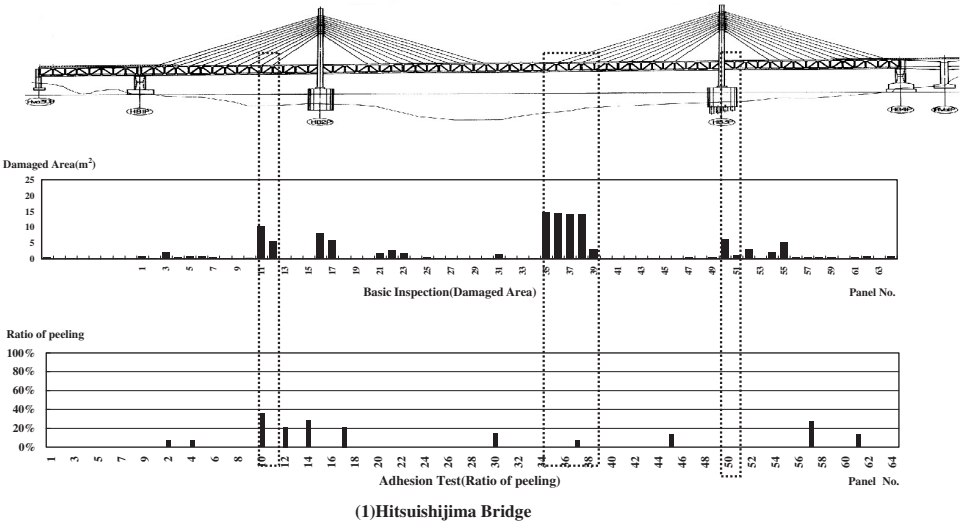
From the results of the coating film inspections, the priority for repainting was set as shown in Table 4.

4.5 *Bridges and panels to be repainted in 2006*

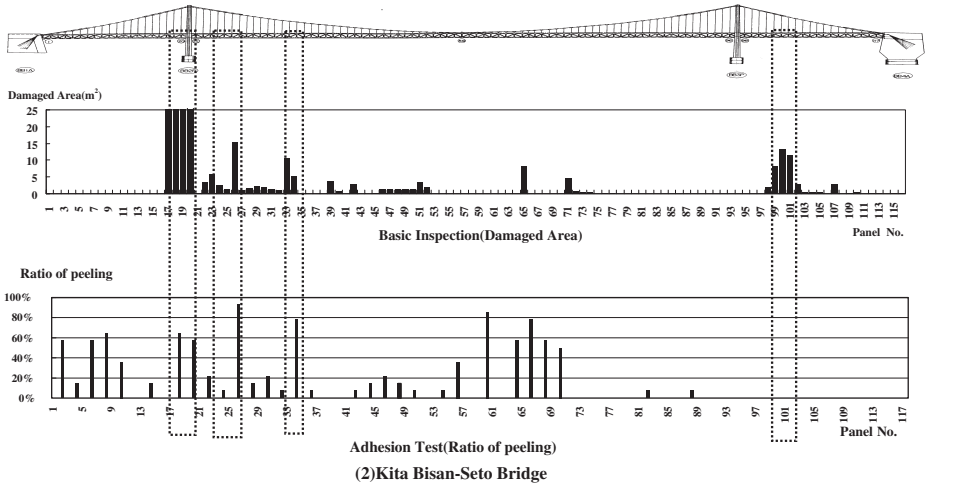
The Hitsuishijima Bridge and the Kita Bisan-Seto Bridge were decided to be repainted in 2006 considering the priority. And the panels to be repainted were decided as shown in Figure 16 in consideration of the damaged area and the ratio of Peeling.

Table 4. Priority for repainting.

Bridge	Basic inspection	Scoring inspection		Average	Priority
		Adhesion test	Transparency test		
Hitsuishijima	1	5	1	2.3	1
Kita Bisan-Seto	2	1	4	2.3	1
Iwakurojima	3	4	2	3.0	3
Yoshima	6	2	5	4.3	4
Minami Bisan-Seto	4	6	3	4.3	4
Shimotsui-Seto	5	3	6	4.7	6



(1)Hitsuishijima Bridge



(2)Kita Bisan-Seto Bridge

Figure 16. Bridges and panels to be repainted in 2006.

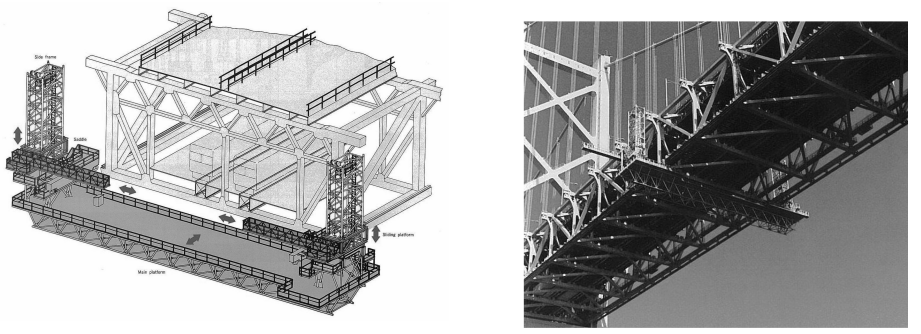


Figure 17. Outside girder maintenance vehicle for suspension bridges and cable-stayed bridges.

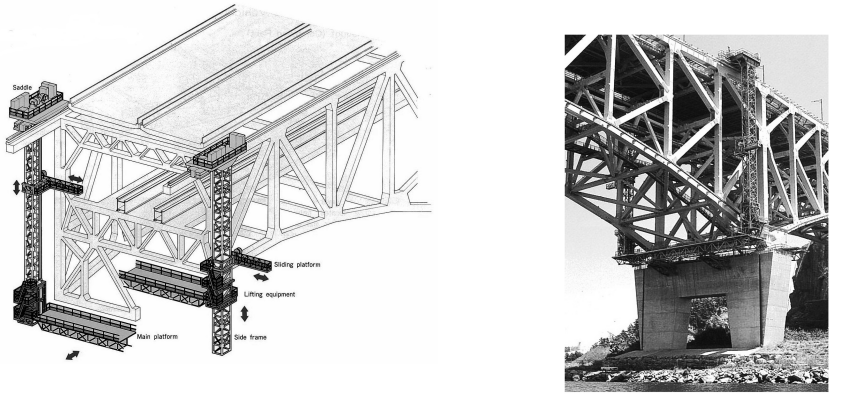


Figure 18. Outside girder maintenance vehicle for curved-chord truss bridge.

5 REPAINTING WORK

5.1 *Scafolding for repainting*

Maintenance vehicles are prepared to conduct inspections and repainting works efficiently, because the structures of suspension bridge, cable stayed bridge and truss bridge in the Seto-Ohashi Bridge are complicated in three-dimension.

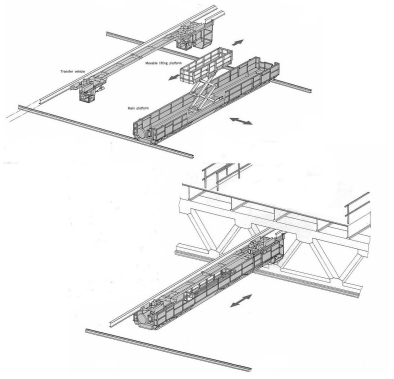
Outside girder maintenance vehicle is U-shaped, surrounding the girder. The overall Maintenance vehicle moves in the longitudinal direction, and the expansion stage moves in the transverse direction and up and down direction. By the three dimensional move, approach to extensive areas of repainting is possible (Figure 17).

Maintenance vehicle for the curved-chord truss bridge is prepared in the place where the lower side of the bridge is curved. This vehicle has a function of automatic positioning by sensor so as not to strike the bridge at the time of traveling. The expansion stage moves in the transverse direction and up and down direction (Figure 18).

Inside girder maintenance vehicle is for the under surface of deck and the upper portion of the girder. Hoist stage is equipped to approach the deck easily (Figure 19).

5.2 *Quality control*

The severe quality control is conducted for the repainting of the strait crossing bridges in order to secure the high grade coating film and improve the durability. The grade of surface treatment is



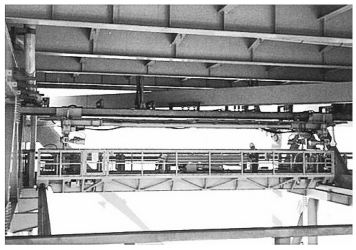


Figure 19. Inside girder maintenance vehicle.

confirmed by the test execution. The amount of the attached salt is checked, and the limit is specified as 20 mg/l. If the measured value exceeds the limit, the surface of the bridge is washed by water. The higher grade qualification is required for the repainting manager of the strait crossing bridges than that of on-land bridges.

6 CONCLUSION

The measures conducted for increasing durability are summarized as follows:

1) The heavy duty coating system is used to prevent corrosion.
2) The detailed coating inspection is conducted to check the deteriorated condition of the coating early.
3) The planned repainting work is scheduled based on the inspection results and the prediction on deterioration in accordance with the basic policy of preventive maintenance.
4) The careful quality control is conducted in the field repainting work to obtain the high quality coating film.

After 18 years of service, the Seto-Ohashi Bridge stepped into the full-scale maintenance period, as typified by the initiation of overall repainting. Various efforts have been paid for coating inspections, quality control and so forth in order to keep the bridges in sound condition and to reduce the maintenance cost.

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3 Instrumentation, testing and monitoring

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Chapter 8

Instrumentation and monitoring of critical structural elements unique to suspension bridges

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ABSTRACT: The assessment of the condition of cables in suspension bridges and other cable-supported structures has typically been problematic. The inaccessibility of all but the outer layers of wire has limited the value of visual inspection of the cables, and traditional non-destructive testing techniques, such as radiography and magnetic flux, have severe limitations that render them unsuitable for most applications. Also, over the past ten years, new proprietary monitoring and inspection technologies have been introduced that have provided bridge owners and engineers with better information about the condition and rate of deterioration of these critical structural components. Continuous remote acoustic monitoring of the main cables of a suspension bridge was first implemented in North America on the Bronx Whitestone Bridge in 2000, having previously been used successfully to monitor the condition of post-tensioned structures. Since that time it has been applied on several suspension bridges in the US, France and United Kingdom. In 1999, Southwest Research Institute used its proprietary magnetostrictive sensing ("MsS") technology to assess the condition of suspender ropes on the George Washington Bridge in New York City. Both of these techniques have since been refined and improved to the point where they now provide comprehensive information about the health of bridge cables, and allow informed decisions to be made about repair or replacement. This paper provides an overview of current cable inspection practices. It traces the evolution of the new technologies, presents case studies, and discusses the limitations and challenges associated with each technique. The example of the FORTH suspension bridge will be particularly developed.

1 INTRODUCTION

The assessment of the condition of cables in suspension bridges and other cable-supported structures has typically been problematic. The inaccessibility of all but the outer layers of wire has limited the value of visual inspection of the cables, and traditional non-destructive testing techniques, such as radiography and magnetic flux, have severe limitations that render them unsuitable for most applications. Also, over the past ten years, new proprietary monitoring and inspection technologies have been introduced that have provided bridge owners and engineers with better information about the condition and rate of deterioration of these critical structural components.

Continuous remote acoustic monitoring of the main cables of a suspension bridge was first implemented in North America on the Bronx Whitestone Bridge in 2000, having previously been used successfully to monitor the condition of post-tensioned structures. Since that time it has been applied on several suspension bridges in the US, France and United Kingdom. In 1999, Southwest Research Institute used its proprietary magnetostrictive sensing ("MsS") technology to assess the condition of suspender ropes on the George Washington Bridge in New York City. Both of these techniques have since been refined and improved to the point where they now provide comprehensive information about the health of bridge cables, and allow informed decisions to be made about repair or replacement.

This paper provides an overview of current cable inspection practices. It traces the evolution of the new technologies, presents case studies, and discusses the limitations and challenges associated with each technique. The example of the FORTH suspension bridge will be particularly developed.

2 RISK ANALYSIS OF A SUSPENSION BRIDGE

Handling the risks of a structure is a work that is done naturally by any Design Engineer that has the design criteria given by the project documentation. For most of the common structures, design criteria are conservative, and leave no risk apart. Although in terms of maintenance strategy, most common structures are inside typical inspection policies, which generally allow detecting the risks before any bad event occurs, for special structure, the common regulations for inspection and maintenance are not applicable. Typically they don't consider the specific risks of the structure, and they require extensive investigations on areas where no risk could have an impact on the function of the structure, or where risks could be detected by much cheaper ways. For those reasons common regulations are not applicable for large structures. In terms of risk assessment, several families of risks can be defined:

- Design risks: This category includes the ignorance of a problem, or the design mistakes, that leads to a structural insufficiency
- Construction risks : This family involves the risks that are left after the construction, linked to deviation between the design expectations, and the field reality
- Ageing risks: Materials ageing is inevitable; the process is however well known and is usually able to be controlled. An insufficient control could lead on the long term to structural insufficiencies
- Operation risks: During the lifespan of the structures alteration to the theoretical behaviour can appear, even if they are not directly related to structural aspects, but still causing structural modifications, or persons injuries, (ie: water filling a foundation, ice on the pavement, …)
- Hazards risks: During its life a structure will be submitted to external events, that might be taken into account into the design, but that exceptional, and that still require specific reactions (ie: earthquake, boat collision, fire, …)
- Maintenance risks: Maintenance of a structure may imply to modify its static conditions, and therefore create operation risks by itself. An insufficient knowledge of the risks related to an operation could lead to unwanted damages to the structure and to the workers, or users of the structure.

Each of these risks appears at different periods of the life of the bridge, and for each risk a different prevention or corrective action has to be set. Talking about long term cost management, each one of these risks calls for specific actions, to allow reducing the cost impact of the risks. Simulations have been performed on structures that shows that significant saving of money are generated by a proper risk reduction, and increased knowledge of the condition of the structure. Fighting against risks is an everyday task of any manager of a facility. In case of a civil structure, it could be handled by several ways:

- Design: Increase of safety coefficient or increase of redundancy, control that the behaviour of the bridge is in accordance with the design once it is under operation
- Construction risks: Effective quality control, and transfer of the knowledge of the problems liable to persist through the time
- Ageing risks, these risks mechanism is well known, and the control of their evolution can be clearly defined from the beginning of a project
- Operation risks: simple indicators might be set to call for modifications of the operation of the structure
- Hazards: once a hazard occurs, detailed procedures allow managers to apply checklists to evaluate the risks and the availability of the structure to fulfil its goal
- Maintenance: the designers have the knowledge by the time the structure is built of any important situation in which the structure shall never be into. Before performing a maintenance operation, risks shall be transferred to the main contractor in charge of the works. Tools shall be given to

the structure manager, in order to guide him into the preparation of these tasks, to avoid any unwanted situation.

Although every risk listed above is perfectly known, their management rarely appeals to rational techniques. The risk assessment methodology is therefore a good reply to anticipate according to the knowledge at a given time the risks to which a structure will be submit and then to identify appropriate remedial actions.

The following chapters will give examples of monitoring strategy for a better assessment of three typical risks of suspension bridges:

- Corrosion of main cable
- Corrosion or fatigue of hangers
- Slippage of suspension collars.

3 ACOUSTIC MONITORING OF MAIN CABLE OF FORTH ROAD BRIDGE

3.1 *Description of Forth Road Bridge*

The Forth Road Bridge was opened to traffic the 4th of September 1964. At that time it was the major suspension bridge of the world outside the United States, and the 4th of the world, with a main span of 1006 m.

The two main aerially spun cables from which the suspended deck is hung are 590 mm in diameter, and each is made up of 11,618 high tensile wires with a 4.98 mm diameter. The suspended deck is made up of a steel stiffening truss, with three longitudinal air gaps at roadway level to improve aerodynamic stability.

The main cables are anchored at each end to take the 13,800 tonnes of total load in each cable. These anchorages are concrete, cast in tunnels of tapering section, cut into rock at an inclination of 30° to the horizontal. The tunnel lengths vary between 56 and 79 metres.

The suspended span decks are hung from the main cables by 192 sets of four hanger ropes with diameters of 44.5 and 52.4 mm. These hanger sets take loads of 176 and 224 tonnes. Between 1998 and 2000 all these hanger sets were replaced without interruption to traffic.

The main towers are of welded cellular high tensile steel construction and rise up over 150 metres above high water level. These towers were strengthened in the late 1990's to take the ever

Figure 1. View of Forth Road Bridge.

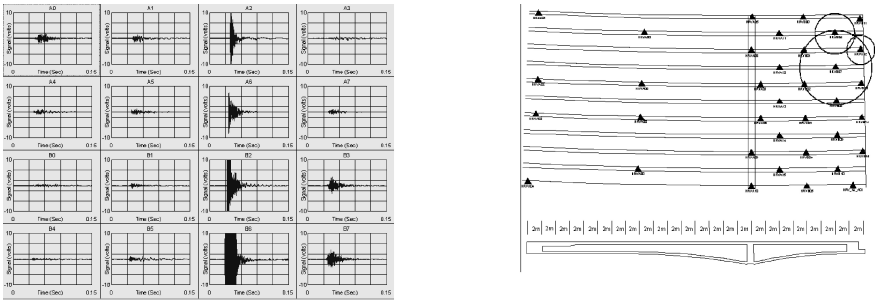

Figure 2. The acoustic signal collected by each sensor is used to identify and calculate the position of the acoustic event.

increasing weights of heavy goods vehicles crossing the bridge. The traffic loading now experienced on the bridge is now more than double that expected by the engineers who designed the structure in the 1950's. This has been due to the increasing weight and number of heavy goods vehicles on UK roads.

In 2004 and 2004 an inspection of the main cable has been carried out under the supervision of Faber Maunsell, and some wires have been found broken due to corrosion. Consequently a reduction in the cable strength has appeared, and if no action is taken it is expected that the bridge will not be safe to car traffic near the year 2020.

FETA (Forth Estuary Transport Authority) has therefore decided to undertake two main actions:

- Installation of an acoustic monitoring system to control that the sections inspected are representative of the overall condition of the main cables, and to control the speed of deterioration
- Installation of a dehumidification system to stop the corrosion process. Its efficiency will be analysed using the acoustic monitoring system, which will allow re-evaluating the lifespan of the bridge.

3.2 *Principle of acoustic monitoring*

The principle is simple and effective: acoustic sensors (accelerometers) distributed in key locations along a structure will detect the energy released by a fracture event, Figure 2. Each sensor is connected using coaxial cable to an on-site data acquisition unit. The system monitors continuously but collects no data until triggered by an acoustic event falling within pre-set limits. Software filters are then applied to reject events of no further interest. Events that successfully pass these tests may be wire fractures. They are sent via the Internet to a processing center for review by an engineer who will classify, calculate their position and notify the owner, Figure 1. Provided with high quality data of this type, the engineer can appraise a structure with knowledge of the actual failures in tensioned elements, and their locations, in the entire structure over the monitoring period.

Potentially it can be applied to any structure in which the integrity of steel tensioned elements is difficult to appraise visually. However, until recently, continuous, unattended, remote monitoring of large structures was not practical or cost-effective. The availability of low-cost data acquisition and computing hardware, combined with powerful analytical and data management software, resulted in the development of a continuous acoustic monitoring system which has been successfully applied to a variety of civil engineering structures since 1994.

This continuous acoustic monitoring system has passed successfully several evaluations carried out by independent organizations. Evaluation trials demonstrated that:

- The signals generated by wire fractures can be detected above general noise levels and the acoustic events can be captured reliably in the presence of the ambient noise,
- They can be distinguished from any larger events which are not of interest (other events that might generate acoustic responses in a bridge include vehicles going over discontinuities in the road

Figure 3. Mono-directional acoustic sensor and wireless data transmission unit.

surface, small objects such as stone chips striking the concrete and expansion joint defects responding to traffic),
- The source of each event can be located with satisfying accuracy,
- The system can perform long-term unattended remote monitoring with minimum maintenance.

3.3 *Implementation on suspension bridges*

The Bronx Whitestone Bridge in New York City has a main span of 701 meters and was opened to traffic in 1939. The acoustic monitoring system was installed during a rehabilitation of the main cables in 1997. This work involved removal of the circumferential wire wrapping, repair of broken wires and the application of corrosion-inhibiting oil to the wires.

Consequently, it was possible to cut wires in the cable to test the system's recognition and location capabilities. Ambient background noise detected by the accelerometers was insignificant compared with the events caused by wire cuts. Construction activity involving wedging of the main cables did produce sufficient noise to cause acquisition of events but this was easily distinguishable from wire cuts. Wire cuts were located within 0.22 m of the actual location.

Following successful implementation on The Bronx Whitestone Bridge, the system has been further developed in order to facilitate its installation: highly sensitive mono-directional sensors allow to capture any single wire break over 150 m along the main cable, while wireless data transmission eliminate the tiresome wiring task, Figure 3.

Wired uniaxial sensors have been installed in 2004 on the Ancenis Bridge in France. Wireless uniaxial sensors are used on the Waldo Handcock bridge in the USA and are going to be installed in 2006 in Scotland on the Forth Road Bridge.

3.4 *Conclusion*

The technology of acoustic monitoring is non-destructive and non-intrusive. It has been demonstrated that it can monitor continuously an entire structure, detect and locate failures or abnormal events. Provided with valuable information about deterioration, engineers and owners can perform focused investigation, optimize the timing of repairs and develop more accurate maintenance budgets.

Acoustic monitoring is a management tool that can assist owning authorities to monitor the residual life of their structures and provide confidence in their structural integrity.

4 INSPECTION OF HANGERS

The magnetostrictive sensor ("MsS") technology is a guided-wave technology that Southwest Research Institute developed for inspecting a long-length of structures such as cables, rods, and pipelines. Guided waves are elastic waves in sonic and ultrasonic frequencies that propagate in a

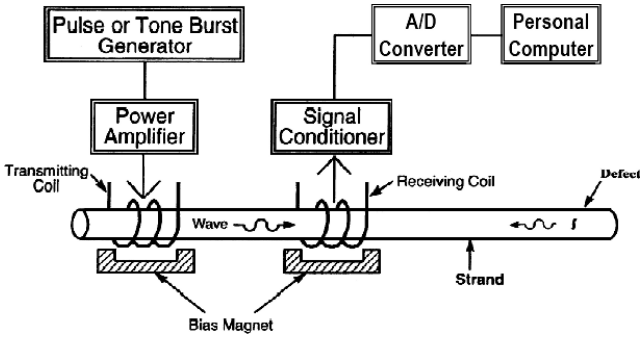

Figure 4. Schematic diagram of MsS and instrument components.

bounded medium (such as rods, pipes and plates) along the direction parallel to the plane of the boundary. With the long-range guided-wave inspection, a pulse of guided-wave typically in frequencies under 100-kHz is launched along a structure, and signals reflected from defects such as corrosion damage or cracks are detected from the same launching position. The location and severity of the defect are determined from the occurrence time of the defect signal and the signal amplitude. Since guided waves can travel a long distance (up to 100 meters or more), a large area can be examined quickly using this inspection method. Guided waves could also be used to examine inaccessible areas by launching waves from an accessible location and detecting, from the launching area, signals coming from inaccessible areas.

The MsS is a probe that generates and detects guided waves electromagnetically in the material under testing based on the magnetostrictive effects present in ferrous materials. For inspection of cables, the MsS consisting simply of a coil that encircles the cable and operated in longitudinal guided-waves is used. A schematic diagram of the MsS and instrument components for generation and detection of guided waves in steel cables or strands is illustrated in Figure 4. A typical field arrangement is shown in Figure 9. For guided wave generation, a short pulse of electric current is supplied to the transmitting MsS coil. The time-varying magnetic field produced by the transmitting coil expands and contracts the cable material underneath the sensor via the magnetostrictive effect, thus generating the guided waves that travel along the individual wires comprising the steel cable in both directions from the coil.

Detection of guided waves is achieved via the reverse process, where the guided waves arriving at the sensor location cause the magnetic induction of the cable material beneath the sensor to change with time. The changing magnetic induction, in turn, induces in the receiving MsS coil an electric voltage through the Faraday effect, which is detected, digitized, and subsequently displayed, recorded and analyzed. To operate the MsS in longitudinal wave mode, a DC bias magnetic field is applied along the length of the cable using either permanent magnets or electromagnets.

In practice, to control the direction of guided-wave transmission and detection, two coils are used and the MsS instrument (MsSR® 2020 or 3030) is equipped with two channel transmitters and receivers for this purpose, Figure 5. The MsS instrument can be operated either in the pulse-echo or the pitch-catch mode. For the former mode of operation, the same coils are used for both transmission and detection. For the latter mode of operation, two separate sets of coils are used; one for transmission and the other for detection.

With the support of the FHWA, the U.S. Department of Transportation, and collaboration from the Port Authority of New York and New Jersey, the MsS technology was first used in the field to assess the suspender ropes on the George Washington Bridge in New York in 1999. From the bridge deck level, the technology was able to examine the entire length of suspenders from the socket end to the main-cable band and provide data that indicate the degree of degradation and their locations in suspenders . Since then, the technology has been used on several highway suspension bridges including the George Washington Bridge to assist the bridge engineers to determine appropriate inspection/maintenance/replacement plans.

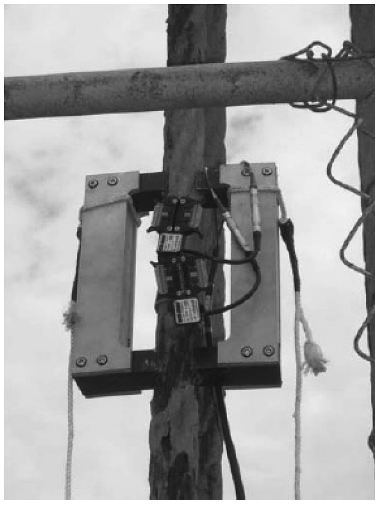

Figure 5. Typical MsS field arrangement.

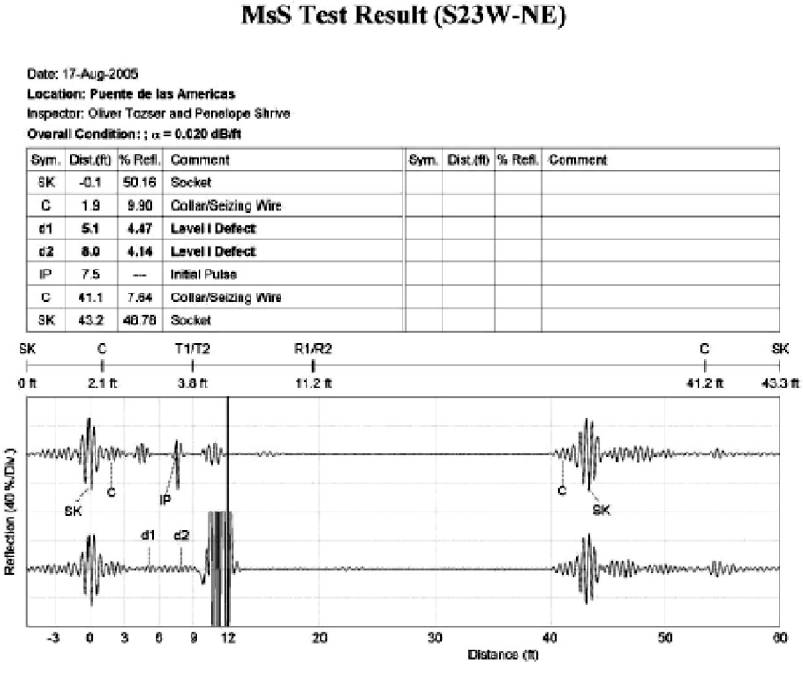

Figure 6. Output from MsS report.

In 2005 the MsS system was used to evaluate all 168 suspender ropes of the Bridge of the Americas, crossing the Panama Canal. This was used in conjunction with 100% hands-on visual inspection of all ropes by Ammann & Whitney's engineers. The MsS technology identified defects that correlated closely with the results of the visual inspection and identified damage in the interior of the ropes that was not visible from the outside. Based on this evaluation recommendations have been made to replace the ropes. Output from the MsS system showing cable damage is shown in "Figure 6 – Output from MsS report."

Figure 7. Ultrasonic sensor at top of bar.

4.1 *Limitations and challenges*

Currently, the MsS technology is capable of detecting anomalies in cables with diameters up to approximately 100 mm. While this includes most suspender ropes or hangers, it precludes the main cables on many suspension bridges, although research is continuing in this area. Furthermore, large reflections generated by cable sockets, gatherers and other discontinuities in the cable may hide the reflections from small defects. However, even in these cases, where periodic inspections of the same cables are carried out, changes in the condition of the cables in these zones may be detectible.

The technology has no significant drawbacks in deployment and operation. Its portability and ease of use facilitates speedy inspection of large structures without the need for extensive supporting infrastructure or traffic control.

5 SLIPPAGE OF SUSPENSION COLLARS

5.1 *Importance of bridge balancing on suspension bridges*

Among the risks of accelerated ageing of a suspension bridge, two important categories arise : Corrosion and fatigue.

Techniques to control the first point have already been presented in the previous paragraphs. Fatigue shall also be considered to increase the lifespan of the structure. This chapter will explain how one of the fatigue mechanisms can be managed. One of the sources of problems creating fatigue is the unbalancing of the loads inside the hangers. During the life of the bridge, cyclic vibration induced by traffic or wind, generate a vibration that can cause a loss in the pre-stressing of the cable clamps. If a sliding of the collars occurs, there is a transfer of load from one hanger to the neighbour ones. An increase of stress in the neighbours hangers can increase dramatically the sensitivity to the fatigue of all the steel elements nearby. A rupture of an anchor plate of a hanger have already been observed in the UK and in Turkey. In this condition it becomes important to guaranty that the fastening of the cable clamps is sufficient to prevent any unwanted slippage.

5.2 *Ultrasonic control of tension in collars bolts*

Many items of equipment are attached to structures by anchorages consisting of short prestressing bars, Figure 7. The safety of such equipment calls for a minimum tensile load being maintained in these bars. The only known and reglementary method of verifying the value of this load is by overstressing the bar and deducing its value from the “unseating” graph of the anchorage. This is the principle of LIFT-OFF.

This article describes the UPUS® system which can determine, during the first and only LIFT-OFF, an ultrasonic characteristic of the bar which, when associated with the LIFT-OFF results,

Figure 8. Measurement on Chavanon suspension bridge.

then allows one to calculate the load by simple measurement of a time of propagation of an ultrasound from one end of the bar to the other. The initial measurements as well as those taken afterwards are automatically processed for calculation of the required values, not only to provide a written report on the operation but also for analysing the behaviour during the initial tension. The computer which processes these data and measurements also contains the database of the bars.

Many tests on different configurations of bars have shown that the initial incertitude with regard to the determination of the force is less than 2% when using the curve force/displacement and less than 1% when using the curve force/variation in time. Then, determination of the force by simple measurement of time leads to an incertitude of under 4%, which is close to the common value of 5% admissible with a traditional LIFT-OFF. Figure 8 shows measurements on the Chevanon suspension bridge.

5.3 *Control of bridge balancing*

Techniques have also been developed to control from time to time the tension into the hangers. The starting point is the typical lift-off, but when applied on big suspension bridges, it becomes a problem, as accesses for hydraulics is difficult.

Advitam has developed several techniques on case by case situations to perform this type of investigations.

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Chapter 9

Non-destructive testing of suspender ropes with magnetostriction

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ABSTRACT: Suspension bridges and through arch bridges with suspended decks often use suspender ropes that must be periodically inspected and assessed. Research and testing has shown that corrosion damage typically starts and is often most extensive on the interior of the rope where the damage cannot be observed with conventional visual inspections. Thus a visual inspection of the exterior of the rope does not necessarily provide a representative assessment of the actual conditions of the rope.

This paper describes a non-destructive testing (NDT) technique used on suspender ropes to detect either cross section loss or broken wires located on either the interior or exterior of a rope. The technology relies on the principles of magnetostriction to generate a guided acoustic pulse that travels up and down the rope. Where there is a defect in the rope resulting from corrosion or fatigue damage, a portion of the pulse is reflected back to the source. These reflections can be measured and quantified to determine the relative size and location of corrosion or fatigue damage on a suspender rope. This technique does NOT use magnetic flux principles, which has been used on suspender ropes in the past with limited success.

This paper discusses the principles of using magnetostriction to test the condition of suspender ropes and the capabilities and limitations of such an approach.

1 CHALLENGES ASSOCIATED WITH INSPECTION OF SUSPENDER ROPES

Suspender ropes are important structural members for suspension bridges and through arch bridges with suspended decks. Like, all structural elements, they must be periodically inspected and assessed, but it is challenging for bridge inspectors to obtain a realistic condition assessment of these ropes. Conventional inspections typically rely on visual inspections of the exterior of the rope to document visual evidence of damage (e.g. broken wires, corrosion product) resulting from corrosion or fatigue problems. However, research and testing has shown that inspection of the exterior of the rope does not necessarily correlate with the overall condition of the rope. Corrosion damage is often initiated and most extensive on the interior rope, thus it may not be apparent during a visual inspection.

To obtain a more comprehensive assessment of a suspender rope system, bridge owners may remove some of the ropes from the structure and subject them to comprehensive laboratory testing. This provides a detailed, comprehensive analysis for the ropes that were removed, but the overall sampling percentage is usually less than 5% of the number of ropes. Therefore, extending the laboratory results from this sample size the remainder of the ropes on a bridge, may provide misleading results.

Another challenge when assessing ropes, is that performing a close-up visual inspection is often logistically challenging due to the long lengths of ropes. Moving inspection personnel and equipment up and down ropes that may exceed 300 feet is often not practical.

Corrosion of suspender ropes leads to broken wires and loss of wire cross section. Ideally, a proper assessment of a rope will detect and report locations with this type of damage on the interior and the exterior of a rope. The magnetostriction NDT technique can provide inspection personnel with important data to compliment existing inspection techniques so that more comprehensive inspection programs are deployed.

2 PRINCIPLES OF MAGNETOSTRICTION

Magnetostriction is a characteristic of all ferromagnetic materials to undergo a small change in physical dimension when subjected to a magnetic field. Although, the change is small, on the order of several parts per million in carbon steel, the phenomenon can be used to non-destructively test the condition of various ferromagnetic objects. By inducing a controlled dynamic magnetic field around a steel object, such as a suspender rope, an acoustic pulse can be generated that propagates through the steel.

The acoustic pulse consists of structure-borne elastic waves, called guided waves, which propagate along the structure and are confined by the geometry of the structure. Since steel is an excellent acoustic medium, the waves travel for long distances at relatively high speeds, more than three miles per second.

When using guided waves for non-destructive testing of steel objects, relatively low frequency (under 200 kHz) guided waves can be launched into the structure from a convenient, fixed location of the structure. The guided waves will propagate along the structure and where it encounters a cross sectional variation (e.g. broken wires or general wire cross section loss), a portion of the waves will be reflected toward the source. These reflections can be detected and measured through the inverse magnetostrictive effect with the same sensor (Kwun 1991 & 2003).

By identifying the reflections, defects in a rope can be detected and by measuring the size of a reflection, the relative size of the defect can be determined.

3 USING MAGNETOSTRICTION TO TEST SUSPENDER ROPES

When using magnetostriction to test the condition of suspender ropes there are four general steps:

1. Generating a guided acoustic pulse that travels along the suspender rope
2. Allowing the pulse to travel the rope and be reflected by damage
3. Measuring and recording data on reflections
4. Evaluating data to determine the position and amount of damage

Generating an acoustic pulse: By generating and altering a dynamic magnetic field around a steel suspender rope in a controlled fashion, precise guided waves are created which travel the suspender rope. The magnetic field is generated by placing a ribbon coil around the rope and clamping a magnet to the rope. The inspection can be performed at any convenient location along the rope and only requires access at one location along the rope. The sensor arrangement is shown in Figure 1.

Allowing the pulse to travel the rope: Steel is an excellent acoustic medium. The acoustic pulse travels along a suspender rope at approximately 13,000 feet per second. Where the pulse encounters damage in the form of broken wires or reduced cross section, a portion of the pulse will be reflected and will travel back toward the source. Reflections are also generated by objects in contact with the rope (e.g. spreaders, gathers, collars, etc.).

Measuring and recording data: By reversing the process used to generate the acoustic pulse, through the inverse-magnetostrictive or Villari effect, the presence and quantity of reflections can be measured and recorded. This measurement is performed at the same location as where the pulse was generated.

Evaluating data: By evaluating the size and pattern of reflections compared to the initial pulse, both the position and relative amount of damage can be estimated for the cable.

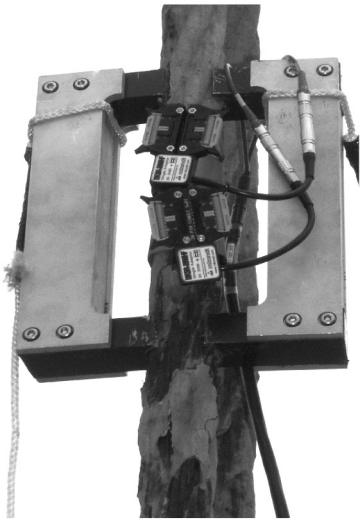

Figure 1. Magnetostrictive sensor consisting of two ribbon coils and two magnets mounted on a steel suspender rope.

Figure 2. Inspection of George Washington bridge suspender ropes.

4 REPRESENTATIVE DATA

When using the above techniques on suspender ropes, the data is recorded on a data acquisition system while the field work is being performed and later evaluated to develop the final results. Magnetostriction has been used to test suspender ropes in laboratories and on operational bridges. Using magnetostriction as a NDT tool was originally developed for steel pipelines, but through a research program sponsored by the Federal Highway Administration and Port Authority of New York and New Jersey, it was successfully extended to suspender ropes, initially on the George Washington Bridge (Khazem et al) and subsequently other bridges.

Typical data obtained from an operational bridge is shown in Figure 3. The plots show signal amplitude vs. distance above the bottom socket for four different ropes, numbered from top to bottom as Rope 1 to 4 (Pure Technologies 2005). Reflection coefficients are calculated from these plots. These coefficients are compensated to account for wave attenuation to permit a fair comparison between reflection coefficients at varying distances from the signal generator.

By reviewing the data from Rope 1, a typical plot for a rope in good condition can be observed. Large signals were recorded from the initial signal input and from reflections by the top and bottom sockets. However, the remainder of the rope did not have any reflection coefficients greater than 1.5%. Reflection coefficients in excess of 1.5% are typically reported as potential locations for defects in a rope. The large signal obtained from the signal input at approximately 9 to 12 feet, means that the data in this region can not be effectively evaluated to detect defects. Thus with the arrangement of sensors used for this test there is blind zone between 9 and 12 feet. However, by moving the sensors and performing a second test, the blind zone can be shifted to another location. Combining the results from these two tests, enables the successful testing and evaluation of the full length of a rope. Thus, the full lengths of ropes can be tested. This combination of test results also permits the accurate reliable identification of objects in contact with the rope (e.g. separators, collars). Furthermore, it allows for the accurate identification of signal noise so that it is not reported as damage. For instance, on Rope 1 there is a reflection at 20 feet; however, the reflection did not show up on the second test (data not shown) and therefore it was determined that the reflection was the result of noise, not damage on the rope.

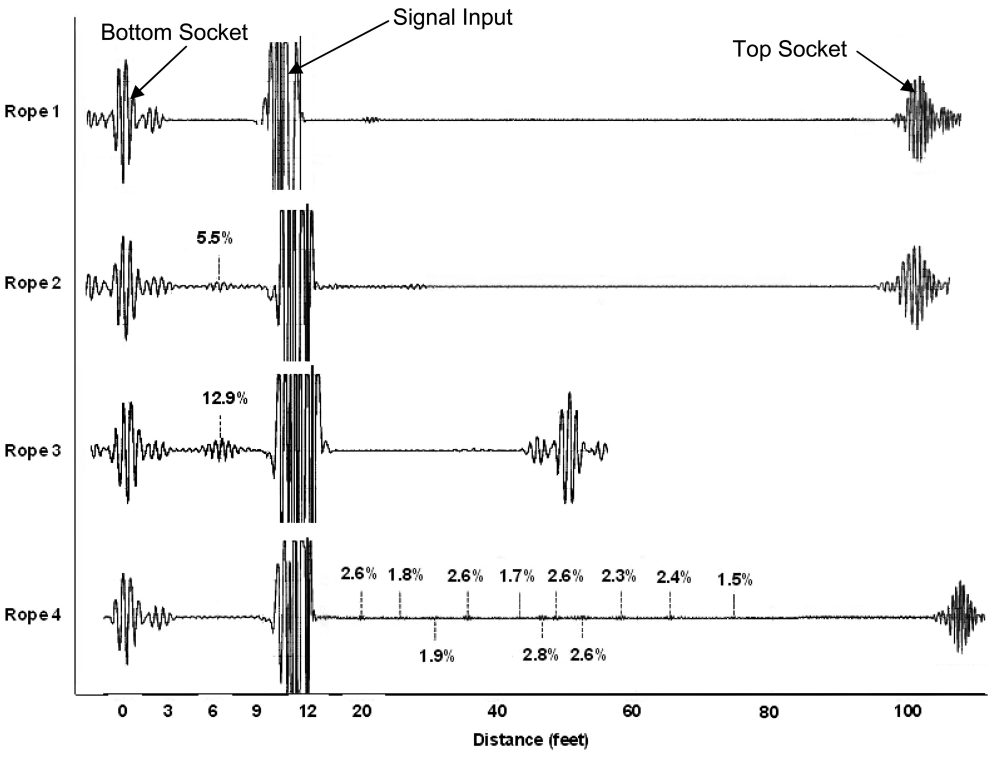

Figure 3. Reflection coefficient vs. distance above bottom socket data for four ropes from an operational bridge.

Rope 2 shows an example of a rope with a reflection coefficient of 5.5% at approximately 6 feet above the bottom socket. There are no other reported defects on this rope. Rope 3 shows a defect with a reflection coefficient equal to 12.9%.

Rope 4 shows an example of a rope that has several defect locations along the rope between 20 and 80 feet above the bottom socket. All of the reflections were small, between 1.5% and 2.8%.

5 LIMITATIONS

The use of magnetostriction to test the condition of suspender ropes has one limitation worthy of noting. When there are objects in contact with the rope, data analysis is more complicated. Objects in direct contact with the rope (e.g. split collars, sockets, gatherers) will produce a significant reflection. These reflections are easily identified and are not reported as damage; however, evaluating the data in this region is more challenging than in the free length of the rope. Anomalous regions can be detected in these areas by comparing the signals for the objects on multiple ropes to each other (i.e. comparing all the reflections from gathers tested), but the reliability of this approach is unknown. This way if there is damage near a gather, it may be identified as anomalous.

Another way to address this limitation is to perform multiple tests over a significant period of time (e.g. several years). By comparing reflections from the same object over multiple years, it can be determined if the reflection is consistent or has changed in value. If the reflection for an object in contact with the rope has changed, this is a sign of corrosion or fatigue damage.

Given this limitation, it is recommended that if these are areas of particular concern, visual inspections should be performed. Alternatively, some objects could be temporarily removed while the test is performed or loosened and pulled away from the rope.

6 CONCLUSION

The principles of magnetostriction have been used to non-destructively test the condition of suspender ropes in both controlled environments and on operational structures. The technique provides cost effective evaluation technique of suspender ropes in situ without the need for expensive or intrusive techniques. The equipment required to test the ropes is relatively lightweight and easy to move in the field and reduces the logistical challenges associated with assessing ropes.

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Chapter 10

Wire test results for three suspension bridge cables

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ABSTRACT: The evaluation of the cables on a recently inspected suspension bridge provided an opportunity to rigorously test 80 wires removed from the cable. Full length stress-strain curves for the samples were developed. These curves were found to vary from each other by as much as 13%, and calculations were made to determine the effect on the estimated cable strength of this variation. It was found that the simple Brittle-Wire Model underestimates the cable strength by approximately 3%. The tensile strengths of the wires on this bridge are compared with those of two formerly inspected bridges, and the cable strengths using these data in the Brittle-Wire Model are calculated. The variation in the stress-strain curves was found to be caused by variations in the carbon contents of the wires.

1 INTRODUCTION

The recent internal inspection of the cables of a major suspension bridge provided an opportunity to apply the recently published Guidelines for Inspection and Strength Evaluation of Suspension Bridge Parallel Wire Cables (the "Guidelines"), issued by the National Cooperative Highway Research Program in November, 2004 (Mayrbaurl and Camo, 2004), for the first time. This internal inspection, as have all such inspections in the past, involved unwrapping the cables and driving wedges into them to enable to visually observe and record the conditions of the wires inside. The condition of the wires was categorized into four stages of corrosion, as follows:

- Stage 1 Spots of zinc oxidation on the wire, a virtually new wire
- Stage 2 Zinc oxidation over most of the wire surface up to the entire surface, no ferrous rust
- Stage 3 Spots of brown ferrous rust covering up to 30% of the wire surface, over lengths from 25 mm up to the entire length of the wire
- Stage 4 Ferrous rust covering over 30% of the wire surface over the same lengths

Broken wires were also counted and located. Six meter lengths of wires were removed from the cable for testing to determine their strength along with observing whether they are cracked at the fracture surface. These cracks often are present in Stage 4 wires and are driven by hydrogen (hydrogen assisted cracking) that is generated by the galvanic process between the zinc coating on the wire and the iron in the wire (Yahalom and Aladjem, 1980, Latanision, 2000). Tests for the thickness of the zinc coating and its continuity were made on Stage 1 and 2 wires as well. Although the chemical content of the wires was specified and tested during manufacture, these tests are also often performed to verify the quality of the wire, especially in older suspension bridges, and were performed on several the wires removed during this inspection.

The results of the tensile tests on the wires of this bridge are of special interest, because a special effort was made to determine the shape of the stress-strain curve of each wire sample during the tests, and the number of specimens cut from each sample for testing was as specified in the Guidelines, though only 80 samples were removed from the cable because only 10 panels of the cable were unwrapped.

Why are wires removed from a suspension bridge cable tested? The simple answer is to determine the cable strength. The only strength that can be determined (other than the original design strength) is the current strength, the strength at the time at which the cable was inspected. Before discussing the wire tests, how the strength of the cable is calculated should be described.

2 ESTIMATING CABLE STRENGTH

2.1 *Early strength models*

In past inspections, only very small parts of the interior of a cable were inspected, because only very short lengths of the wrapping wire on the outside of the cable were removed. If the wires looked new, or only a little deteriorated, and no broken wires were found, the cable was pronounced good, and no effort was made to estimate the cable strength except to say that the cable had its original or only slightly less than its original strength. This was true even when a few broken wires were found. When the corrosion looked more serious, it was recommended that further inspections should be made, with additional wrapping wire removed. In the case of the Mid-Hudson Bridge, this led to recommendations that the cables be treated with a corrosion inhibitor, and a study of possible cable replacement. The cable strength on this bridge was estimated by multiplying the area of the "remaining" wires (after one layer of wires at the outer surface of the cable plus two additional layers at the bottom of the cable were deducted) by the smallest wire strength found in the tests. This was a suitable method, because the original safety factor for the cable was about 4, and the reduced cable strength by this simplified method was still greater than the design safety factor used in newer cables (Modjeski and Masters, 1991).

On the Williamsburg Bridge, the wire deterioration took the form of loss of section, and because a test on a strand made from deteriorated wires taken from the anchorage exhibited signs of ductility on failure, it was felt that a ductile strength model should be applied to the cable, in which all wires can stretch until they all break at once. In this model, the remaining cross section of the wires is multiplied by the average tensile strength found in testing (Steinman, et al., 1988). This model must be approached with extreme care, as it also assumes that each wire has a constant area over its length, and can actually elongate as much as an un-corroded wire. There are some test results that indicate that this is not the case, even in a new cable. Tests on parallel wire strands made during the design process for the Bear Mountain Bridge (Roebling, 1925) and the Benjamin Franklin Bridge (Board of Engineers, 1927), both in the 1920s, showed that the strength of these strands were 4 to 6% smaller than the area multiplied by the average tensile strength of the wires in the strands (i.e., the strands were weaker than the sum of the strengths of the individual wires in the strand). The remaining 94 to 96% of the sum of the individual wire strengths was referred to as the "efficiency" of the parallel wire cable. The reduction in strength was ascribed to the impossibility of making the tension of all the wires equal when placed into a strand. This is not a likely reason, as the total stretch of the wires to failure is 6 to 7%, and when a bundle of wires is tested, wires will usually start to fail before the entire bundle fails. The reason for this early failure is that not all wires can elongate to the same extent.

The Williamsburg Bridge investigation is mentioned here because it included a rigorous testing program (Steinman, et al., 1988, Matteo, et al., 1994) that showed that more than a few simple tests are needed to find what is needed to estimate the cable strength. As a specific example, when large variations in the tensile strengths of wires that seemed the same were found, a chemical testing program was instituted and it was found that the carbon content of the wires varied from 0.6% to 1.0%, a very large variation that would not be expected on a more modern bridge. Unfortunately, not all the findings are applicable to most suspension bridges because, unlike most suspension bridges, the wires of the Williamsburg Bridge were not zinc coated, a very significant difference. With the exception of wires that were in contact with zinc coated wrapping wires that were added long after the bridge was finished, the wires on this bridge failed because of loss of section, while galvanized wires inside a cable almost always fail by breaking without accompanying section loss. They develop cracks that lead to strength loss and resulting failure under dead load.

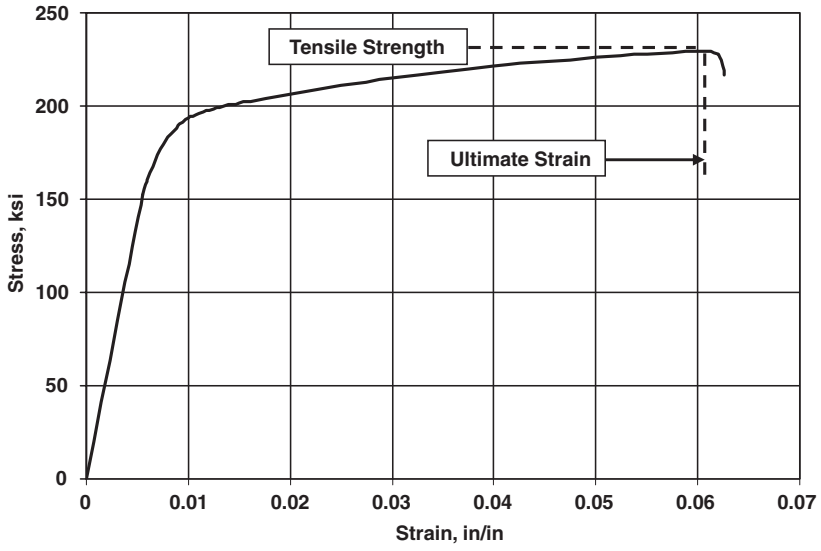

Figure 1. Stress-strain curve for the Bear Mountain Bridge, average of 126 wire tests.

The varied techniques used in inspecting suspension bridge cables and testing the wires led to the development of the Guidelines in order to standardize cable inspection and evaluation.

2.2 *Cable strength models in the guidelines*

Included in the Guidelines are models for estimating the current strength of a cable composed of many parallel wires. These models utilize the stress-strain curve of bridge wire. The average stress-strain diagram of the Bear Mountain Bridge wires, derived from 126 wire tests, is shown in Figure 1 (Roebling, 1925). The diagram shows that there is no yield plateau in bridge wire; the stress increases monotonically through the proportional limit and the wire immediately goes into strain hardening. For every value of strain, there is a discrete value of stress. Thus, if a cable were composed of many wires that followed such a stress-strain curve, some of which had a failure strain lower than others, these wires would break before the others if the strain in the cable were to be gradually increased.

A statistical distribution, such as the Weibull distribution shown in Figure 2, describes how a property, such as failure strain, varies among a large population of objects, such as wires. The value of the distribution at any specific value of strain, *e*, describes what fraction of the entire population has that particular failure strain (i.e., how much of the population would fail at that strain). The cumulative area under the distribution curve up to a given strain describes the fraction of the entire population that has a failure strain equal to or smaller than that specific strain. This is the same as the fraction of the entire population that would have failed if the strain were to be gradually increased up to that specific strain. The remaining portion of the distribution is known as the survivor function, and it describes the fraction of the entire population that consists of unbroken wires at the specific value of strain. Multiplying the area of these unbroken wires by the average tensile stress at the specific strain results in the force in the cable at that strain. If the strain were to continue to be increased, additional wires would break, and the force in the cable would reach a maximum value and then begin to grow smaller as the strain continues to be increased. This maximum value is the cable strength. This procedure of incrementing the strain and calculating the cable force at each increment until a maximum is found is called the Limited Ductility Cable Strength Model (shortened to Limited Ductility Model).

This model requires that the failure strain be found for each wire tested. Testing laboratories do not like to leave an expensive extensometer attached to a wire to failure for two reasons; the first

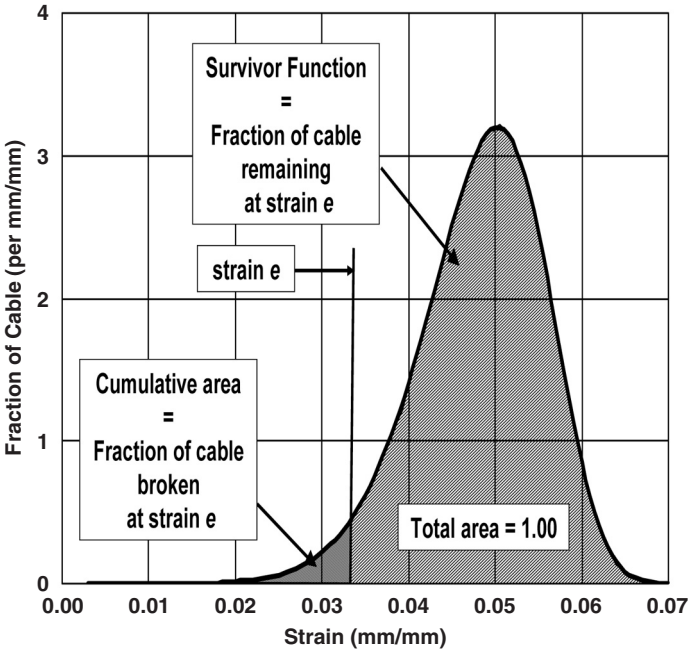

Figure 2. Distribution curve (Weibull distribution is shown).

is that the extensometer will often fall off the specimen and break when the specimen fails; the second is that the strain rate is usually increased after sufficient data is obtained to calculate the 0.2% yield stress, shortening the test and reducing the cost. The model also requires an accurate average stress-strain curve, which cannot be obtained if the extensometer is removed.

A simplification of this model assumes that all wires follow the same stress-strain curve, and that all have equal area. In this case, the wires would fail in the order of their tensile strength, and the strain is not needed in the calculation. Only a statistical distribution for the tensile strength of the wires is needed; this is more easily developed than that for ultimate strain. This reasoning describes the Brittle-Wire Model.

3 RESULTS OF TENSILE TESTS ON BRIDGE WIRES

Three sets of wire tests were presented in the Guidelines, identified by the letters X, Y and Z. The tests on wires from Bridge Y were made primarily on wires found broken in the cable. In another set of tests (Bridge Z) wires were cut into specimens and the tests results reported in a combined table from which it was difficult to separate out the wire samples. The third set (Bridge X) reported each specimen, which was numbered by the number of the sample that it came from plus a specimen number. In this third set of tests, the samples sometimes had two standard 18″ specimens cut, sometimes one standard specimen plus one long specimen and sometimes (in a few cases) into many standard specimens. A test program done by Columbia University on the Williamsburg Bridge cable wires showed the right way to do it, cutting a sample into many specimens and randomly selecting some of them to test (Matteo, et al., 1994). The wires on that bridge, however, were not galvanized and did not exhibit cracks, except possibly in a few exceptions.

During the testing of wires from Bridge X, the extensometer remained on the wire to a strain of about 2% The elongation of the specimen over a 254 mm gage length was measured after the specimen was removed from the testing machine. To this elongation was added the elastic strain in the specimen up to the failure stress to estimate the failure strain. It should be noted that the modulus

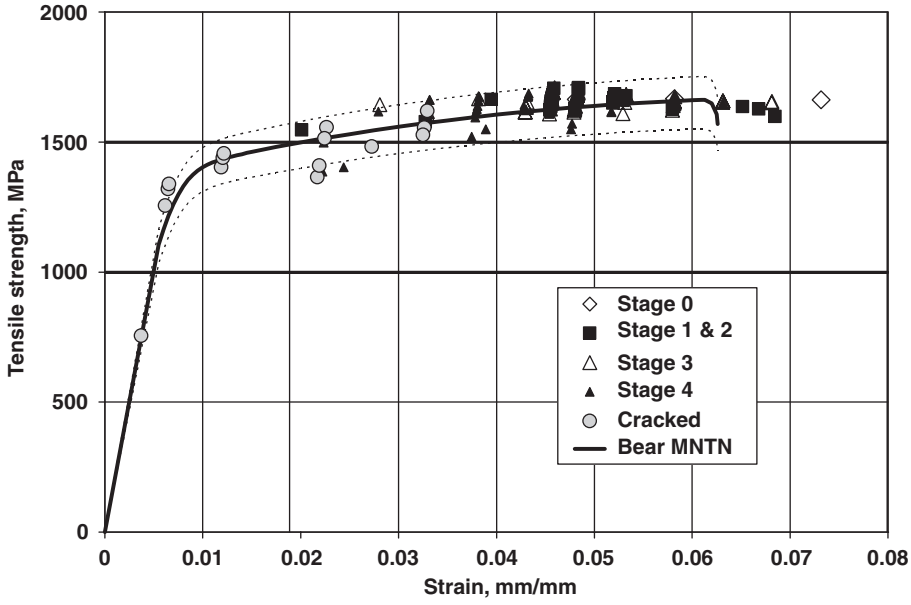

Figure 3. Tensile strength of Bridge X wires plotted against the ultimate strain, with Bear Mountain average stress-strain curve multiplied by constants shown as an average curve and upper and lower bound envelopes.

of elasticity is smaller after the wire has been stretched to failure; this effect was not taken into account in the calculation. The additional stretch induced by necking down also was not accounted for. Thus, this technique of estimating failure strains is not suitable for determining the parameters needed for the Limited Ductility Model. A plot of failure stress vs. strain thus determined is shown in Figure 3, along with a plot of the Bear Mountain average stress-strain curve multiplied by a constant to make it approximately equal to the average values for Bridge X. The individual stress-strain points for individual Bridge X wires on this diagram lie above and below this "average" curve, with a total variation of about 11%. From what was found in the Williamsburg Bridge investigation, this is most likely due to variations in carbon content of the wires. The dashed lines in Figure 3 are also the Bear Mountain Bridge stress-strain curve modified to approximate an envelope of values of the bridge X data. The similarity of shape between the Bridge X data and the Bear Mountain Bridge average curve is striking.

On Bridge W, where the technique recommended in the Guidelines was first applied, the opportunity presented itself to develop full length stress-strain curves for each of 80 sample wires. Tension tests were made in accordance with ASTM A370 and A586 (the latter requires that a gage length of 10 inches be used for elongation measurement after failure), modified by the use of a 200 mm long LVDT for an extensometer. Eight 6 meter long sample wires were removed from each of the 10 panels of the cables that were unwrapped for inspection. Ten 457 mm long specimens were cut from each sample wire; four randomly selected specimens from wires that were Stage 1 or 2 were subjected to tension tests and all 10 from sample wires that were Stage 3 or 4 were tested. The difference in the number of specimens tested is because Stages 3 and 4 are expected to have a higher standard deviation in tensile strength than Stages 1 and 2, and a larger number of specimens is required to result in the same accuracy of results, and because cracks are possible in Stages 3 and 4 and the entire length of the sample must be searched for cracks; often only one is found in the entire sample length.

One specimen from each sample was tested with the extensometer attached until the specimen failed. At the start of testing, the last wire tested from each sample was used for this purpose. After two of these specimens failed at cracks, resulting in very short stress-strain curves, the procedure

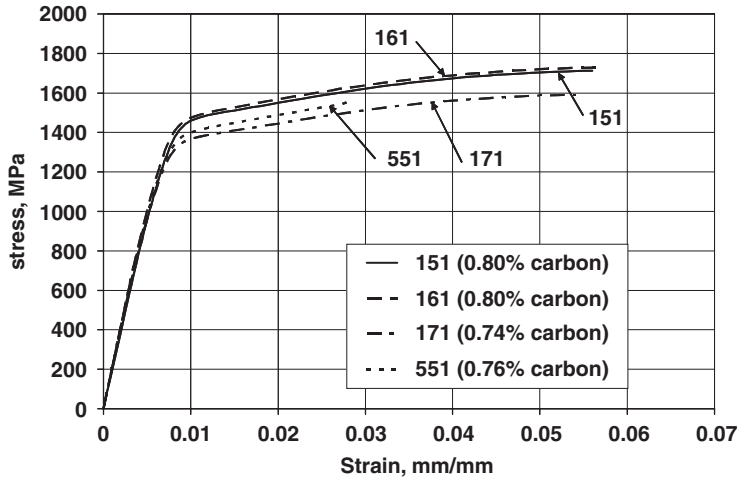

Figure 4. Selected stress-strain curves from Bridge W wires, showing variation and carbon content of steel.

was reversed to test the first specimen in this manner, and making a second test if the first specimen failed at a crack. The stress-strain curves up to 2.7% (at which point the extensometers were removed) were substituted for the wires that failed early.

The data were acquired digitally and reduced to the desired stresses at 110 specific values of strain for each sample. Because the thickness of the zinc coating varies, and the zinc itself adds virtually no strength to the galvanized wire, the net steel areas of the wires were used in these analyses. This eliminates variability caused by zinc loss in many of the wires. A short specimen was cut from each end of the sample wire and the zinc was removed by dipping in acid. The diameter of each was measured in two mutually perpendicular directions and averaged. This average was rounded to two decimal places and the area of the specimen calculated from this rounded average diameter. The tensile strength of each wire was calculated using these calculated areas, and a stress-strain curve plotted.

Early in the investigation, it became obvious that the stress-strain curves were not all alike. Figure 4 shows four of these curves that clearly demonstrate their variability. It was suspected that the variation is caused by differing carbon content of the wires, and the samples from which these curves had been developed were sent for chemical analysis. The resulting carbon contents of the samples are also shown in Figure 4, and it was concluded that the variation in carbon content is indeed the cause of the varying stress-strain curves. The original specification for the wires used in this cable stated that the carbon content should be at least 0.75% and not more than 0.80%. There was a concern that the carbon content of the highest curves was 0.80%, and that it was already known that some sample wires were stronger than those corresponding to the upper stress-strain curve.

To enable the comparison of all the stress-strain curves easily, a characteristic value was needed for each curve. The stress at a strain of 0.025 mm/mm was selected as the characteristic value in this case to enable inclusion of all 80 stress-strain curves, as two of the curves extend only to a strain of 0.027. All 80 characteristic values are plotted in Figure 5, in order of increasing value. Ten additional samples for chemical testing were selected at random from this curve, with two at the highest strength samples and another at the lowest, adjacent to the wire on which a chemical test already had been made. The results of these ten additional tests plus the four existing tests are plotted on the abscissa of Figure 6, with the characteristic stresses of the corresponding samples as the ordinate. Also shown are the corresponding tensile strengths of the samples. Both the characteristic values and the tensile strengths are seen to vary nearly linearly with the carbon content. The maximum carbon content found is 0.85%, above the limit specified.

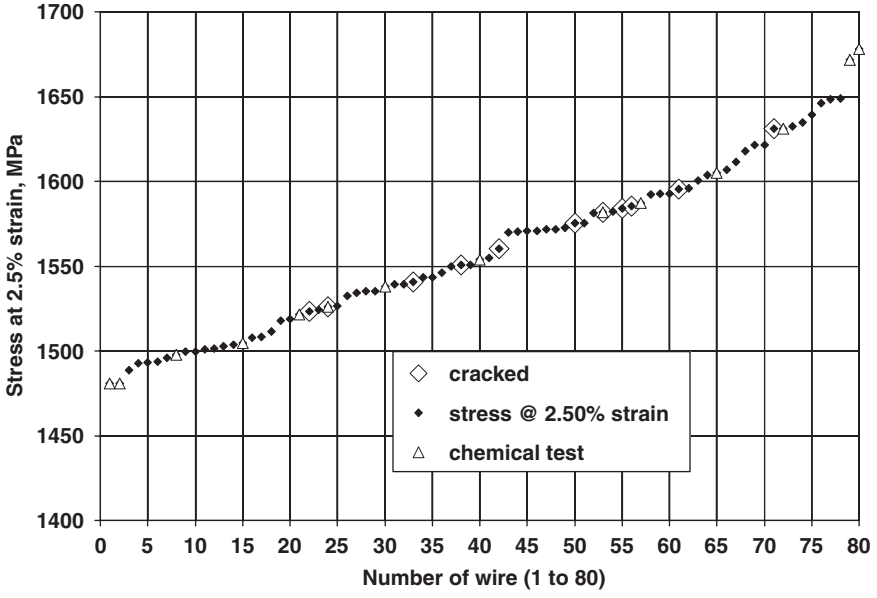

Figure 5. Characteristic stresses of sample wires from Bridge W. The characteristic stress is the stress at 2.5% strain.

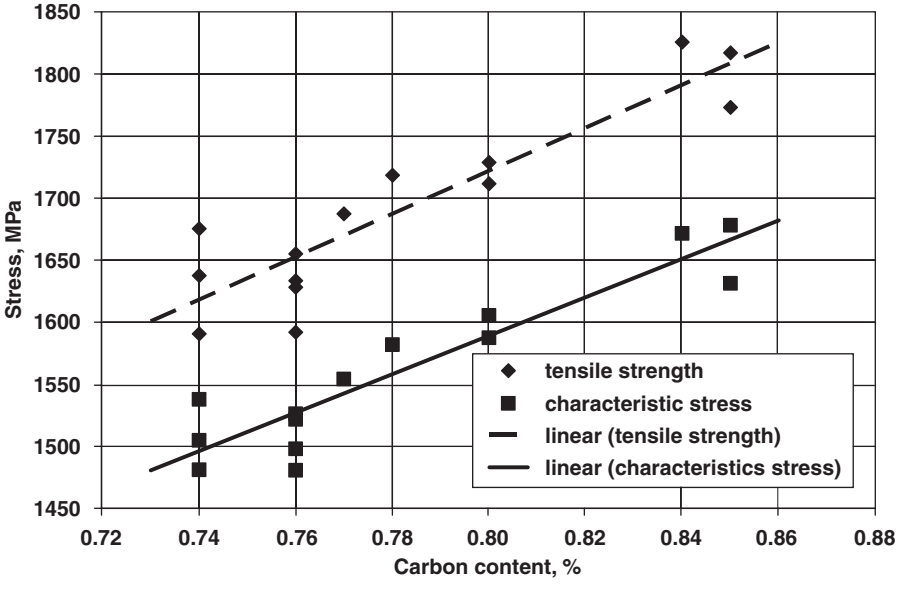

Figure 6. Characteristic stress and tensile strength vs. carbon content of 14 wires from Bridge W.

The stresses in all wires at each value of strain were averaged to determine the average stress-strain curve for all 80 wires. This average is shown in Figure 7. As the strain increases, the number of sample wires remaining in the average grows smaller, and the curve develops some irregularity at its end. For most of the length, however, the average curve is fairly smooth with only

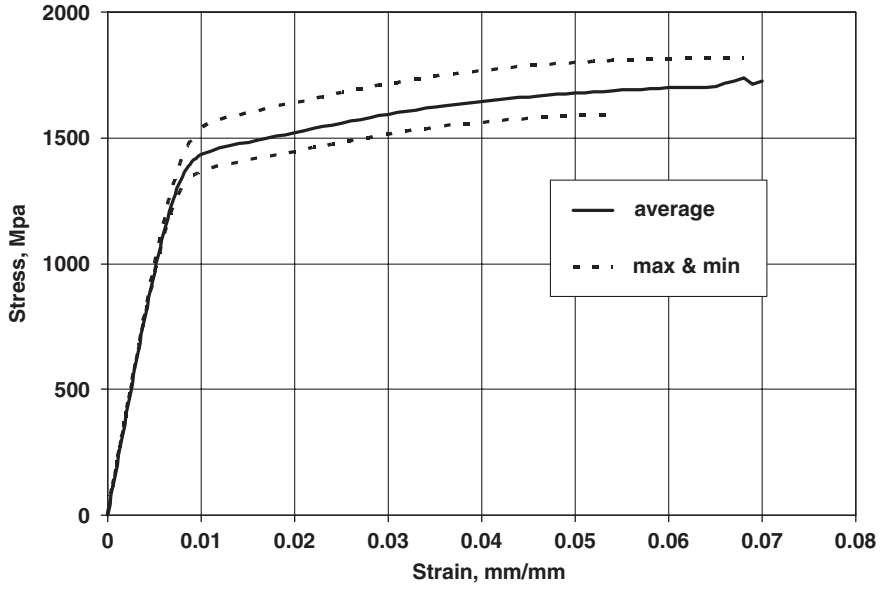

Figure 7. Average stress-strain diagram for Bridge W (80 wires). The maximum and minimum stress-strain curves are also shown, in dashed lines.

small irregularities that are inherent in the data acquisition system. To smooth the curve, the curve fitting function in Excel was used to fit one straight line and five parabolas to the data. This curve is not distinguishable from the average curve in Figure 7 except at the upper end, and is not plotted. The highest and lowest stress-strain curves are also shown in Figure 7. These curves are 7.6% higher and 5.0% lower than the average (at 2.5% strain), a total range of 12.6%. This is only slightly wider than the data shown for Bridge X in Figure 3, where the difference is about 11%. It is thus suspected that many suspension bridge cables may also have larger than specified carbon content variation and variations in stress-strain curves similar to those found in these two bridges. Unfortunately, there is not sufficient data available to determine the variation that may be present in Bridge Z.

The Guidelines recommends the use of the nominal net steel wire area as a constant for the calculation of the tensile strength of specimens for use in cable strength estimates. This was done for Bridges X and Z. On Bridge W, the actual average area of all the sample wires was used for this purpose. The exact value used for this area, which is a constant for all wires, does not affect the estimated cable strength when using the Brittle-Wire Model, since the same area is used in the strength calculation. This has the effect of using the wire breaking load to determine the cable strength.

Table 1 gives the normalized mean values of the tensile strengths for each stage of corrosion thus calculated for Bridges X, Z and W. The tensile strengths are normalized on the tensile strength of a new wire for easier comparison between bridges. The table also gives the coefficient of variation, which is the standard deviation divided by the mean, for each stage, and the fraction of Stage 4 specimens found to be cracked.

In this table, Group 2 wires consist of Stage 1 plus Stage 2 wires, which have been found to have the same or very nearly the same tensile strengths. Groups 3 and 4 are Stage 3 or 4 wires that do not have cracks, while Group 5 consists of all the cracked wires found in testing.

Only 3 wires that could be classified as new were tested from Bridge W; the mean tensile strength of these was smaller than the mean for Stage 3 wires; the properties of Stages 1, 2 & 3 were nearly the same, and the average of all 3 was used for new, Group 2 and Group 3 wires in evaluation of the cable.

Table 1. Mean tensile strengths of wires in three bridges.

	Bridge W			Bridge X			Bridge Z		
	No. of samples	Mean tensile strength	Coeff. of variat'n	No. of samples	Mean tensile strength	Coeff. of variat'n	No. of samples	Mean tensile strength	Coeff. of variat'n
Tensile strength of new wire		1695 MPa			1657 MPa			1644 MPa	
New wire	3	1.000	0.026	2	1.000	0.005	20	1.000	0.021
Group 2	17	1.000	0.026	30	0.994	0.018	29	0.986	0.029
Group 3	17	1.000	0.026	18	0.981	0.024	22	0.945	0.039
Group 4	35	0.980	0.038	10	0.961	0.038	33	0.944	0.041
Grp 5 (crkd)	11	0.890	0.128	15	0.834	0.131	6	0.696	0.263
Fraction of Stage 4 crkd		0.24			0.64			0.60	

For Bridge X, the smallest tensile strength of at least two specimens is taken to be the tensile strength of the sample. The data for Bridge Z was not segregated into samples and all specimens are included as though they were separate samples. On Bridge W, a statistical probable minimum value of tensile strength in a panel length as described in the Guidelines was calculated for each sample wire. For cracked wires in Bridges X and W, the smallest tensile strength found in a sample is used as the minimum in a panel length since there are not enough cracked wires in a given sample to develop a minimum statistical value. An effort was made to find which specimens belong to which samples of Stage 4 wires on Bridge Z to estimate the fraction of Stage 4 wires that are cracked, and the lowest tensile strength for each cracked sample was used.

The fraction of Stage 4 wires that are cracked for Bridge X was determined from Stage 4 samples that were either tested as many specimens or that were tested as one long plus one short wire. Samples from which only one or two short specimens were tested could not be used for this purpose because it was not known whether or not the remaining length of the sample contained a crack. The fraction of wires cracked was not determined by using the number of sample wires included in the table. On Bridge Z, the fraction cracked was estimated by grouping Stage 4 wires into sample wires and using that data for this purpose. The mean strength of Group 4 was determined by averaging all specimens in this case. The data for Bridge W was carefully grouped by sample and all samples were tested in the same manner, thus the fraction of Stage 4 wires that are cracked corresponds with the number of samples reported in the table for Groups 4 and 5. In evaluating a cable, the use of the testing technique that was used for Bridge W is highly recommended.

4 CABLE STRENGTHS CALCULATED USING THE DATA FROM 3 BRIDGES

To demonstrate the effect that various stages of corrosion have on cable strength, a cable consisting entirely of Group 2 wires, in which the wires are subjected only to deterioration of the zinc coating, and which has virtually the same strength as a new cable is compared with one that contains Group 3 and Group 4 wires that are not cracked, as well as one that also contains the same proportion of Groups 3 and 4, but in which some of the Stage 4 wires are cracked. In the latter case, the Group 4 wires are reduced, to be replaced by a number of Group 5 (cracked) wires. The proportions of each corrosion group are given in Table 2. The sample cable contains 9,990 wires. The proportions of each group in the cable are the same as those that were used in the Guidelines for parametric studies.

Only the first and last rows in Table 2 are used in the current study.

Table 2. Proportions of corrosion stages.

Description deterioration	Condition	Corrosion group 2	3	4
Virtually new	0	1.00	0.00	0.00
Slight	A	0.84	0.13	0.03
Moderate	B	0.60	0.26	0.14
Heavy	C	0.23	0.55	0.22

Table 3. Cable strengths for various combinations of deterioration and mean tensile strengths.

	Cable W		Cable X		Cable Y	
Actual % cracks	24		64		60	
Condition	Strength (kN)	Percent loss	Strength (kN)	Percent loss	Strength (kN)	Percent loss
0	294,000	0	292,000	0	279,000	0
C (no cracks)	285,000	3	274,000	6	258,000	8
C (50% cracks)	261,000	11	244,000	16	224,000	20
C (actual % cracks)	270,000	8	241,000	17	221,000	21

To show the effects of the tensile strength distributions of the various groups, a uniform 50% rate of cracks in Stage 4 is first used; to show the effects of varying the rate of cracking, the actual rate of cracks in Stage 4 for each bridge is used. The actual mean tensile strengths and standard deviations for each of the bridges on which Table 1 are based are used in this demonstration. The Brittle-Wire Model given in the Guidelines is used in these calculations. The Normal distribution is used for wires in Group 0, rather than the Weibull distribution that is specified in the Guidelines, because these wires act like new wires, and when the wires are manufactured, any coils that do not satisfy the minimum tensile strength specification are rejected. This eliminates the tail at the left hand end of the Weibull distribution; the Normal distribution better describes the new wires. The percent reduction from condition 0 to Condition C is calculated for ease in comparing the results. The results of these calculations are given in Table 3.

When no cracks are present, the reduction in cable strength varies from 3 to 8 percent when using the data from these three bridges. The presence of cracks in Stage 4 wires, however, almost triples the strength loss. It is clear that the cracked wires are the primary determinant of strength loss in cables, and that a larger number of Stage 4 samples than samples from other stages is desirable to accurately determine the percent of this stage that are cracked.

5 EFFECT OF VARIATION IN STRESS-STRAIN CURVES ON CALCULATED CABLE STRENGTH

The question arises: what is the effect of the variation in stress-strain curves on the calculated cable strength? It has been known for some time that the Brittle-Wire Model underestimates the cable strength, because wires in this model are assumed to break in the order of the wire strength. It is inherently assumed that a weaker wire breaks at a lower strain, on the average stress-strain curve, rather than the actual ultimate strain of the wire; this is the cause of the underestimation of the cable strength. The development of individual wire stress-strain curves, along with an accurate average stress-strain curve, as part of the investigation of Bridge W enables the determination of the magnitude of the error in the Brittle-Wire Model, as well as using the Limited Ductility Model on the cables of this bridge. This is done by developing a Modified Brittle-Wire Model based on

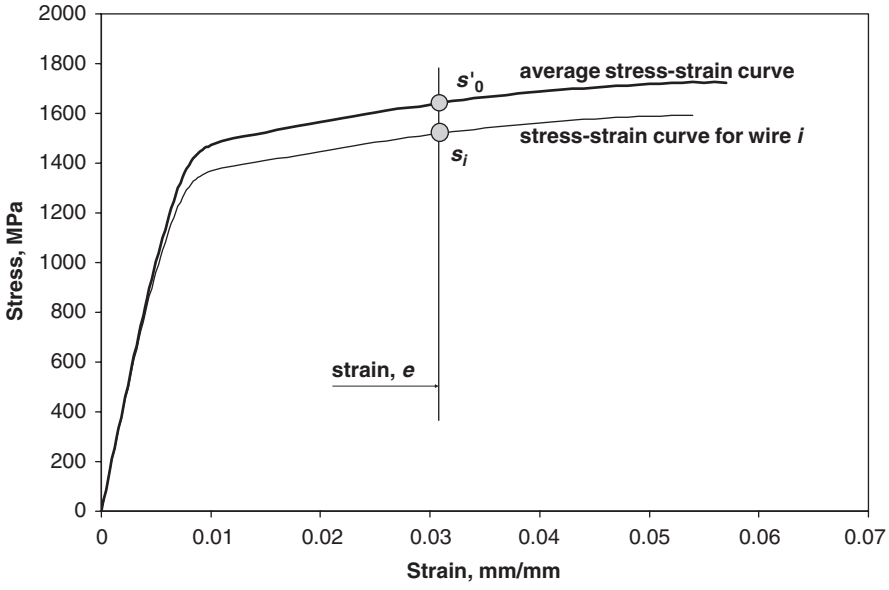

Figure 8. Average stress-strain diagram of *n* wires, and stress-strain diagram of specific wire *i*.

the variation in the individual stress-strain curves. The derivation of this modified model is described in Mayrbaurl et al., "Testing of the Wires from the Forth Road Bridge" (Mayrbaurl, et al., 2006), presented at the IABSE conference in Copenhagen, May 16, 2006. This derivation is repeated here because it may be difficult for interested readers of this paper to obtain a copy of that presentation.

5.1 *Proposed modification to the Brittle-Wire Model*

Figure 8 shows an average stress-strain curve for all *n* tested wires along with a stress-strain curve for a specific wire that was tested, designated by the subscript *i.*

The proposed modification still uses the average stress-strain curve of the tested wires, but in this case the stress used is based on the measured net steel area of each individual sample wire. The wires are assumed to break in the order of their ultimate strain. Assuming that the *n* tested wires are assembled into a cable, and assuming a single stress-strain curve that is the average of all curves determined in testing, the force in one wire (identified by the subscript *i*), at a given strain, *e*, can be expressed as:

$$T_i = s_0' \cdot \alpha_i a_i \tag{1}$$

in which α_i is the ratio between s_i and s'_0, and a_i is the net steel area of wire *i*. The total force in all *n* tested wires will be

$$T_n = \sum_{i=1}^{n} T_i = s_0' \cdot \sum_{i=1}^{n} \alpha_i a_i \tag{2}$$

Denoting the ratio of the area of wire *i* to the average area of the wires (a_w) as β_i, and multiplying and dividing by *n* results in

$$T_n = n a_w \cdot s_0' \cdot \frac{1}{n} \sum_{i=1}^{n} \alpha_i \beta_i = n a_w \cdot s_0' \cdot avg(\alpha_i \beta_i) \tag{3}$$

This derivation depends on α_i being constant over a range of strains between 1.5 and 4.5%, where most cables reach their computed strength. It was found that α_i is constant in this range, varying by less than 0.5%, and the derivation is valid. It was further determined that $avg(\alpha_i\beta_i)$ varies by less than 0.1% over the same range and can also be considered constant, thus Equation 3 can be simplified further by substituting a modified average stress-strain curve that is the average curve multiplied by the average of $\alpha_i\beta_i$. This curve is denoted as the s_0 curve, and

$$T_n = na_w \cdot s_0 \tag{4}$$

The wires that were tested were randomly selected and thus are assumed to be representative of the entire cable. The force in the entire cable at strain e is then:

$$T = Na_w \cdot s_0 \tag{5}$$

and maximizing this force by varying the strain e or stress s_0 results in the cable strength. In this equation, N is the number of unbroken wires in the cable as defined in the Guidelines.

In the Modified Brittle-Wire Model, the stress s_0 is incremented and the cable force calculated at each increment of stress. In order to utilize this model, the tensile test data must be converted to tensile strengths on the s_0 curve corresponding to the failure strains of the sample wires. In the investigation of Bridge W, the method given in the Guidelines to estimate the probable minimum tensile strength in a length equal to one panel (the distance between cable bands) for each sample is used. The tensile strengths of the specimens from one sample are assumed to follow a Normal distribution, and the number of standard deviations that results in a probability of l_o/L, where l_o is the free length of the specimen in the testing machine and L is the length of one panel is calculated. The tensile strength at this number of standard deviations represents the probable weakest specimen in a panel length.

The strain in a sample at this probable minimum tensile strength is obtained from the stress-strain diagram of that sample by interpolation; this is the failure strain, or ultimate strain of that sample. The value of the tensile strength on the s_0 curve for that sample can then be obtained from the s_0 curve, again by interpolation.

The resulting tensile strengths, ts_0, are separated into the appropriate corrosion groups and the mean and standard deviation of each group is computed for use in the Modified Brittle-Wire Model, which from this point is identical to the Brittle-Wire Model except that the modified tensile strengths are used. The mean tensile strengths and ultimate strains are given for Bridge W along with those of the tensile strengths found directly from the data in Table 4.

The equation given in the Guidelines for the cable strength, R_u, can be expressed as

$$R_u = \max\left(T_u(s_0) + T_{cr}(s_0)\right) \tag{6}$$

in which $T_u(s_0)$ is the force in the unbroken wires in the cable at stress s_0,

$$T_u(s_0) = N_{eff} \cdot a_w \cdot \left(s_0 \cdot \left(1 - F_c(s_0)\right)\right) \tag{7}$$

where

N_{eff} = number of unbroken wires in the cable
a_w = average net steel area of the wires
s_0 = stress in unbroken wires in the cable
$F_c(s_0)$ = compound cumulative distribution of the tensile strength, the sum of the individual distributions of tensile strength for each corrosion group

and $T_{cr}(s_0)$ is the redeveloped force in cracked wires that have failed at the stress s_0,

$$T_{cr}(s_0) = N_{cr} \cdot a_w \cdot \left(0.95\mu_{s0,2}\right) \cdot F_5(s_0) \tag{8}$$

Table 4. Means and standard deviations of tensile strengths and ultimate strains of wire groups.

		Tensile strength using avg. wire area		Tensile strength using s_0 curve		Ultimate strain using s_0 curve	
No. of samples	Group	Mean tensile strength (MPa)	Standard deviation (MPa)	Mean tensile strength (MPa)	Standard deviation (MPa)	Mean ultimate strain (mm/mm)	Standard deviation (mm/mm)
17	2	1680.7	40.4	1692.3	19.7	.05751	.00715
17	3	1709.5	43.8	1698.0	7.8	.06102	.00663
34	2 & 3 Combined	1695.1	44.0	1695.1	15.0	Not combined	Not combined
35	4	1660.9	62.4	1670.7	28.1	.04824	.00733
11	5 (cracked)	1508.5	93.8	1497.1	194.3	.02564	.01781

where

N_{cr} = effective number of redeveloped cracked wires in the effective development length, the number of cracked wires that could be redeveloped in the evaluated panel if all cracked wires were broken

$\mu_{s0,2}$ = mean tensile strength of Group 2 wires

$F_5(s_0)$ = Fraction of cracked wires that are broken at stress s_0 = Weibull cumulative distribution of tensile strength of Group 5 wires at stress s_0

For further explanation of these equations, refer to the Guidelines. In the following calculations, it is assumed that there are no broken wires present in the cable. If there were, they would be treated in the same manner as in the Guidelines, substituting s_0 for s and redefining a_w as the average area instead of the nominal area. The equations for the Limited Ductility Model given in the Guidelines are similarly modified, redefining a_w and substituting s_0 for s. The definition of e is unchanged.

5.2 *Calculated cable strengths using the Modified Brittle-Wire Model*

The sample cable described above has been evaluated for all four conditions in Table 2, using the data from Bridge W, by both the Modified Brittle-Wire and the Limited Ductility Models. The resulting cable strengths are shown in Table 5, along with the cable strengths computed by the unmodified Brittle-Wire Model. The cable strengths are also shown graphically in Figure 9, using the fraction of Stage 4 wires present in the cable, which is proportional to the number of cracked wires in the cable, as the abscissa.

The cable strength calculated by the more rigorous Modified-Brittle Wire and Limited Ductility Models is from 2.5 to 4.6% stronger than that calculated by the original Brittle-Wire Model. The cause of this increase is the variation in stress-strain curves for the individual wires. It would be expected that the difference would be smaller if there were less variation in the stress-strain curves. The larger increase for new wires is the result of narrowing distributions for Groups 2 and 3 wires (i.e., smaller standard deviations) when compared to those developed using the nominal wire area. The cable strengths become closer together as the fraction of Stage 4 wires increases because the means and standard deviations of cracked wires are nearly the same in both cases. The Limited Ductility Model results in strengths from 0.3 to 0.9% smaller than does the Modified Brittle-Wire Model. This is caused by applying the Weibull distribution to two different aspects of the stress-strain relationship.

5.3 *An alternate method to derive the data for the Modified Brittle-Wire Model*

Is a 2.5% increase in the estimated cable strength worth the expense of such an elaborate investigation and analysis? In most cases, it is not. A simpler technique, however, is possible, based on

Table 5. Cable strengths calculated by the Brittle-Wire, Modified Brittle-Wire and Limited Ductility Models.

	Cable strength (kN)			Change from Original Brittle-Wire Model (%)	
Condition	Original Brittle-Wire Model	Modified Brittle-Wire Model	Limited Ductility Model	Modified Brittle-Wire Model	Limited Ductility Model
New	294,000	308,000	305,000	4.6	3.7
A	290,000	303,000	301,000	4.3	3.6
B	279,000	287,000	286,000	3.1	2.7
C	270,000	277,000	276,000	2.5	2.3

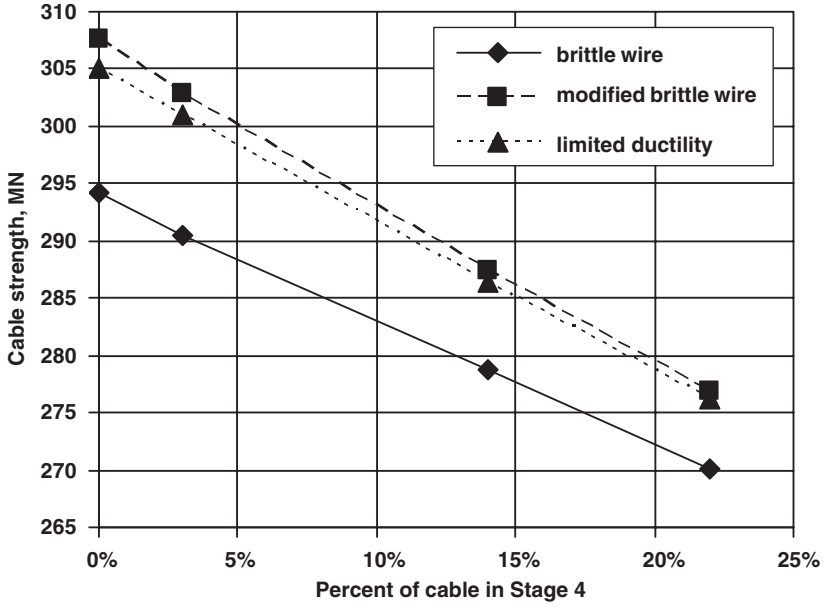

Figure 9. Comparison of Brittle-Wire, Modified Brittle Wire and Limited Ductility Models, using Data from Bridge W.

the similarity of stress-strain curves of bridge wires. The Bear Mountain average stress-strain curve can be made to fit the data measured on Bridge X, and it has been shown that it can be made to fit the data from Bridge W to within 0.5% by simply multiplying the entire curve by a constant. The individual stress-strain curves found for Bridge W also fit the average curve to less than 0.5% in the range of 1.5 to 4.5% strain, the portion of the curve within which the cable strength lies. This is sufficient to improve the accuracy of the cable strength calculation by using the Modified Brittle-Wire Model.

Thus, it would be sufficient to measure not more than eight stress-strain curves, producing an average and comparing it to one of the existing average stress strain curves as a check that the wires of the investigated bridge follows a similar curve. If so, all that is needed is to determine the stress at a strain of 2 or 2.5% for each specimen and to determine the ratio between this characteristic value and the average stress at the same strain, for use in adjusting the stress values to the average curve. The average curve is simply determined by adjusting one of the existing average curves by a constant. The s_0 curve is then determined from this average curve by multiplying it by

the term $avg(\alpha_i\beta_i)$ determined at the strain used for the characteristic value. The minimum tensile strength of each sample, ts_i, is adjusted to the s_0 curve using the equation

$$ts_{0i} = ts_i \cdot avg(\alpha_i \beta_i)/\alpha_i \tag{9}$$

and the analysis is completed in the usual manner. The ultimate strain for each sample can be found also if the Limited Ductility Model is to be used, by interpolation from the s_0 curve. Only the stress at a pre-selected value of strain need be measured, greatly simplifying the testing program.

The data for Bridge W has been used to test this simpler method, using the stress at 2.5% strain for the characteristic values of the individual stress-strain diagrams. The average curve that was found for the Bridge W wires was used in this test, as the average stress-strain curve for the Bear Mountain Bridge does not extend to a high enough strain to apply it to the Bridge W wires. The cable strengths calculated by both the Modified Brittle-Wire Model and with the Limited Ductility Model with the resulting data are generally smaller than those given in Table 5 by 0.2%; the largest difference is 0.4% in the case of the strength of a new cable estimated by the Limited Ductility Model. These differences are well within the accuracy of the two models, and the technique can be applied with confidence.

6 CONCLUSION

- The results of tensile tests on all specimens taken from a single sample wire should be kept separate from those taken from other sample wires; they should not be reported in a single table that includes all specimens.
- The stress-strain curves for bridge wires are similar to within 0.5%, even for bridges built 40 years apart.
- The stress-strain curves for the wires within an individual cable can vary by as much as 12 to 13% because of variations in carbon content, which may be outside the specified limits.
- Carbon content tests should be made on about 15 samples in a cable investigation.
- The full length stress-strain curve for several samples wires should be developed to verify their similarity to those found in other investigations.
- At least one specimen from each sample wire should have its characteristic stress value determined at a strain of 2.5%.
- An average stress-strain curve can be developed for the cables by averaging the characteristic stresses for the individual samples found at a single value of strain and adjusting an already known average curve to fit that average characteristic stress. The strain recommended for this technique is 2.5%.
- The Modified Brittle-Wire Model produces more accurate cable strengths than does the more simplified Brittle-Wire Model given in the Guidelines.
- The Limited Ductility Model gives nearly the same results as does the Modified Brittle-Wire Model, and either can be used to evaluate the cable strength.

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4 Cable studies and planning

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Chapter 11

Planning and engineering for the future: Capacity increase and cable replacement at the Bronx-Whitestone Bridge

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ABSTRACT: Long-term planning to meet future traffic demands on the Bronx-Whitestone Bridge has many significant challenges. Several traffic studies of the BWB, the Throgs Neck Bridge and the region's highway network have been performed which indicate that the traffic demand will continue to increase in the region, particularly at the BWB. Based on these predictions, several planning studies investigating the feasibility of providing for the future traffic demands have been performed. Two main concepts were investigated for increasing traffic capacity; the addition of a second level and the widening of the bridge. Both concepts require cable replacement to accommodate the heavier loads. Both concepts have myriad associated technical challenges as well as even more significant planning issues such as access, environmental impacts, maintenance of traffic, and flexibility for accommodating future traffic demand.

1 BACKGROUND

The Bronx-Whitestone Bridge, a 1150 meter long suspension bridge with viaduct approaches, was originally constructed in 1939 to carry four lanes of vehicular traffic, as well as sidewalks for pedestrians, between the boroughs of Queens and the Bronx across the East River. The bridge has a suspended main span of 700 meters with two 225 meter side spans. The suspension system consists of two 552 mm diameter main cables, 22.6 meters apart, with 51 mm diameter suspender ropes in pairs spaced every 12 meters. The cables are anchored by means of large concrete anchorages on both the Bronx and Queens sides. The anchorage on the Bronx side is on park land and is set back from the water's edge. The anchorage on the Queens side is also on park land, but is also close to private property and is right at the water's edge. The original deck on the suspended spans consisted of concrete filled steel grid supported on subfloorbeams between stringers, which in turn were carried by floorbeams that are connected to 3.4 m deep stiffening girders every 6 m. The original bridge experienced excessive wind induced vibrations after its completion in 1939.

Over the years, several modifications have been made to the structure. After the collapse of the Tacoma Narrows Bridge in 1940, a bridge with similar structural characteristics as the BWB, modifications were made to mitigate wind induced vibrations. First, cable stay ropes were added to increase stiffness, followed by the addition of a heavy stiffening truss in 1946 which was added on top of the main girders on each side of the bridge to add stiffness to the suspended span and improve the aerodynamic performance of the bridge. In addition to adding the stiffening truss, the sidewalks were

removed and a third travel lane in each direction was added to the bridge creating the current configuration of six travel lanes that are only an average of 3.2 m wide with no shoulders. However, even with these changes, the bridge continued to experience excessive twisting during high winds. Therefore, a tuned mass damper was added to at the center of the main span in 1985 to counter and dampen the symmetrical torsion of the bridge during high winds. All of these modifications to the bridge have significantly increased the dead load on the main cables.

In 1995, it was determined that the eyebars that anchor the main cables to the concrete mass of the anchorages were deteriorated at the concrete interface. A re-anchoring system was designed and implemented in 1997 to transfer a portion of the load from the eyebars using a large transfer girder system to transfer the loads to anchor rods anchored deep into the anchorage concrete. Also, as is typical with a suspension bridge of this age, the main cables have experienced normal deterioration along the cable and within the anchorages. A cable inspection was performed in 1998 to ascertain the condition of the main cables and make any necessary recommendations for future actions. During this investigation it was recommended that some of the dead load that was added over the years be removed.

Two recent construction contracts have contributed to a significant reduction of dead load on the cable, as well as improvements to the bridge's aerodynamic performance. The heavy stiffening trusses were removed and a lightweight triangular wedge shaped wind fairing system was installed to provide aerodynamic stability to the suspended spans. The removal of the stiffening trusses also restored the bridge back to its original sleek appearance envisioned by O.H. Ammann, its original designer (Rastorfer 2000).

In addition, the 65 year old concrete grid deck system had reached the limit of its useful life and needed to be replaced. The concrete deck is being replaced with an orthotropic deck system that is significantly lighter. As an added benefit, the orthotropic deck coupled with the lowering of the lateral system, will stiffen the bridge further. This new deck system with the lowering of the laterals forms a rigid torque tube with the stiffening girders that further improves the aerodynamic stability of the bridge. Upon completion of the deck replacement, the new integrated structural system will significantly reduce strain on the main cables as well as improve the aerodynamic stability of the suspended spans.

The approach viaduct structures are almost 70 years old and have experienced deterioration due to age as well as today's heavier truck loads. In addition, the approach structures do not meet current highway standards for lane or shoulder width. The replacement of the approach structures is currently under design and scheduled to begin construction in 2008. The new approaches will be constructed to modern standards resulting in wider roadways on the reconstructed approach spans.

2 TRAFFIC STUDIES

The Bronx-Whitestone Bridge (BWB) is one of three bridges crossing the Long Island Sound or the East River connecting the boroughs of Queens and the Bronx. The Throgs Neck Bridge (TNB) is situated just to the east of the BWB, and is the last connection, between Long Island and the Bronx, Westchester and Connecticut. The Triborough Bridge (TB) is situated a greater distance away and to the west of the BWB, and also connects to the borough of Manhattan as well as the Bronx. All three bridges are interconnected with a series of highways in both Queens and the Bronx.

On the Bronx side, the highway system connecting the BWB, the TB, and the TNB are redundant. Users can access either the BWB or TNB through a number of interchanges between several highways. There is much less interconnectivity between the BWB and the TNB on the Queens side than on the Bronx side. This is due to the overall layout of the highway network in Queens. The highways on the Queens side serve somewhat different areas and destinations, and are less redundant than the bridge/highway system in the Bronx. In the Bronx, the user can choose between the BWB and the TNB, and to a lesser extent the TB, at various interchanges that are close to the bridges, and likewise can choose the various highways in the Bronx from any of the three bridges. In Queens, the choice of which bridge (TNB or BWB) to use is often made somewhere in the highway network far from the bridges.

A complex traffic study was performed in 2004 to model the interaction between the BWB, the TB, and the TNB and the surrounding highway network. The traffic study was performed using the Best Practice Model (BPM) to model the highway traffic in the region and specifically at the Bronx Whitestone Bridge and Throgs Neck Bridge. The BMP model was developed by the New York Metropolitan Transportation Council (NYMTC) to meet federal requirements for long-range transportation planning, including air quality conformity and regional and subregional area and corridor analyses. It is a travel demand model that is used to identify future transportation needs. The BPM highway network was locally refined in the vicinity of the BWB and TNB to better represent base year traffic conditions (2002), and then used to forecast travel demand with and without the various improvements to the BWB and the TNB. Modeling potential capacity increases on the BWB and the TNB provided information on future travel demand on the bridges and also information on forecasted traffic redistribution among the Bronx Whitestone, Throgs Neck and Triborough Bridges. The BPM was used to model the following:

- Base year (2002) traffic at the Bronx Whitestone, Throgs Neck and Triborough Bridges
- Forecast (2025) travel demand for baseline conditions
- Forecast (2025) travel demand for the various capacity improvement alternatives

The traffic study modeled traffic interaction between the BWB and the TNB using the BPM to determine whether the BWB and the TNB could be made to work together as tandem facilities; whether traffic could be moved more efficiently between the Bronx and Queens, and whether the TNB in general could serve as an alternate route for the predicted future increased traffic on the BWB as an alternate to increasing capacity on the BWB. Alternatives for additional lanes on the BWB were modeled to determine what would be needed to meet future traffic demand. As an alternative, adding the missing connection ramps from the TNB to the Cross Island Parkway, to improve interconnectivity between the TNB and the BWB on the Queens side, as well as providing an additional reversible lane and movable barrier on the TNB was modeled to see if these proposed improvements to the TNB would draw traffic away from the BWB. The TNB was also studied to see if it could serve as an effective detour route during the reconstruction of the bridge approaches. The results of the study were as follows:

- Traffic volumes on both the BWB and the TNB would continue to increase during the 6 to 10 AM and 4 to 8 PM peak periods for the 2025 study year.
- A fourth lane in each direction is needed at the BWB and would be fully utilized in 2025 based on the forecasted demand from the local highway network. Providing more than 1 additional lane in each direction on the BWB would exceed the traffic capacity of the local highway network.
- An additional reversible lane with movable barrier is needed at the TNB to provide the capacity for the projected traffic increase at the TNB. The additional lane at the TNB would also pick up displaced BWB traffic during periods of congestion.
- Providing the missing ramps from the TNB to the Cross Island Parkway (CIP) improves interaction between the BWB and TNB by only a limited amount, and is not cost effective.

The recommended actions were to provide additional capacity at the BWB in the form of a fourth lane in each direction, as well as to provide an additional reversible lane at the TNB with movable barrier.

3 PLANNING FOR THE FUTURE

Based upon the results of the traffic study, there are several goals for the BWB. First and foremost, the structure must be maintained in a state of good repair. A secondary goal is to study possibilities for increasing the traffic capacity of the suspended span, given the current constraints due to the configuration of the bridge towers, anchorages, cables and deck. Besides completely replacing the bridge or adding a twin bridge, neither of which are being considered, it was determined that there are two realistic possibilities for increasing the traffic capacity of the BWB:

- Double Decking – Add a second level to the bridge
- Widening the bridge – Two alternatives

- Modification of existing towers and Cantilevered Roadways
- New Towers

The approach structures are also narrow and have reached the end of their useful life and require replacement. Since the replacement of the approach structures is imminent and currently under design, it was necessary to investigate the feasibility of providing an approach structure capable of meeting the existing narrow roadways of the suspended spans, yet also capable of accommodating future capacity improvements on both the suspended spans and on the approaches, in either a widened or double decked form.

After a thorough investigation of the available options, the approach structure replacement will be designed to accommodate either of the future traffic capacity improvement options on the suspended spans (i.e. the addition of a second level or widening). Since the approaches are being widened to meet current standards when they are replaced, accommodating widening of the suspended spans can be accomplished with a minimum of re-construction of the approach structures. However, significant new construction and some reconstruction of the approaches would be necessary to accommodate double decking on the suspended spans.

The Queens Interchange of the Whitestone Expressway and the Cross Island Parkway, which feeds the Queens Approach to the BWB, was also investigated to determine whether the current interchange can provide access to either a second level on the bridge or a widened bridge. It was determined that the interchange does not need significant modification if the bridge is widened. However, if a second level is added to the bridge, the interchange must be significantly re-configured to provide access to both levels of a two level BWB without operational restrictions.

In order to investigate feasible options for increasing the capacity of the suspended span on the BWB, two separate studies have been performed. The first study investigated the feasibility of adding a second level to the bridge to provide four lanes with shoulders in each direction. This study looked at the need to either augment or replace the main cable and suspender ropes to carry the heavier loads associated with the addition of a second level, the need for tower reinforcement, modifications to the anchorages, and the ability to maintain traffic during construction of the new cable and the addition of a second level. The second study investigated the feasibility of widening the bridge to provide four standard width lanes with shoulders in each direction. This study looked at several alternatives and the associated needs for a new cable, modifications to the towers and anchorages, and maintenance of traffic during construction. Both options (double decking or widening) for adding capacity to the suspension bridge are feasible, however there are significant differences in the impacts to traffic and surrounding property, as well as the cost and the amount of work needed on roadways beyond the limits of the suspension bridge.

3.1 *Double decking*

Adding a second level to the bridge requires significant reinforcing of the towers, replacement of the cable, enlargement of the anchorages, and all construction will be over existing traffic. In addition, to access a two level bridge, a second level will be constructed on the approach structures (once again over traffic), the toll plaza will need significant reconfiguration, and the Queens Interchange will need to be completely reconfigured. Environmental impacts in the East River will be minimal, since the towers are only being reinforced, however, the enlargement of the anchorages may have impacts on the East River on the Queens side. There are also significant environmental issues associated with adding a second level to the approach structures and reconfiguring the Queens Interchange.

3.2 *Widening*

3.2.1 *Modification of existing towers and cantilevered roadways*

One alternative is to replace the existing cables with larger cables capable of carrying the widened structure, reinforce the towers, and add additional separated roadways cantilevered outside the

existing tower legs. This alternative requires significant reinforcing of the towers, and enlargement of the anchorages, similar to the double deck approach. This alternative minimizes environmental impacts in the East River from the tower work, however, there would still be impacts to the East River from the enlargement of the Queens Anchorage. In addition, the approach roadways would need some modification to tie into the separated roadways. The main disadvantages to this widening alternative are potentially large deflections from heavy trucks on the cantilevered outer roadways, and the more difficult operation of the bridge due to the separated roadways.

3.2.2 *New towers*

The other alternative for widening the bridge includes the construction of new towers outside of and over the existing towers, with a new pair of cables to support a contiguous roadway with four lanes and shoulders in each direction between the new tower legs. The roadway deck would be widened, and the existing towers removed. The anchorages would be enlarged to a greater extent than under the double deck or widening alternative based on re-using the towers. There will be significant impact to the East River from the construction of the new towers, as well as impacts to the East River from the enlargement of the Queens Anchorage. However, this alternative minimizes the need for modifications to the approach structures. This alternative has several advantages over the double decking and other widening alternative; the old steel towers would be replaced with new towers, roadways will be contiguous, the bridge will have better aerodynamic performance, and would have a better appearance. Under either widening option, the majority of the construction will have little to no significant impact on the traveling public, the toll plaza would not need to be modified, and the reconfiguration of the Queens Interchange is not necessary.

Using the results from these studies, it is possible to chart a future course in the effort to meet the growing demands of regional traffic.

4 DOUBLE DECKING ALTERNATIVES

Several options were evaluated for replacing the main cables to allow a second deck to be added to the bridge. Two of the main options investigated were:

- Installing new cables either above or outboard of the existing cables, and replacing the existing cables, resulting in a totally new, four cable system to support both the existing structure and the additional deck.
- Install one larger diameter cable either above or outboard the existing cables and removing the existing cables, resulting in one set of new main cables capable of supporting both the existing structure and the additional level.

The feasibility study concluded that the addition of cables directly above the existing cable would not be as cost effective as the addition of cables outboard of the existing cables due to the close proximity of the cable work to the roadway below, the need to re-construct the top of the towers to carry the load, the difficulty in tying into the anchorage with the new, higher cable, and the difficulty inherent in placing the new suspender ropes around the existing cable, Figure 1, removing the existing cable from between the ropes, and building the new cable (in the case of the four-cable system) between the suspender ropes of the higher cable below.

4.1 *Four cable system*

4.1.1 *Superstructure*

The new additional cables are located outboard of the existing cable and stiffening girders. New suspender ropes are attached to the replacement cable at alternate panel points (i.e. between existing suspenders). The existing superstructure will require retrofitting of the stiffening girders for the new suspender connections. This will consist of extending the existing floorbeams to provide an attachment point for the new suspenders, removing the old suspender rope anchorages and repairing the existing steel in its vicinity.

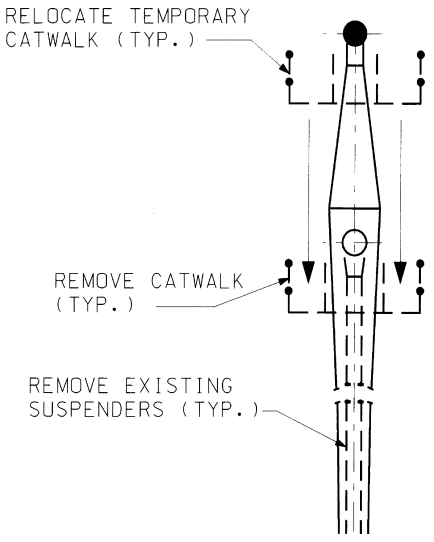
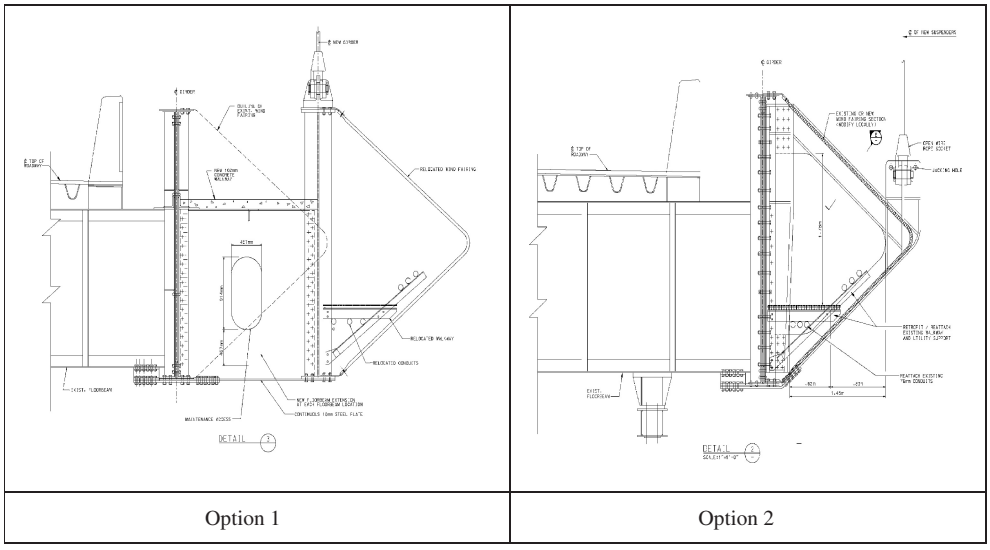


Figure 1. New cable placed above existing cable.



Option 1	Option 2

Figure 2. Floorbeam extension options.

Two options for the floorbeam extension were evaluated: Option 1 consisting of a traditional extension with a new longitudinal stiffening girder, and Option 2 consisting of a V-frame extension, Figure 2. Option 1 requires that the wind fairing be removed, the floorbeam extended and a new longitudinal girder installed, and the wind fairing re-located to the new girder. However, Option 1 also allows the second level supports to be placed on the existing safety walk location and allows room for a new safety walkway location between the second level supports and the new girder. Option 2 was explored to allow the wind fairing to remain in place, only being removed and retrofitted at every other panel point where the new suspender rope would be connected. This

would eliminate the need for a full floorbeam extension with additional girder, however, would limit the available space for the second level supports.

4.1.2 *Main cables*

The new additional main cables would be located 2.1 m outboard of the existing cables, attached to the floorbeam extensions to support the superstructure, and then the existing cables would be replaced in kind. The geometry of the new outboard cables places them on a profile essentially identical to that of the existing cables. The size of the new cables for the 4-cable system was assumed to be the same as the existing cable, which is 552 mm diameter, consisting of 37 strands of 268 wires each.

4.1.3 *Suspender ropes*

The new suspenders were designed to improve on the existing configuration by utilizing two single-part strands pinned to the cable bands below the main cables. Suspenders are located at every other panel point on each cable, even panel points for one set of cables and odd panel points for the other set of cables. The suspender strands would conform to the requirements of ASTM A586 with Class A galvanizing. For floorbeam extension Option 1, the wire strand would be 57 mm nominal diameter, with a minimum catalog breaking strength of 340 metric tons. For floorbeam extension Option 2, the wire strand would be 54 mm nominal diameter, with a minimum catalog breaking strength of 305 metric tons. The strands shall be socketed with spelter or resin cones. The socket would be designed for 110% of the strand ultimate strength. For design, the strands the minimum factor of safety is 3.5 for Allowable Stress Design (ASD) loads based on ultimate catalog breaking strength values.

4.1.4 *Anchorage modifications*

Under this alternative, the new cables would be positioned 2.1 m outboard of the existing cables. The modified anchorage is designed to maximize the use of the existing anchorage mass in resisting the new cable pull. The design concept was developed assuming the new cables will be formed by air spinning (AS). If the replacement cables will consist of preformed parallel wire strands (PPWS), the cables will have more strands and therefore the cable splay details will require more space. The existing cables will be replaced in kind and will be anchored into the original anchorage chamber. Prior to the installation of new cables in the same location as the original cables, the existing anchorages need to be modified.

Modifying the anchorage to accommodate a four-cable system will require the following:

- A new concrete anchor block will be added to the side and back of the existing anchorage for each of the new outboard cables in which new enclosures would be constructed for the new outboard cables and saddles, Figure 3. Each new anchor block would be partially on the back of and extending about 12.8 m beyond the existing anchorages. The anchorage block would be extended 2.1 m at each side.
- Shear friction reinforcement between new and existing concrete makes the new concrete integral with the existing anchorage. The shear friction reinforcement bars are drilled and grouted into the existing concrete.
- At the Queens Anchorage each new anchor block would be supported on drilled shafts.
- At the Bronx Anchorage the anchor blocks would be supported on spread footings similar to the existing foundations.
- The existing splay chamber enclosures will be retrofitted and re-used for the new cables installed in the same position as the original cables. Modifications in the existing anchorages include refurbishing the existing splay saddles, refurbishing the existing eyebars, the installation of additional concrete at the rear of the anchorage block and the installation of additional concrete at the toe of the anchorage that incorporates a shear key for additional seismic retrofit. The existing eyebars would be refurbished prior to the installation of the new cable. The refurbishment would consist of removal of concrete to expose sound steel on the embedded portion of the eyebars. Any deteriorated portion of the eyebars would be cut off and new steel would be spliced to the existing sound steel.

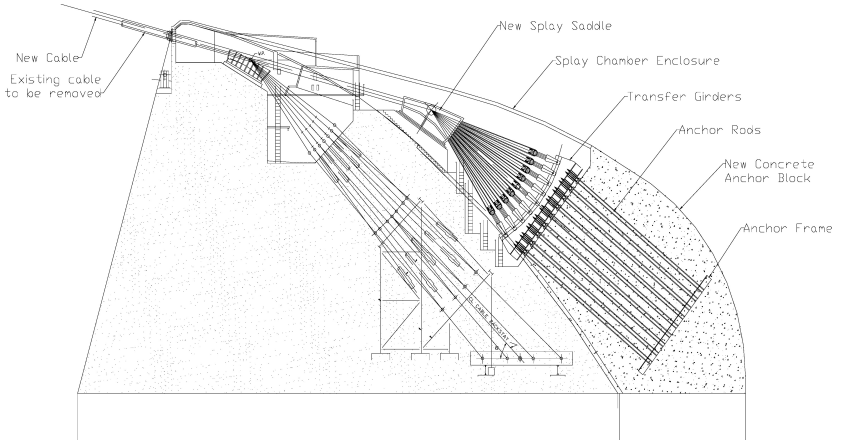


Figure 3. Anchorage extension – new outboard cable.

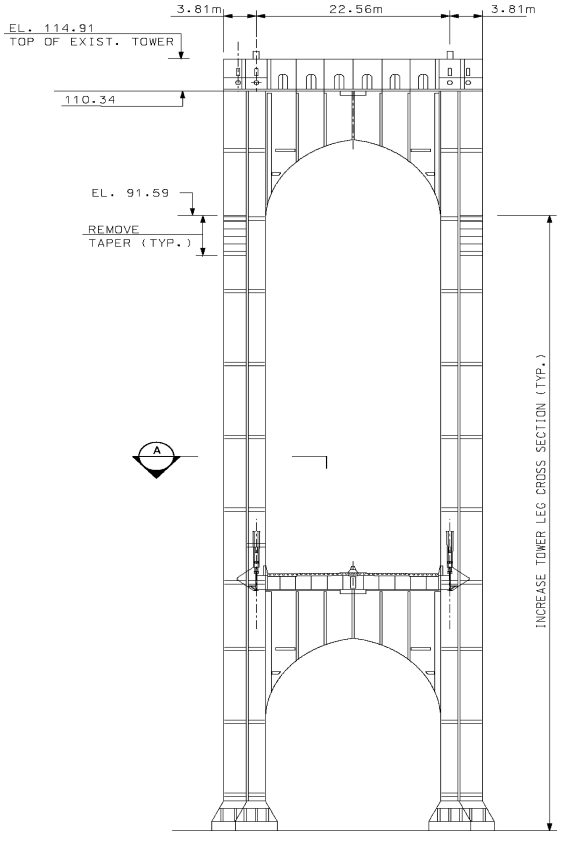


Figure 4. Tower modification for four cable system.

4.1.5 *Tower modifications*

The new saddles will be located 2.1 m outboard of the existing saddles. In order to support these new cable saddles and carry the additional load of a second deck, the overall tower leg cross section must be increased (full height), and localized strengthening of the tower is required in several

areas, Figure 4. The existing saddles must also be refurbished once the original cable is removed, and prior to the installation of a new cable in place of the original.

Preliminary analysis indicates that existing tower leg cross section is inadequate for carrying the anticipated loads of double decked superstructure. It is therefore deemed necessary to increase the tower leg cross section for the full height of the towers. The preliminary tower analysis also indicated that at several discrete locations, the existing tower steel would experience minor overstressing due to the loads imposed by the addition of a second level. The overstressing is limited to the existing tower leg steel located near the top and bottom struts. These areas would be strengthened with additional steel plates.

The existing tower foundations appear to be adequate to support the additional load of a second roadway level. The allowable bearing capacity of the caissons founded on bedrock is 391,000 kg/m^2. The maximum bearing pressures from the original design were 230,000 kg/m^2. The bearing pressure resulting from a new second level is in the range of 318,000 kg/m^2 to 342,000 kg/m^2.

4.1.6 *Load transfer*

An incremental jacking procedure would be used to transfer the load from the existing main cables to the new outboard cables. Following load transfer, the existing main cables would be removed strand by strand. The new outboard cables would remain in a configuration that is practically identical to that of the existing cables.

The pre-socketed replacement cable suspenders would be manufactured to specified lengths as determined through analysis of the stretch induced into the suspenders under full dead load. To erect and load a replacement cable suspender, the suspenders are first connected to the cable bands and free hung from the replacement cable. Unloaded the suspender strand ends will be free hanging at some distance above the top of the stiffening girder. The two sockets are then attached to a jacking beam, which in turn is connected to an upper jacking bracket and jacks; the bracket is connected via tension bars to the floorbeam extensions.

Following a prescribed sequence, groups of suspenders are gradually jacked into position. In between the intermediate jacking stages, the suspenders are temporarily connected to the superstructure using threaded extensions and couplers. After the erection of replacement rope suspenders, the old suspenders and the existing main cable will be removed. Both the erection of the new suspenders and removal of the old suspenders will follow a well-designed sequence of operations so as not to overstress any bridge member.

4.2 *Addition of second deck*

4.2.1 *Suggested construction sequence*

The addition of the second deck would increase the capacity of the bridge from six (6) substandard traffic lanes to eight (8) standard width lanes and shoulders; the future configuration would be 2 lanes each direction on each of the two decks.

The challenge to implementing this option lays primarily in the necessity of working overhead while maintaining the traffic on the bridge with a minimum of disruption. The suggested method relies first and foremost on staging most of the work on the upper deck of the approach spans being expanded as part of the same capacity improvement, combined with the use of a pair of gantry cranes traveling on the top chord of a new truss designed to carry the second deck, Figure 5.

Before commencing the work on the new superstructure it would be necessary to install temporary catwalks and the new inner cables. Once the new longitudinal truss is installed for support of the upper level framing, the new suspenders would be connected to the truss using temporary extensions, and a partial load transfer performed to adjust the truss geometry. Final load transfer would be performed incrementally after the second level roadway framing and new orthotropic deck are installed. As previously discussed, the load transfer operation is designed to minimize stresses in the superstructure members. The final result is that the outer and inner cables equally share the loads of the double-decked superstructure.

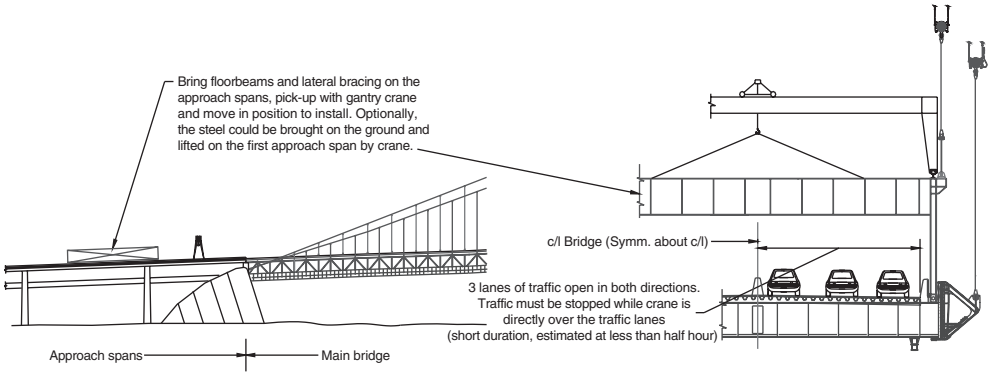


Figure 5. Construction sequence for erecting second roadway level.

Clearly, there would be some impact on the existing traffic, however, in this suggested method it is limited to a lane closure. Other than this lane closure, the rest of the work would be performed from above, by advancing the work while shielding the traffic by means of the new upper deck. It would be necessary to stop the traffic for brief periods of time when the gantry crane is traveling beyond the extents of the new upper deck. These interruptions are estimated to be of short duration. It is anticipated that most of this work would be performed during off-peak periods.

4.3 *Single larger cable system*

Under this alternative, 30″ diameter cables would be constructed 2.1 m outboard of the existing cables, the load is transferred to the new cables via floorbeam extensions, and the existing cables are removed. The 762 mm diameter cables would be capable of supporting both the existing structures and an additional second level. The impacts to the superstructure, towers and anchorages are very similar in nature to the impacts of the four-cable system discussed above. This option abandons the existing saddles, benefits only partially from the existing anchorages and results in a more severe visual impact on the anchorages when compared to the four-cable system.

5 WIDENING ALTERNATIVES

The suspension bridge towers as they are currently configured do not allow for installation of new cables far enough outboard of the existing cables to accommodate widening of the bridge. The towers are a direct obstacle to widening the roadway and they need to remain in service to support the existing roadway during any construction alternative. For these reasons, the alternatives studied for widening are all based on what can be done with the towers. All other aspects such as widening at the anchorages or widening of the roadway deck are accommodated in similar fashions. Four alternatives were studied, two alternatives involving reuse of the existing towers which requires the addition of separated outer roadways, and two alternatives involving replacement of the towers with new towers, which have wider spaced legs allowing for a continuous width roadway without the need for separation. The following are the four alternatives evaluated:

- Modification of Existing Towers
  - Cantilever brackets – Add large cantilever brackets to the existing tower tops to support a second outer pair of cables that would provide support to a new roadway added outside of the tower legs.
  - Cantilevered Roadways – Replace the existing cable with a larger cable to support a new roadway added outside the existing tower legs through cantilevered floorbeams.

- New Towers
  - New Tower on new Foundation – Provide a new tower on new foundations constructed outside of and over the existing tower, with enough room between the new tower legs for a full width roadway.
  - New Tower on Existing Foundation – Provide a new tower, supported off the existing tower foundations, constructed outside of and over the existing tower, with enough room between the tower legs for a full width roadway.

5.1 *Modification of existing towers and new cantilevered roadways*

Widening the bridge by using the existing towers requires new travel lanes to be constructed on the outside of the tower legs. There are two options for this; (1) the four cable option, with a pair of new cables supported by brackets at the top of the towers which will partially support the two outside lanes, and two new cables placed 2.4 m outboard of and replacing the existing cables partially supporting the outside lanes and fully supporting the inside lanes; and (2) the cantilevered roadway option which includes constructing one set of larger main cables 2.4 m outboard of and replacing the existing cables, that are capable of supporting both the existing roadway and the new cantilevered outer lanes. The cantilevered roadway option, Figure 6, is discussed herein because it is deemed to have greater feasibility due to lower cost, better constructability and smaller anchorages than the four cable option.

5.1.1 *Tower modification and reinforcement*

At the roadway level, the existing towers need to be modified and strengthened to carry new lanes and associated loads. Additional brackets attached to each side of the towers below the deck level as extensions of the lower strut would support a compression link for the new stiffening girders. Steel plates spliced through the existing rivet holes into the exterior surfaces of the tower legs and struts would also be used to increase the strength of the existing tower legs to carry the additional loads.

At the top of the tower, the new main cables and their saddles would be installed on the outboard side of and adjacent to the existing cables and saddles, similar to the double deck alternative. Local reinforcement of the towers will be required to support and distribute loads from the new cables. Constructability issues include construction of reinforcement in confined areas while maintaining stability of surrounding material, and access to deliver materials into tower cells. The construction sequence would be based on maintaining the stability of the towers during retrofitting activities around the saddle. A seismic evaluation performed in 1996 determined that the horizontal concrete strut connecting the pedestals between the tower shafts is vulnerable in both shear and bending. During the modification of the towers this vulnerability can be addressed by either retrofitting this strut or replacing it entirely.

5.1.2 *Anchorage expansion*

A widened bridge with main cables spaced further apart than the existing cables will require larger and wider anchorages, similar to the double deck alternative. Since the new cables will be placed only 2.4 m outside of the existing cables, and to minimize property impacts, the new anchorage will be placed on top of, and behind, the existing cable anchorage, causing bending about the vertical axis of the anchorage. The new bent saddle concrete base will be integrated into the existing concrete mass block using dowel rods and shear friction to engage the existing anchorage structure, countering the bending moment effects.

The new anchorage mass block would be designed to maximize the space between cable strands for maintenance. The anchorage would be widened by 2.7 m on each side, and be extended by approximately 15.5 m on the backsides past the existing anchorage, Figure 7.

On the front of the Bronx Anchorage, a shear key, Figure 7, would be provided for additional capacity against sliding due to either the pull of the cable or during a seismic event. The new foundation for the expanded Queens Anchorage would be composed of drilled shafts which would

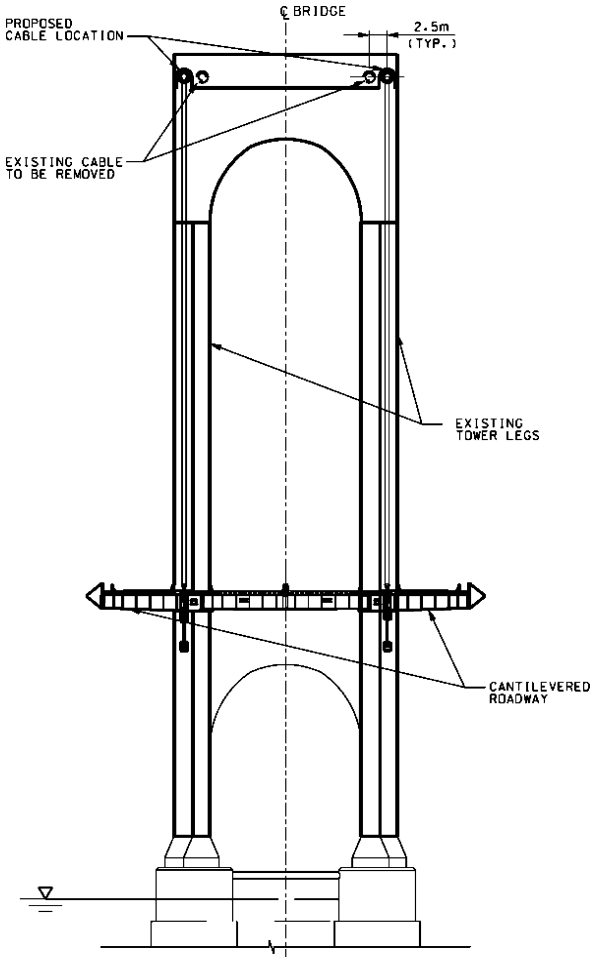


Figure 6. Cantilevered roadway option.

share the weight of the expanded anchorage in conjunction with the existing caissons and resist and the moment derived from the total cable pulling force.

5.1.3 *Suspension system*

The new cables would require significantly more carrying capacity than the existing cables, since they would be carrying both the existing roadway and the new outer roadways. The new saddles would be placed at the same elevation as the existing saddles and the sag of the new main cables would mirror the existing sag, which is limited by the height of the existing towers. The connection detail for the suspender ropes at a new stiffening girder would need to allow for temporary jacking devices to transfer the loads from the existing cables to the new cables, and distribute load to the new cable, similar to the double deck alternative.

There are two ways of erecting cables: Aerial Spinning (AS) and Prefabricated Parallel Wire Strands (PPWS). One of the advantages of the AS method is that single spools of wires can be handled more easily than PPWS, which needs large drums. Modern aerial spinning devices, which use the constant tension method, will provide superior parallelism of wires without crossing, which is necessary for good cable compaction (few internal voids) and permits more direct anchorage to the anchoring bars. One of the disadvantages of the AS method is that the internal voids must be

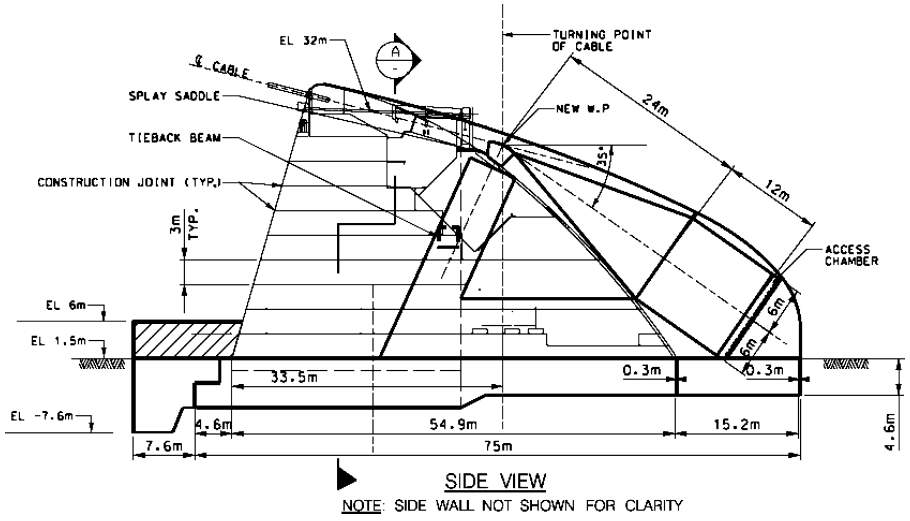


Figure 7. Bronx anchorage modification – cantilevered roadway option.

compacted out, which is not easy. This leads to a longer erection time for the AS method when compared to the PPWS method. Standard PPWS strands are hexagonal and are much easier to compact, leaving few internal voids.

Considering both aerial spinning and prefabricated parallel wire strand (PPWS) as cable construction options, the following are potential designs for the new cables:

- Spun Cable – Thirty seven strands each with 331 wires, for an approximate cable diameter of 660 mm.
- PPWS Cable – Sixty one strands each with 217 wires, for an approximate cable diameter of 686 mm.

The 61 strands may be compacted slightly farther apart in the vertical plane than the 74 half strands of the 37 spun strands. However, the horizontal space between strands in the anchorages will be equally accessible for both strand types.

New suspender ropes would be approximately 83 mm in diameter. The connection detail for the suspender ropes at the new stiffening girder or truss would need to allow for temporary jacking devices or similar means in order to be able to transfer the loads from the existing cables to the new cables.

5.1.4 *Floorbeam and deck extensions*

A widened deck will require extensions to the existing floorbeams that need to have continuity by pass-through of the existing stiffening girders, Figure 8. Floorbeam extensions can be connected at the top of the floorbeams by cutting holes through the existing stiffening girder and connecting the new floorbeam flanges directly to the existing floorbeam flange angles, and at the bottom of the floorbeams by connecting a plate across the lateral brackets to transfer dead load. Since the orthotropic deck acts compositely with the floorbeam, effectively increasing the floorbeam capacity, it should not be necessary to reinforce the existing floorbeam. The cantilevered floorbeam extensions must be designed to be strong enough to support any applied loads and be stiff enough to satisfy deflection limitations.

The new stiffening girders will have to support the existing superstructure and the new superstructure. These girders will effectively span 12 m between the new suspenders, and will require a section both stiffer and stronger than the existing stiffening girder.

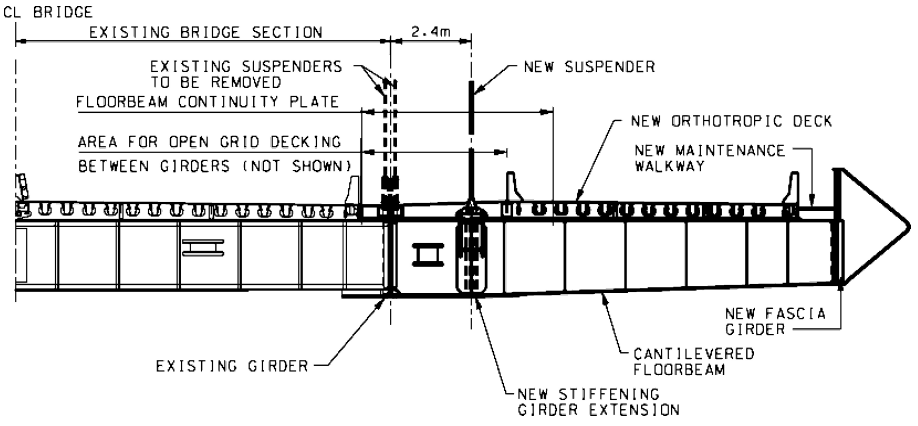


Figure 8. Cantilevered floorbeam extension.

The performance of the suspended tube (i.e., the structure formed by the orthotropic deck, stiffening girders, bottom laterals) to provide resistance to wind excitations will not be adversely affected by the introduction of the cantilevered floorbeams. The larger cables will somewhat increase the torsional resistance of the bridge to torsional excitation. However, wind effects on the structure are more critical with the addition of the roadway extensions. To offset this, vents could be installed between barriers of the inner and outer roadways to allow the bridge to "breathe"; which decreases wind effects such as flutter. This would be a measure that will be considered in conjunction with the re-installed existing wind fairings.

5.1.5 *Load transfer*

As the existing cable is unloaded, the existing cable will want to slide through its tower saddle. The cable will be clamped to the saddle and jacks will be provided to allow for the slow release of the friction forces after the cable is unloaded. After the existing cable is unloaded, suspenders and main cable strands can be removed from the suspended spans, and disconnected at the anchorages.

The process of load take-up on one cable and release from another must be accomplished in steps and needs to address vertical geometry differences that can be of several feet in magnitude. The profile change must be accounted for because it will create a kink in the roadways at the towers, and will affect utility conduits along the bridge. The magnitude of maximum profile change will determine the number of construction steps in the load transfer operation.

5.2 *Tower replacement*

The two alternatives based on tower replacement are essentially the same, except for how the new tower will be supported down to rock, and the composition of the tower, concrete versus steel. One alternative has a new foundation composed of drilled shafts, which allows for the use of concrete for the new tower, which is heavier but less costly. The other alternative relies on a deep and heavy capbeam on top of the existing tower piers to transfer the substantial new tower loads to the existing tower caissons. For this alternative, steel towers are proposed which are lighter but higher in cost.

The alternative that relies on the existing foundations is less attractive than the new foundation alternative because of the difficulty in constructing an adequate and serviceable cantilevered capbeam, and making the connection from the capbeam to the caissons. In addition, the mass of the capbeam itself imposes additional seismic forces into the existing caissons for which they are not designed. The new foundation alternative is considered the most promising of the tower replacement alternatives, and will be the focus of this discussion, Figure 9. This alternative provides a

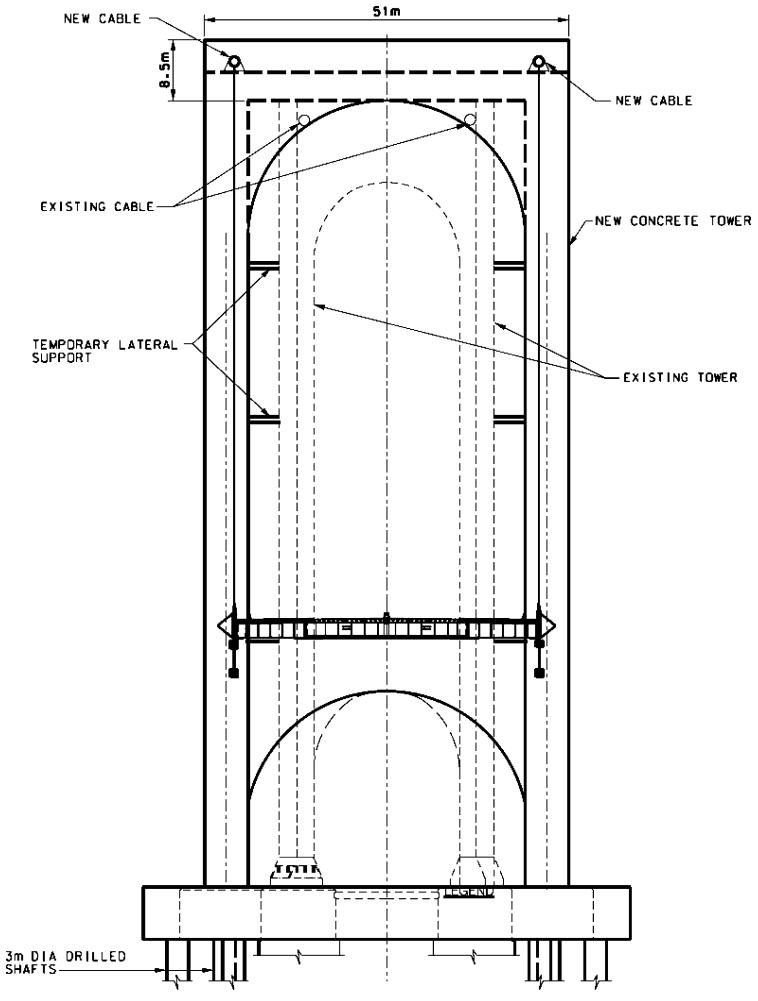


Figure 9. New tower with new foundation.

single deck that will carry four lanes with standard shoulders in each direction, separated by a median barrier.

5.2.1 *Towers*

The proposed new tower foundation consists of five 3 m diameter drilled shafts working as one group to support one leg of each tower, for a total of twenty drilled shafts for both towers. After the full vertical loads from the new tower legs are in place on the new foundation, the pair of drilled shaft groups will be bound together with the existing foundation with a new capbeam that will encompass the existing tower leg bases. The drilled shafts will carry the full bridge vertical load and the combined foundation will resist lateral loads.

The cable saddle elevation at the top of the towers would be 8.5 m higher than the existing saddle elevation, due to the new towers being taller than the existing towers. The new hollow concrete tower legs would be approximately 7.3 m $\times$ 7.3 m with 0.6 m to 0.9 m thick walls. The upper 4.6 m of the existing tower is non-structural and will be removed to minimize the height of the new tower, and facilitate the construction of the top strut. The existing tower will be utilized to laterally support the new tower and to provide working access during construction.

5.2.2 *Anchorages*

The new cables will be anchored in expanded Bronx and Queens anchorages that engage as much as possible the mass of the existing anchorages. The existing anchorage must be modified to accommodate the wider roadway and anchor the new cables. The anchorage would be widened by 10.7 m on each side, and extended past the existing anchorage approximately 10.7 m front and back.

The existing Bronx anchorage rests on a spread footing. The new cable anchorage will be placed about 4.6 m outside of the existing anchorage. The new cable pulling forces, which will approximately double the existing force, will transfer through the existing anchorage to the new combined spread footing partially through a solid reinforced concrete beam, constructed above the heel of the existing anchorage to connect the two new cable anchors, Figure 10. This beam provides enough weight to resist the bending moment created by the cable pulling force and transfers part of the horizontal component of the cable pulling force to the existing anchorage. A certain amount of cable pulling force will also transfer from the new concrete to the existing concrete through shear utilizing dowels.

The downward loads caused by the bending of the main cables over the splay saddles need to transfer partially to the existing anchorage concrete. In addition, the 4.6 m distance from the splay saddle to the face of the existing anchorage concrete generates a large moment which will be resisted with a tension tie and the anchorage footing. A prestressed concrete tie will be constructed between the concrete blocks where the existing cable anchors are located. Holes drilled through the anchorage under the existing cable anchors will allow the post-tensioning tendons to be installed. The post-tensioning will also generate shear friction between the new and old structures, helping to transfer cable force from the new concrete to the existing.

The Queens Anchorage is supported by a pair of rectangular caissons 10 m × 30.5 m and 52 m deep at the anchorage toe and a pair of circular caissons 7.3 m in diameter 48.8 m deep at the heel. All caissons are supported by rock directly. Three meter diameter drilled shafts are proposed to support the additional anchorage and resist the additional cable pulling forces. The drilled shafts are expected to resist the vertical load only. The existing rectangular caissons alone have enough capacity to resist the horizontal load.

The new saddles would be positioned below the deck level and as far as possible away from the suspended span to make the weight of the cable anchor concrete block more efficient at resisting the moment generated by the cable pull at the splay saddle.

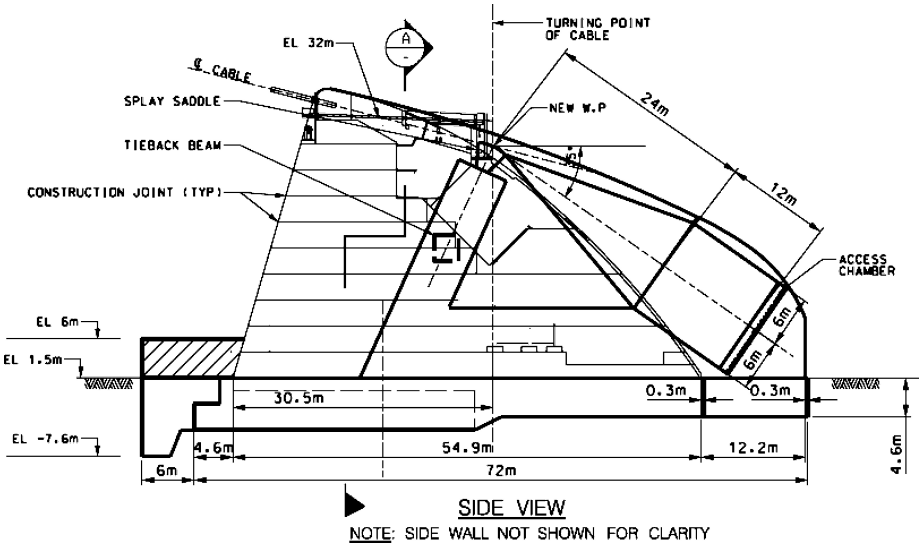


Figure 10. Bronx anchorage modification – new tower alternative.

5.2.3 *Suspension system*

The new cable location would be 4.6 m outside of the exterior wall of the existing anchorage. Considering both aerial spinning and prefabricated parallel wire strand (PPWS) as cable construction options, the following are potential designs for the new cables:

- Spun Cable – Thirty seven strands each with 306 wires, for an approximate cable diameter of 635 mm.
- PPWS Cable – Ninety One strands each with 127 wires, for an approximate cable diameter of 635 mm.

The 91 strands may be compacted slightly closer in the vertical plane than the 74 half strands of the 37 spun strands. However, the horizontal space between strands in the anchorages will be equally accessible for both strand types.

Saddles for the new cables will be somewhat larger than the existing saddles because of the larger cable. The saddles at the towers will be approximately 8.5 m higher than the present saddles. The added height of the tower saddles increases the sag of the cable, which helps minimize cable size as well as anchorage space. New suspender ropes would be the same as discussed for the other widening alternative.

5.2.4 *Floorbeam and deck extensions*

The new position of the cable 21.3 m away from the center line of the bridge engenders floorbeam moments in the order of 3.5 times greater than present. The floorbeam must be reinforced to triple the beam capacity at center span by either adding plates bolted through existing rivet holes, welding an inverted T to the bottom of the existing cover plate, or adding diagonals and a bottom chord and converting the floorbeam to a floor truss system. Converting the floorbeam to a truss system would provide greater structural capacity, but could encroach upon the clearance below the bridge.

The floorbeam moment capacity at the point of the existing stiffening girder must be enhanced through continuity of the floorbeam bottom flange and of the new orthotropic deck plate to existing orthotropic deck connection. The bottom connection can be made either below the existing stiffening girder flange, Figure 11, or by cutting holes through the girder and must be strong enough to support the dead load and the existing traffic until the existing suspender ropes are removed. Once the suspender ropes are removed, the existing stiffening girder top flange and upper part of the web will be cut to provide continuity of the deck plate. This can be done only after the existing suspenders are removed and the floor structure is supported by new suspenders at the outer and new stiffening truss or girder. This means that the floorbeam must have sufficient continuity to sustain moment from traffic on the existing deck, which is not possible without the aid of the orthotropic diaphragm and top plate. One possible solution to this is to erect new deck with holes around adjustable suspenders, which are connected below the existing suspender bracket. This allows the removal of the top part of the existing stiffening girder and the placement of a new deck that would lend strength to the retrofitted floorbeam. The retrofitted existing suspenders would then be removed and the deck retrofitted and bolted to the remaining part of the existing girder.

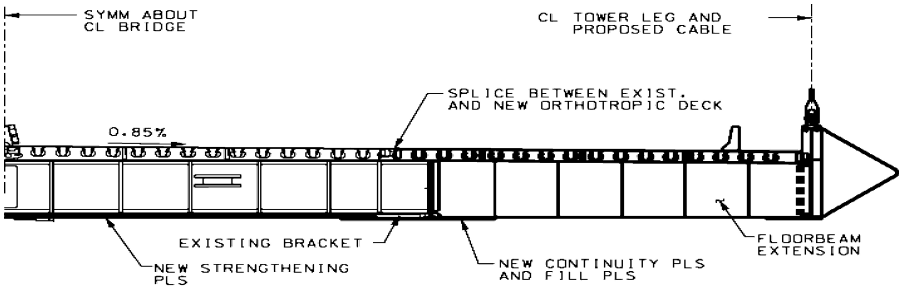


Figure 11. Tower replacement floorbeam extension.

5.2.5 *Load transfer*

The issues associated with the load transfer from the existing cable to the new cable are similar in nature to those discussed above.

6 SUMMARY AND CONCLUSION

Adding a second level and widening the bridge are both feasible solutions for increasing traffic capacity of the BWB. All of the feasible solutions require some acquisition of property near the anchorages, and require the replacement of the main cables. Each of the above discussed alternatives has advantages and disadvantages associated with it mainly concerning constructability and MPT issues and environmental/property impacts. The double decking alternative minimizes the impacts on the East River, but has the largest impact on the surrounding properties especially in Queens where the approach structures would have a second level added and the Queens Interchange would have to be re-configured. The double decking alternative also presents the most challenging constructability issues, given that all of the construction would be directly over traffic and due to the need to retrofit the entire height of the existing towers. Both widening alternatives minimize the impact to traffic, since most of the construction is outside of the existing traveled roadway and also minimize the impact to the surrounding properties since the approaches will require minimal modifications and the Queens Interchange would not need to be reconfigured. However, each widening alternative has varying advantages and disadvantages when compared to each other. The tower modification alternative re-uses the existing towers, minimizing impacts to the East River, but has constructability issues such as the retrofitting of the towers and the cantilevering of the floorbeams. The new tower alternative requires the construction of new tower foundations in the East River, but since it is new construction, constructability issues associated with the towers are minimal. There are, however, some constructability issues associated with extending the floorbeams to the new stiffening girder.

The results of these studies provide the data and evaluations necessary to make informed decisions about the future of the Bronx-Whitestone Bridge.

7 ACKNOWLEDGEMENTS

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8 DISCLAIMER

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Chapter 12

Bear Mountain Bridge cable reinforcement

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ABSTRACT: The Bear Mountain Bridge spans the Hudson River between Fort Montgomery, New York on the west and the mountain known as "Anthony's Nose" on the east, near Peekskill, New York. The bridge was completed in 1924 and was listed on the National Register of Historic Places on November 23, 1982. During an in-depth inspection of four panels of the main cables in 1991, numerous wires were found broken due to stress corrosion cracking and it was decided to rehabilitate the cables by unwrapping, oiling and rewrapping them. This work began in March 2000 on the southwest backstay. It was observed that the broken wire ends were moving further apart as the wedging and oiling progressed – an indication that the broken wires were slipping. Immediate action was taken to stabilize the backstay with clamping bands. Subsequently, the other three backstays were similarly clamped prior to allowing them to be unwrapped. The southwest backstay continues to be monitored by acoustic monitors that can detect breaking wires. While a few new wire breaks have been detected, the condition appears to be stabilized since the addition of the clamping bands. The New York State Bridge Authority, with Ammann & Whitney as consulting engineers, subsequently performed a detailed study and developed a design to replace the entire suspension system if it becomes necessary. As an interim measure, the most severely damaged section of the main cables, the southwest backstay, is currently being reinforced by the addition of supplemental structural strands that will be connected to the existing cable saddle and tower steel and to a new anchor frame on a new concrete block foundation anchored to bedrock with high-capacity rock anchors. Permanent strain gauges will be installed to monitor the loads in the rock anchors and structural strands. Field work began in March 2006 and will be completed in the fall of 2006.

1 INTRODUCTION

The Bear Mountain Bridge, Figure 1, spans the Hudson River between Fort Montgomery on the west and the mountain known as "Anthony's Nose" on the east, near Peekskill, New York. The bridge was designed by Howard C. Baird and construction was completed in 1924 by The Terry and Tench Company with John A. Roebling Sons Company responsible for the cable spinning. The monumental bridge's setting in a spectacular and historic spot in the Hudson River Valley adds to its significance as a landmark structure. Both the Bridge and toll house (present administration building) were listed on the National Register of Historic Places on November 23, 1982.

The bridge is a classical suspension bridge with a suspended main span of 497.56 meters (1632 feet) and simply supported steel truss side spans of 64 meters (210 feet). The west approach comprises a 30.49 meter (100-foot) long simply supported truss span and two 15.24 meter (50-foot) girder spans. At the time of its completion, the bridge's main span was the longest in the world. The 11.6 meter (38 foot) wide roadway carries two lanes of traffic.

The two main cables each contain 7,252 parallel galvanized wires of 4.94 mm diameter (0.195-inch, no. 6 gauge), compacted and wire-wrapped to a finished diameter of about 46 cm (18 inches).

Figure 1. Bear Mountain Bridge looking northeast.

In the anchorages, the cables splay out into 37 strands of 196 wires each, looped around semi-circular strand shoes that are pinned to eyebar chains. The eyebars are anchored inside tunnels that were dug into the natural rock and filled with concrete. The tunnels vary in length from about 27 to 34 meters (90 to 110 feet).

Sixty-three wire rope suspenders connect each cable to the stiffening trusses. Each suspender comprises two parts of 57 mm (2¼-inch) diameter wire rope that passes over the cable band. The lower ends of the suspenders are socketed and pin-connected to a steel assembly at the level of the truss bottom chords.

As discovered during an in-depth inspection of four panels in 1991 (Steinman 1991), numerous wires are broken due to stress corrosion cracking, and it was decided to rehabilitate the cables by unwrapping, oiling and rewrapping them. This work began in March 2000 on the southwest backstay. As longer lengths of the backstay were unwrapped, more broken wires were found than anticipated. It was also observed that the broken wire ends were moving further apart as the wedging and oiling progressed – an indication that the broken wires were slipping and not regaining tension due to friction (there were no cable bands on the backstays, only lightweight clamps supporting the handrope stanchions). There was further evidence of wire slippage and elongation of the backstay in the splay chamber of the anchorage. Immediate action was taken to stabilize the backstay with clamping bands. Subsequently, the other three backstays were similarly clamped prior to allowing them to be unwrapped. The work was then allowed to progress to completion.

The southwest backstay continues to be monitored by measuring the distance between the anchorage and the tower top, and by acoustic monitors that can detect breaking wires. While a few new wire breaks have been detected, the condition appears to be stabilized since the addition of the clamping bands.

American design practice uses the working stress method of design for cables. The design capacity is commonly defined in terms of a "safety factor," being simply ultimate strength divided by the unfactored working load (Fu/T). A safety factor below two is deemed unacceptable and might lead to traffic restrictions or closing of a bridge.

Based on the number of broken and corroded wires recorded during the oiling, it has been estimated that the strength of the cables has been reduced by 12 to 16%, and the computed safety factor for the southwest backstay (ultimate cable strength divided by working load) is currently around 2.4.

While the oiling and rewrapping may have slowed or even halted the deterioration of the cables, they will need to be monitored and periodically inspected to ascertain that additional damage is not occurring either through continued corrosion or propagation of cracks.

The New York State Bridge Authority retained Ammann & Whitney to develop alternative methods to strengthen or replace the main cables should it become necessary in the future. A peer review panel worked with the owner and consultant to assist in development of alternatives.

2 PROJECT OBJECTIVES

Because the long-term effectiveness of the oiling and rewrapping is yet to be determined and the extent and rate of wire cracking cannot be predicted, the Bridge owner needs to consider long term solutions that may be implemented in the future should the cables be deemed unreliable. The objective of the project therefore, was to investigate and develop options to restore the strength of the main suspension system should it become necessary to keep the bridge operating.

With the completion of the preliminary studies, the selected full cable replacement option was developed in sufficient detail to allow the owner to prepare and let contract documents for construction within a relatively short period of time.

As an interim measure, the southwest backstay is being supplemented with new strands that will relieve approximately eleven percent of the load. This will restore the backstay's safety factor to approximately 2.8 and will reduce the likelihood of additional wire breakage and slipping.

3 DESIGN CRITERIA

The two fundamental concepts studied were to strengthen the existing cables with supplemental material or to replace them entirely with new ones. A third, temporary concept was developed to reinforce only the southwest backstay to return it to a stress level similar to the rest of the cable and to provide for long-term monitoring.

Structural steel details, concrete structures and foundations were designed in accordance with American Association of State and Highway Officials (AASHTO 2002) Standard Specifications for Highway Bridges[2], supplemented by state and local design requirements. Cables, suspenders and appurtenances, which are not specifically covered by AASHTO, were designed in accordance with current US practice for new suspension bridges.

Supplemental cable alternatives provided for relief of at least one half of the suspended load from the existing cables.

4 DEVELOPMENT OF ALTERNATIVE CONCEPTS

As the Bear Mountain Bridge study advanced into the detailed design stage certain options were eliminated as permanent solutions early in the project. These are summarized below.

4.1 *Addition of individual wires or strands*

An obvious solution would be to add more healthy wires to the damaged cables, either individually or in bundles (strands). Because of the large numbers of wires required, the idea has been rejected as impractical as a permanent solution. It is immediately obvious that adding supplemental cables is a more practical way to accomplish this.

4.2 *Cable-stayed alternatives*

This is another solution that comes quickly to mind, but is similarly rejected when considered in any detail. The existing bridge towers are half as high as needed for a two-tower cable-stayed bridge with a similar span length. Because of this limitation stays can only be partially effective. In addition, the horizontal component of stays would need to be resisted either through compression at the truss level, or by balancing opposing stays through tension ties. The bridge is discontinuous at the towers and

would need extensive modifications to transmit the thrust of stays. Alternatives utilizing stays that are tied through the main span can be effective, but their behavior becomes that of point-loaded continuous cables, a solution better handled with supplemental suspension cables.

4.3 *Falsework alternatives*

One option is to temporarily support the bridge, remove the existing cables, erect new ones and transfer the load back to the new cables. Temporarily supporting the bridge on falsework is immediately rejected because any such solution would require large structures to be built in the river. Temporary suspension systems are the feasible alternatives for this option.

5 RETAINED ALTERNATIVES

Having rejected those alternatives deemed impractical, the study focused on the following three general scenarios:

Scenario 1: Leave the existing cables in place, add supplementary cables and transfer part of the suspended load to them. This achieves the immediate goal of reducing the load on the existing cables. The existing cables continue to carry a portion of the load and must continue to be maintained. Under the reduced stress the failure of additional wires due to stress corrosion cracking is presumed to be slowed or arrested. The remaining service life of the existing cables is still uncertain and continued monitoring will be necessary.

Scenario 2: Construct new cables, transfer the full suspended load to them and remove the old cables. This removes any uncertainty about the life of the existing cables. The new cables can be expected to perform at least as long as the original ones, and with new protection methods such as dry air injection, they can be maintained indefinitely.

Scenario 3: Construct temporary cables, transfer the entire suspended load to them, remove the old cables, construct new cables in the original location, transfer the load to them, and remove the temporary cables. This has the important advantage of maintaining the original appearance of the bridge but it is twice as costly as Scenario 2 and was ultimately dropped from further development.

The accepted long-term solution is complete cable replacement and this solution was advanced to detailed design. The following discussion covers the important features of this permanent long-term rehabilitation.

6 KEY STRUCTURAL ISSUES AND SITE CONSTRAINTS

6.1 *Towers*

The existing cable saddles completely cover the tops of the tower legs and there is no additional space to support new saddles. This limited the options for supporting new saddles. Options considered included bearing on the existing saddles and heavy bolted connections to the vertical faces of the tower legs. Ultimately, due to anchorage placement limitations, the most practical alternative is to locate new cables to the sides of the existing, supported on new tower extensions. The existing and modified tower configurations are shown in Figures 2 and 3.

6.2 *Anchorages*

New external anchorages may be constructed using mass concrete in excavated foundations, rock tunnels similar to the existing ones, anchor rods terminating in excavated galleries, or high capacity rock anchors placed in holes drilled into the rock. The quality of the rock at the site is excellent and will accommodate new anchorages of any type. The difficulty lies in the lack of free space, particularly at the southwest anchorage where the Bridge Authority's operations, toll and maintenance facilities are located and one existing anchorage lies beneath the 80-year-old stone masonry administration building, Figure 4.

Figure 2. Existing tower.

Figure 3. Tower modified to support new cables.

Figure 4. Existing southwest anchorage.

Figure 5. Existing southeast anchorage.

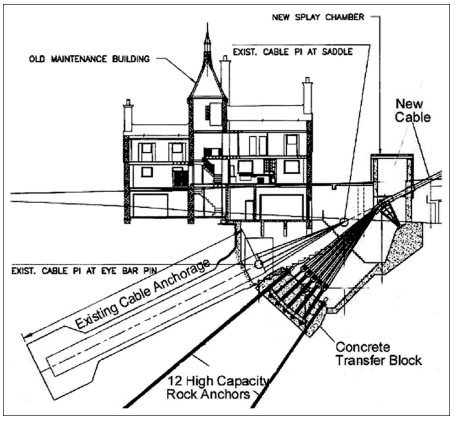

Figure 6. Existing and new southwest anchorage.

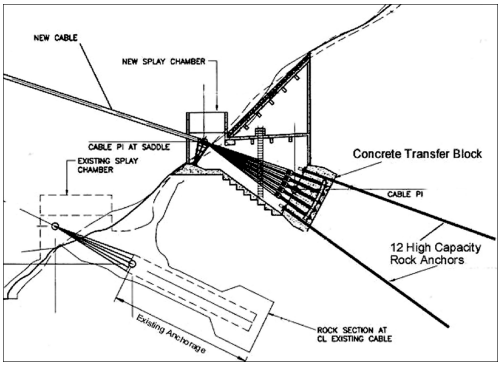

Figure 7. Existing and new southeast anchorage.

On the east side the existing anchorages are situated between the roadway of State Route 9D to their west and the National Guard's Camp Smith to their east, Figure 5. The terrain immediately east of the existing anchorages slopes sharply upward.

Due to the site characteristics at both ends of the bridge, compact anchor blocks utilizing high capacity rock anchors placed in the rock have proven to be the most practical solution. For the replacement cables, each anchorage will utilize twelve anchors with capacities of 5340 kN (1200 kips) each. The general details at each end of the bridge are shown in Figures 6 and 7.

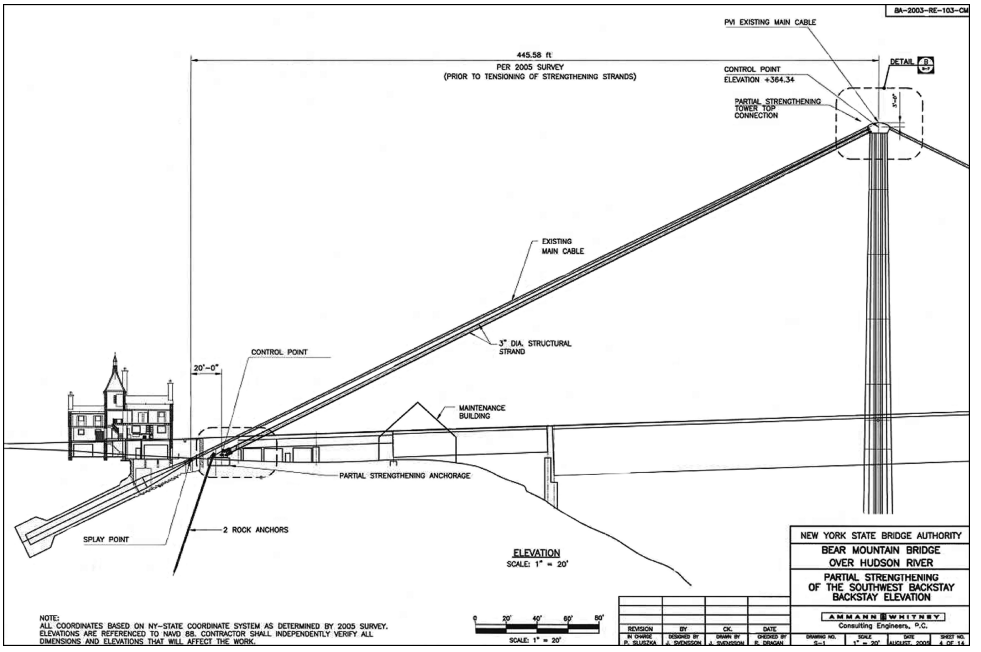

Figure 8. Partial strengthening overall configuration.

7 PARTIAL LENGTH STRENGTHENING OF THE SOUTHWEST BACKSTAY

7.1 *Structural design*

The southwest backstay has been shown to have the lowest factor of safety of any part of the main cables. The partial length strengthening of the southwest backstay has been designed to relieve approximately 14% of the dead load and 10% overall load from the existing backstay cable, thereby increasing its computed safety factor from 2.4 to about 2.8. This is to be accomplished by supplementing the SW backstay with four 3″ diameter galvanized structural strands. The overall configuration is shown in Figure 8.

The four strands will be tensioned to a load of 375 kips (1670 kN) each. The equivalent reduction in tension in the existing backstay increases its basic safety factor to approximately 2.8, which is equivalent to all other parts of the main cables. During the stressing of the new strands, the movement of the tower top will be monitored. There is no danger of the existing cable slipping in its saddle with this loading as sufficient friction exists.

The lower end of the strands will be anchored to a steel frame "saddle" supported on a reinforced concrete anchor block located to the east of the existing anchorage and socketed into the rock. The anchor block will transfer the loads to the rock through direct bearing and 2 high-capacity rock anchors designed for a capacity of 1,200 kips (5340 kN) each. Each rock anchor will be fitted with 4 strain gauges that will be tied into the internet-base monitoring system presently used for acoustic monitoring of the backstay cable.

The high capacity rock anchors were designed by Mueser Rutledge Consulting Engineers, as subconsultant to Ammann & Whitney, based on borings taken during the initial study phase of the project, Figure 9. The rock at the western anchorage is Precambrian age Granitic-gneiss and pegmatite. A core was taken near the southwest anchorage to a depth of 72.5 feet (22.1 m), Figure 10. Rock core recovery ranged from 92% to 100% of the core run and Rock Quality Designation (RQD) ranged

Figure 9. Boring rig at southwest anchorage.

Figure 10. Rock cores.

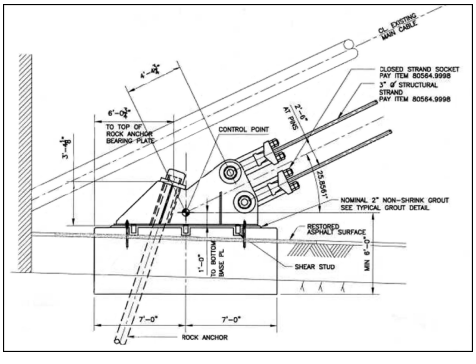

Figure 11. Anchor block, rock anchors and "saddle".

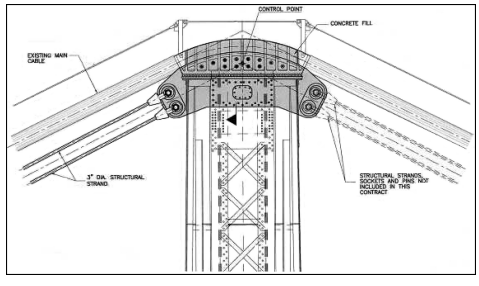

Figure 12. Connection at tower top.

from 72% to 100% of the run. The unconfined compressive strengths varied from 8.6 ksi near the surface to 20 ksi at the bottom of the core.

The anchoring system is located to avoid blocking access to the maintenance garages or interfering with the future cable replacement or supplemental cable schemes. The details of the anchorage system are shown in Figure 11. As shown in Figure 12, the upper ends of the strands will be anchored at the tower saddle. Jacks will be used to apply tension to the lower ends, which will transfer a portion of the backstay cable force to the strands.

The upper anchor point located at the tower saddle is designed using shear friction provisions as given by AASHTO (8.15.5.4)[2]. Large steel plates with headed shear studs are to be installed vertically at either side of the saddle. Concrete is then to be placed in the areas between the saddle and the plates. After the concrete has cured, a clamping force is applied to the system with tensioned threaded rods. The socketed ends of the strands are to be attached with pins to the steel plates. The original saddles were erected on roller nests that were subsequently "frozen" in place by a concoction of chemicals that would corrode them together, shown in Figure 13. Thirty two 1½ inch diameter bolts were also installed through the saddle base and the tower top plate, but this wasn't considered a completely reliable connection between the saddle and tower, so the new plates are also being bolted to the faces of the tower.

Strain gauges will be installed at the anchor rods that will permit long-term monitoring of the loads in the strands. If additional elongation of the existing backstay occurs, it will become evident by an

PLATE No. 62—After the completion of the super-structure, with the total weight of finished roadway suspended from the cables, the tower saddles were rigidly bolted to the tower tops, the rollers burned off even with the sides of the saddle, and, in order to prevent possible rotation, were frozen fast by the use of metal cement.

Figure 13. Original saddle rollers "frozen" in place and cut off flush with saddle.

Figure 14. Preliminary computer simulation at tower.

Figure 15. Simulation as seen from roadway.

increase in load in the supplementary strands. The gauges will be tied into the existing acoustic monitoring data acquisition system and will be monitored continuously. The monitoring system will be installed and maintained by Pure Technologies US Inc. under subcontract to Ammann & Whitney.

7.2 *Aesthetic and historical considerations*

Bear Mountain Bridge and the Toll House are listed on the National Register of Historic Places (part of the Hudson Highlands Multiple Resource Area). Appropriate measures needed to be taken to avoid, minimize or mitigate any adverse effects to the historical property and Louis Berger Group was retained as subconsultant to Ammann & Whitney to perform the environmental assessment and develop permitting requirements. Computer simulations were prepared to better assess the visual impact of the work and these visualizations were helpful in obtaining a finding of no significant impact from the New York State Office of Parks, Recreation and Historic Preservation. Examples are shown below in Figures 14 and 15.

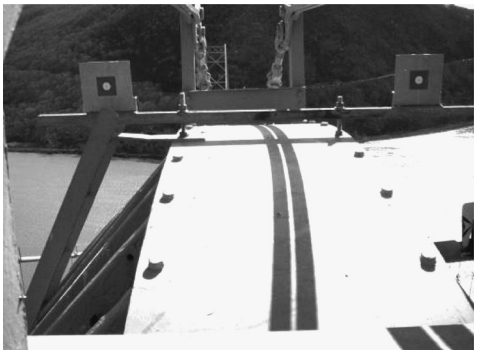

Figure 16. Survey targets at cable saddle.

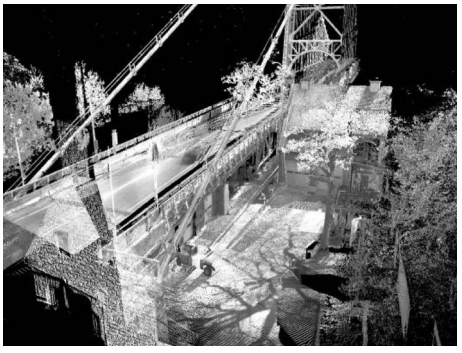

Figure 17. Plot of Lidar scan.

7.3 *Survey*

Prior to proceeding with the final design, it was necessary to obtain accurate dimensions and reliable geometry of the existing bridge. The original design drawings were base on the old New York Central Railroad Datum of the early 1920's. Subsequent surveys of the surrounding topography had been performed more recently, but there was no reliable tie between the old and new data. A first order accuracy survey was therefore called for and subcontracted to Greenman-Pedersen, Inc. The survey crew utilized a combination of conventional surveying, GPS positioning and Lidar (Laser Imaging Detection and Ranging) scanning to accurately locate the key points of the structure. A system of targets were installed by Bridge Authority forces to locate the cable PI at the tower top, the cable sag point at mid-span and other specific points needed for the detailed design. Lidar scans were used to produce "point clouds" of the existing cable, tower and anchorage that enabled our designers to find the coordinates of virtually any point on the structure. An example of one target location and a Lidar plot are shown in Figures 16 and 17.

7.4 *Monitoring system*

The southwest backstay is currently monitored to detect breaking wires by a continuous acoustic monitoring system operated by Pure Technologies US under subcontract to Ammann & Whitney. The principle of examining acoustic emissions to identify change in the condition of the structural elements is not new. However, until recently, continuous, unattended, remote monitoring of large structures was not practical or cost-effective. The availability of low-cost data acquisition and computing hardware, combined with powerful analytical and data management software, resulted in the development of a continuous acoustic monitoring system which has been successfully applied to unbonded post-tensioned structures in North America since 1994 and is now in use on several suspension bridges in the US and Europe. A typical sensor as installed on the Bear Mountain Bridge is shown in Figure 18.

The proprietary data acquisition software has been "trained" to recognize wire breaks. When events possess all the known properties of a wire break, they are classified as "probable wire breaks". Events possessing some of these properties are classified as "possible wire breaks". All other events are classified as "non-wire break events". By analyzing the time taken by the energy wave caused by the break as it travels through the cable to arrive at different sensors, the software is able to calculate the location of the wire break. Figure 19 shows the locations of wire breaks detected by the system since its installation.

The acquisition system includes a conventional telephone as well as a back-up cellular service. Data is transmitted over the Internet to a processing center in Calgary, Canada, where it is analyzed and archived. The data is also routed from the Calgary center to a second data processing computer located at the bridge administration building. This permits bridge operations personnel to review the data and reports using the same proprietary processing software in use at the processing center.

Figure 18. Acoustic sensor.

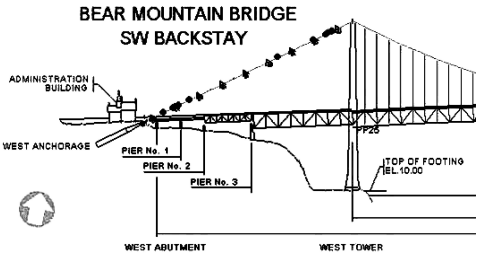

Figure 19. Wire break locations.

Figure 20. Anchor block, April 2006.

Figure 21. Anchor saddle, April 2006.

When a wire break is detected, the bridge owner and consultant receive e-mail notification. Since its installation in 2001, the acoustic monitoring system has detected 12 wire breaks. In the last two years, there have been none detected.

The strain gauges for the rock anchors and socket anchor rods will be monitored by a separate data acquisition computer that will be tied in to the existing acoustic monitoring acquisition system. While acoustic monitoring is continuous, the strain gauges will be monitored daily.

7.5 *Construction*

The contract to construct the southwest backstay partial strengthening was awarded to Piasecki Steel Construction Corporation of Castleton, New York in November of 2005.

Excavation and placement of the anchor block concrete was completed in March of 2006, Figure 20, and as of May 1, 2006, the steel "saddle" for the lower strand anchorage had been assembled and was in shipment for final welding and galvanizing at a plant in Buffalo, New York, Figure 21.

8 CONCLUSION

The Bear Mountain Bridge is an important link in the Hudson Valley Transportation corridor, as well as a national historic treasure. Although the main cables have been seriously damaged by corrosion induced cracking of wires, the condition has been stabilized for the near term by oiling and rewrapping them. Injection of dry air to completely preclude further corrosion is being considered,

and a long-term replacement solution has been designed and is available for quick implementation if necessary in the future. The current partial strengthening will help to further stabilize the southwest backstay and continuous monitoring of the cable load and detection of any new wire breaks will provide an additional level of security, allowing the bridge to continue operating normally for the foreseeable future.

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Chapter 13

Planning and design of Jeokgeum Grand Bridge

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ABSTRACT: The Jeokgeum Bridge (Sag ratio = 1/9) is a suspension bridge with 850 m main span, a concrete pylon and stiffening girders. The completed bridge will support 2 motorway lanes. However, its girder design is generally considered for the future extension to 4 lanes due to increasing traffic. The diameter of its cables is 491 mm, consists of 380 bundles of parallel wires and 19 strands, the height of pylon will be 137.8 m. Its span-length-to-width ratio of 43.15, which constitutes an important wind design parameter, is the fifth largest ratio compared with the world's largest one of Huga Kusten Bridge (ratio of 55). In this way, this paper presents various design features of the Jeokgeum Bridge.

1 INTRODUCTION

The Jeokgeum Bridge is a suspension bridge located in Goheung peninsula on the southern coast of Korea. It is the 11th bridge involved in the Yeosoo-Goheung sea belt construction project shown in Figure 1. Following to the 2003 years Yeosoo-Goheung sea belt construction project, it will be firstly constructed bridge which has an 850 m single suspended span. The important technical design features of Jeokgeum Bridge are 1) its length-to-width ratio (L/B) of 43.15 (=850/19.7), which is a major measure of the aerodynamic stability 2) a saddle setback system considering the structural character of the concrete pylon 3) the simple shape of a pylon without horizontal diaphragms in pylon leg 4) an optimized anchorage foundation considering the site conditions.

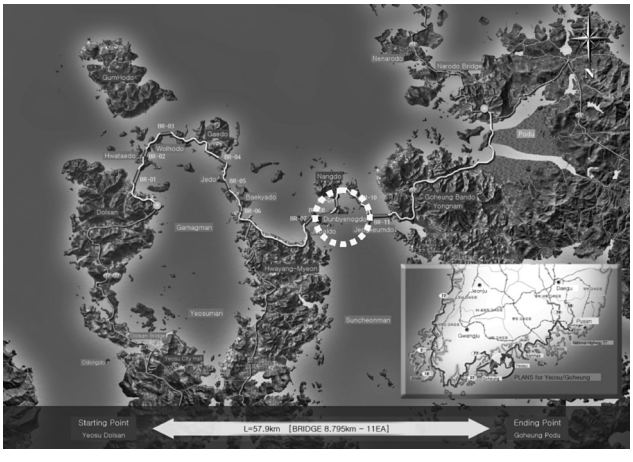


Figure 1. Yeosoo-Goheung belt way construction project.

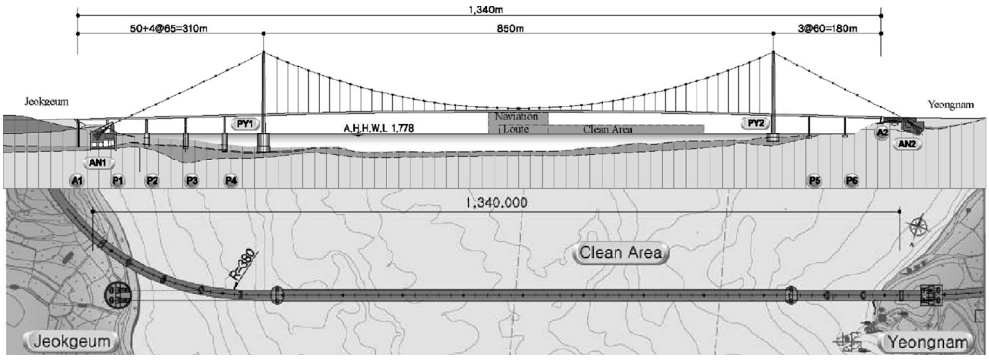


Figure 2. Bridge layout.

Table 1. Structure type of Jeokgeum Bridge.

Structure	Type			Width (m)	Span length (m)
	Superstructure	Substructure	Foundation		
Approach (Jeokgeum Side)	Steel composite bridge	Pier Inverse T-shape abutment	Pile/Open caisson	15.500	50 + 4@65 = 310
Main span	Earth-anchored suspension bridge	Inclined tower	Open Caisson	19.700	850
Approach (Youngnam Side)	Steel composite bridge	Pier Inverse T-shape abutment	Footing	15.500	3@60 = 180

2 BRIDGE GEOMETRY – GENERAL

As shown in Figure 2 and Table 1, the total length of the Jeokgeum Bridge is 1340 m including the 850 m of the main suspended span and both approach bridges of 310 m and 180 m. The approach bridges are traditional steel box girder composite bridges. The sag ratio is 1/9.

The completed bridge will support 2 motorway lanes and 2 walkways, which is generally considered adequate for the low traffic density of the Korean rural area. However, design check has been performed (Table 2) with respect to the design requirements demand for a future extension to 4 lanes and 2 walkways as illustrated in Figure 3.

The distance between the cable planes is 17.5 m and the total width of the girder is 19.7 m. A particular feature of the Jeokgeum Bridge is its length-to-width ratio (L/B) of 43.15 (=850/19.7), which is a major measure of the aerodynamic stability and reveals the remarkable slenderness of the suspended span.

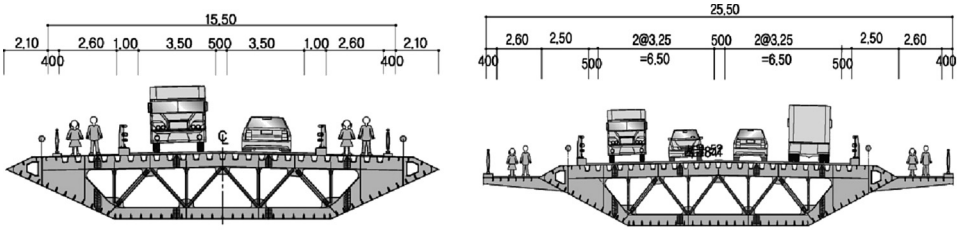
3 STIFFENING GIRDER

3.1 *Design aspects*

The stiffening girder is made by steel plate with 14 mm thickness, based on the Korean specifications of minimum one. The other parts are composed by steel plates with thickness ranging from 10 to 14 mm and is stiffened longitudinally by trapezoidal ribs of 8 mm. Diaphragms of 10 mm are also installed every 3.5 m and the hangers are disposed with spacing of 17.5 m. The design results of the deck are listed in Table 3 and the cross-section is illustrated in Figure 3.

Table 2. Design conditions.

Design Load	Dead load	Deck	118.280 kN/m	Lanes/Design speed			2 lanes, 70km/h
		Cable	28.655 kN/m				
		Total	146.935 kN/m	Vertical gradient			2.7%
	Live load		DB24/DL24				
	Wind load	Wind speed	40.0 m/s	Clearance			AHHWL+35m
		Design speed: Deck	56.2 m/s				
		Design speed: Cable	62.8 m/s	Structure	Deck	Width	19.7 m
						Height	3.0 m
		Design speed: Hanger	60.8 m/s		Cable	Sag ratio of main cable	1/9
		Design speed: Tower	64.9 m/s			Space between cables	16.5 m
	Seismic load		Jeokgeum Grand Bridge Specification			Number of Main Cable	1 EA/each side



(a) Cross-section at completion (2 motorways)

(b) Future cross-section after extension (4 motorways)

Figure 3. Cross-sections of stiffening girder.

Table 3. Design results of the deck.

Deck		
Section	Thickness of slab	14 mm
	Material (steel)	SM490
Deformation	Vertical	2.9845 m
	Horizontal	5.6804 m
	Angle of vertical refraction	0.01768rad

External corrosion protection will be provided by a double layered zinc coating of 50 μm, and 2 layers of high-performance corrosion resistant paint for a total thickness of 225 μm. This corrosion protection conforms to the prescriptions of the Korean specifications and has been applied in most of the domestic sea-crossing bridges. Dehumidifiers will be installed inside each end of the girder in order to control the relative humidity below 40% so as to prevent the development of corrosion. A simple prime coating will be applied in factory for the surface treatment inside the stiffening girder.

The traffic lanes will be paved with 80 mm thick asphalt-wearing surface. The weight applied on the stiffening girder will reach 103.5 kN/m including the wearing surface, railings and secondary dead loads. The weight of the steel girder itself is 74 kN/m. The steel quality of the main structural members is SM490 (yielding stress, 310 MPa).

3.2 *Construction stage analysis*

Since the main cables of the suspension bridge experience large deformation throughout the erection of the stiffening girder, detailed analysis has been performed to prevent the interference between the members with respect to the displacement of the structural system occurring at each construction stage, and also to detect the local slip of the cables and verify the satisfaction of the allowable tension forces at the top of the hangers and the allowable secondary stresses of the main cables.

Before the determination of the block lengths and corresponding erection sequence, careful analyses were carried out to determine the relations between the length of the sections to be erected and the distributed load in the hangers resulting from the suspension of the section on the main cables. The analysis results revealed that, in the case where the first block of 35 m (corresponding to the size of 2 panels) is suspended at mid-span and erection proceeds symmetrically by sections of 35 m toward the pylons (corresponding to 2 pairs of hangers per section), the tension force in the hangers and the additional stress due to local bending in the main cables satisfy the allowances.

4 PYLONS

4.1 *General*

The pylons, extending to a maximum height of 137.8 m, are plane frame reinforced concrete structures with legs slightly inclined in the transverse direction and connected by two cross beams, as illustrated in Figure 4. The legs are exhibiting rectangular hollow section with corner cuts and the cross-section is constant along the whole length except for the out-of-plane section depth (parallel to the bridge axis), which varies linearly from 6.5 m to 4.0 m.

The pylons are occupying an important structural part of the bridge and are therefore influencing significantly the aesthetical aspect of the suspension bridge. Following, their design was evaluated in close cooperation with a bridge architect.

The design motive pictures a traditional stone gate representative of the regional cultural heritage. The straight appearance of the pylons endows people crossing the bridge with the feeling of ascending and open-scenery. The pylons are installed on circular caissons (Φ30 m). The caisson of pylon PY1 rises up to 31 m and is built by cast-in-place concrete on the rock bed. The caisson of pylon PY2 reaches approximately 10 m because of the presence of shallow rock in seabed. This makes it possible to execute its fabrication in the yard and to convey it by a barge.

4.2 *Design aspects and analysis*

Considerable efforts were made to reach a reasonably optimal pylon geometry and reinforcement magnitude. A number of alternatives were considered and evaluated before making the final choice, which was also influenced by aesthetical aspects (e.g. the presence of a cross beam, the shape of column cross section, the appearance of the pylon base, etc.).

Efforts were also made to arrive at a construction – friendly design. Aspects simplifying form work were for instance:

– Gradual change of wall thickness
– No horizontal diaphragms in pylon leg in the section constructed by auto-climbing form

The pylons were analyzed and designed for relevant loads, especially wind load. Wind data with 100 and 5 years return periods were specified for the completion and for the construction term (prior to the installation of the main cables), respectively. Under construction, the pylons act as cantilevers in the longitudinal bridge direction. At this stage, the pylon will present the most slender structure and will be extremely vulnerable to wind. Thereafter, before the construction of the upper cross beam, the pylon legs act also as cantilevers but in both directions. Consequently,

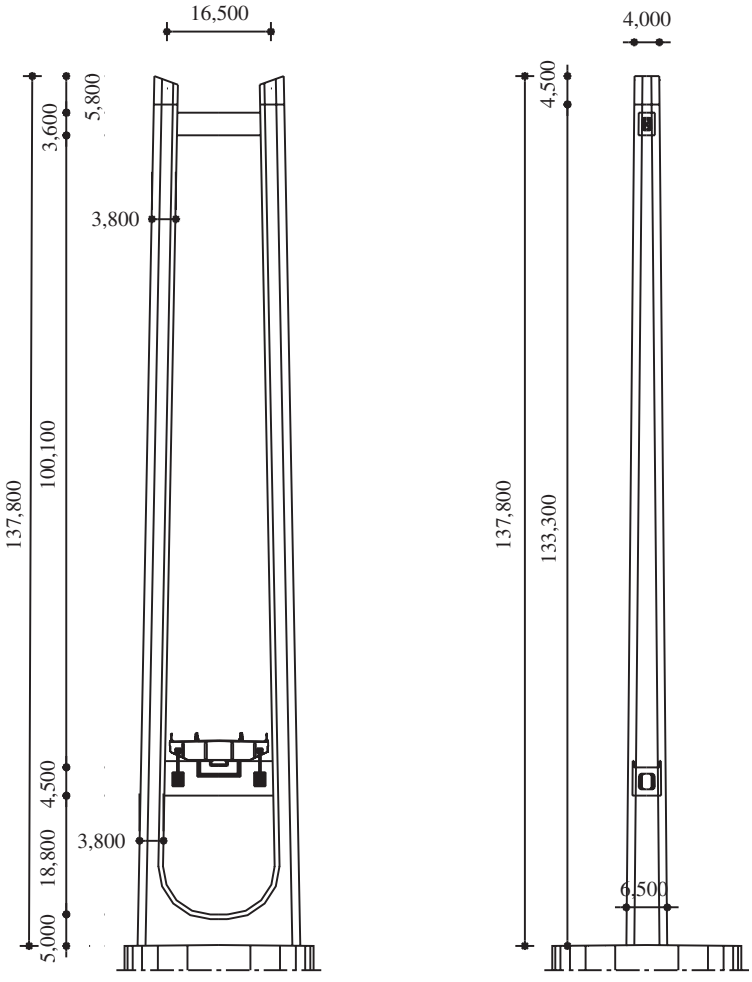


Figure 4. Pylons.

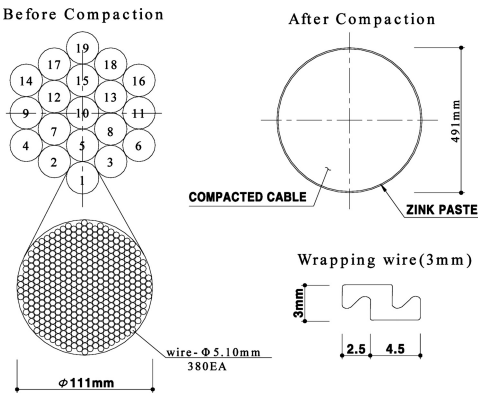
decision was made to install an auxiliary cross beam at the mid-height of the pylon leg during the construction stage.

5 CABLES

The main cables are the most important structural elements of a suspension bridge, which transfer the loads to the anchorage through the pylon. For technical as well as economical reasons, air-spinning of the former alternative has been chosen for this bridge.

The replacement of the main cables of a suspension bridge being tremendously unaffordable, it is of the utmost importance to design the strands and elements connected to the cables so as to prevent corrosion and damages. Table 5 summarizes the details of the main cables of Jeokgeum Bridge. The adoption of high strength steel wire in the design made it possible to improve the economic efficiency by reducing the cross-section of the cables and minimizing the size of cable subsidiaries such as bands, sockets and saddles. The allowable tensile force per cable runs around 100 MN. The cables are installed by the aerial spinning method. During the last decade, the aerial

Table 5. Details of cables.

	Type	Specifications	Cable section
Strand	Material	KSD 3509 piano wire rods	
	Tensile strength	1770 N/mm^2	
	Allowable strength	708 N/mm^2	
Cable	Section	380 wires × 19 strands	
	Diameter	After wrapping: 491 mm (K = 20%)	
	Area	0.1475 m^2	
	Wrapping wire	S type (3 mm)	
Hanger	Breaking strength (Shearing load)	Φ5.0 mm-151strands (Φ80) → 4657 kN Φ5.0 mm-163strands (Φ85) → 5024 kN	

spinning technology has improved considerably in terms of cost and time consumption through the experience gained in the construction of Youngjong and Gwangan bridges. The spinning system proceeds by the installation of the wires, one by one, using a specially designed suspended spinning wheel controlled by a support system. A predefined tension is introduced in advance after the installation of each wire. During spinning, the wires are constituted by 19 strands consisting of 380 wires.

A major decision to be fixed in the design of aerial-spun cable is the selection of the number of strands. The number of 19 strands has been selected in this design because it eases the layout of the strands in the saddle trays, allows suitable shape of the cable prior to compaction and leads to reasonable amount and size of anchoring points in the anchorages. Moreover, the wire diameter of 5.1 mm is exhibiting remarkable applicability for commonly available spinning equipment.

After installation of all the strands, the cables are compacted into a circular shape before fastening of the cable bands to the cable by bolts. The surface of the cables is then sealed by a zinc paste and wrapped with a 3 mm galvanized wrapping wire (S type). Finally, the cables are surface coated using a flexible fluorocarbon resin.

The hanger system connecting the stiffening girder and main cables shall resist to the vertical force induced by the dead load and traffic load as well as the horizontal force due to wind. The hanger system is composed by the cable bands, hanger cables, and attachment devices. The cable bands secure the position of each wire in the main cables and prevent slip between the cable bands and the main cables. Plan has been established to ease the erection of the hangers by applying pin type cable anchors and prefabricated strand cables.

6 SADDLE

The cables, continuous over the pylon tops, are supported on curved saddles. These ensure that the cable curvature and transverse surface pressure, equilibrating the vertical component of the cable force, do not exceed acceptable specified values.

The saddles were designed to be manufactured as a welded plate structure instead of cast-iron structure for reducing the manufacturing cost. The curved 120 mm thick steel plates, on which the cables are placed, are supported on the assemblage of steel plates. In order to allow for adjustments of the saddle location, relative to the tower in construction stages, a plain, low-friction bearing is provided at the saddle base. The bearing consists of a polished stainless steel plate anchored to the saddle block, and sliding on a Teflon layer fixed to a steel slab anchored to the top of the pylon.

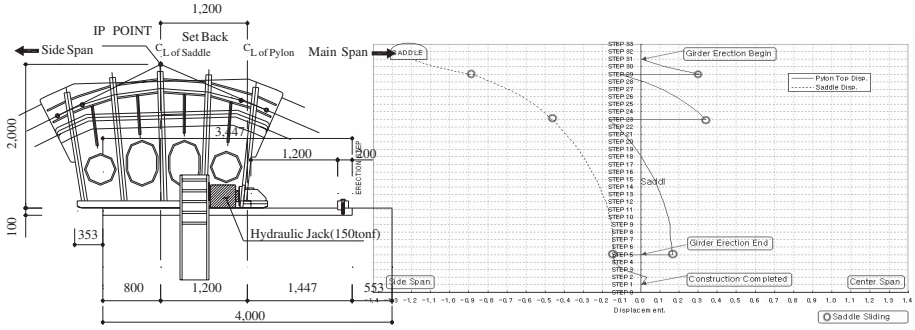
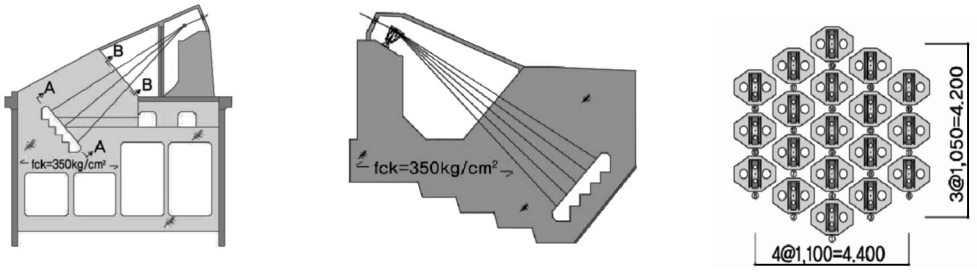


Figure 5. Displacement history of the saddle and top of pylon during construction.



(a) Jeokgeum side (b) Youngnam side (c) splay chamber

Figure 6. Anchorages.

Two 1500 kN hydraulic jacks, placed on each side of saddle, will be used to achieve the desired saddle displacements. Once the saddles are disposed in their final position, the jacks will be removed and the two bearing plates will be welded together.

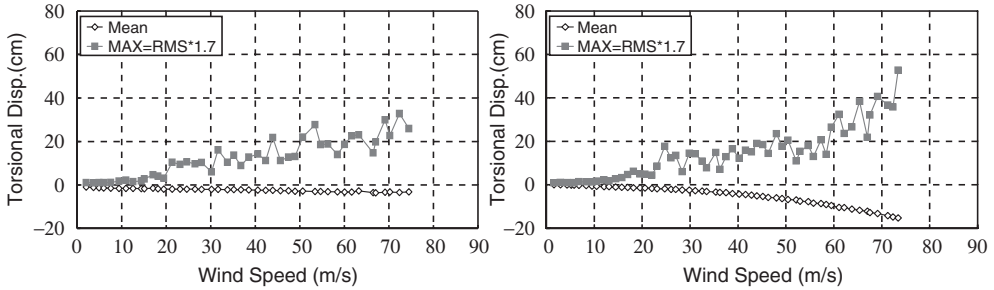
During the installation of the stiffening girder segments, the cable forces increase as the installation progress. As a consequence of the resulting elastic elongation and reduced sag of the side span cables, the tops of the pylons are displaced toward the main span. To reduce this displacement, the saddles, relative to the pylon tops, are gradually jacked toward the main span. According to the results of the calculation, the saddles on the Jeokgeum island side pylon are placed with an initial eccentricity (toward the side span) of 1200 mm. After the completion of the girder and the application of all other dead loads in the main span (asphalt, railing etc.), the saddles are relocated to the center of the pylons, and the pylons become vertical. The displacement history of the saddle and top point of pylon during construction are shown in Figure 5.

Since the cables at proximity of the saddle are not accessible for maintenance, dehumidification systems are placed around the saddles to maintain the humidity below 40%.

7 ANCHORAGES

Both anchorages illustrated in Figure 6 are massive concrete structures, but with different designs. The anchorage 1, on the Jeokgeum side, is a pure soil-anchor and designed as a cylindrical embedded structure. The anchorage 2, on the Youngnam side, is integrated in the rock and the horizontal component of the cable forces are transmitted to the rock via four vertical steps.

The anchorage consists of 3 main parts: the splay chamber, the prestressed concrete transmitting block, and the anchorage chamber.



(a) Wind attack angle of 0° (b) Wind attack angle of +3°

Figure 7. Two-dimensional wind tunnel test results.

In the splay chamber, the main cable, consisting of a bundle of 19 individual strands, is spread out and the individual strand directions are deflected over a splay saddle. The splay chamber is equipped with a dehumidification system in order to minimize maintenance requirements in this critical zone. The individual strands are anchored to the strand shoes which are installed in front of the prestressed concrete transmitting block. Forty four PC strands are embedded in the transmitting block and are stressed at the anchorage chamber.

8 DYNAMIC ANALYSIS – WIND MEASUREMENT AND DESIGN

Wind loads become the dominant load during the design of long-span suspension bridges like Jeokgeum Bridge. Careful investigation regard to wind load effects is thus essential. The wind load investigations included wind tunnel tests with both a full model (3-dimension) and a sectional model of the bridge (2-dimension) in addition to the theoretical analysis. The shape of the stiffening girder is of decisive importance both for the magnitude of wind loads acting on the bridge and for the dynamic properties of the bridge. Wind tunnel test of the selected stiffening girder verified that the shape of the girder exhibit remarkable aerodynamic stability and relatively small generation of wind loads.

Stochastic analyses, taking into account dynamic properties of both the wind (turbulence) and the structure, were carried out to determine wind load effects in the stiffening girder due to wind acting transversally on the bridge. A computer program, ALVSAT (Analysis of Lateral Vibrations of Suspension Bridges due to Atmospheric Turbulence), was used for this purpose. ALVSAT has been developed by SINTEF at Trondheim, Norway, and has already been successfully applied to the design of suspension bridges such as Askøy Bridge and Hardenger Bridge. Wind load effects due to the dynamic portion of the wind were found to reach approximately 90% of the static response. Total wind load effects in the girder, given by the sum of the two effects, may therefore be addressed as the static load effects times a dynamic amplification factor (DAF, or Gusting factor) of about 1.9.

The aerodynamic stability of the girder can be expressed through the flutter speed, which is a theoretical index indicating the critical wind speed at which the suspension span may start uncontrolled oscillations. These oscillations, due to a combination of vertical and torsional oscillation modes, may, if they are allowed to develop, result in girder collapse. The example of Tacoma Narrows Bridge is representative of such flutter-induced collapse. The critical wind speed can be derived from the frequency domain analysis using the flutter derivatives (R.H. Scanlan, 1990). The lowest eigen frequencies of vertical and twisting vibrations are respectively 0.182 Hz and 0.472 Hz. In case of a wind attack angle equal to 0°, the flutter phenomena does not happen within the design wind speed range. But if the attack angle reaches +3°, the twisting flutter for the 1st twisting mode happens at wind speed of about 60 m/s. Figure 7 shows the 2-dimensional wind tunnel results.

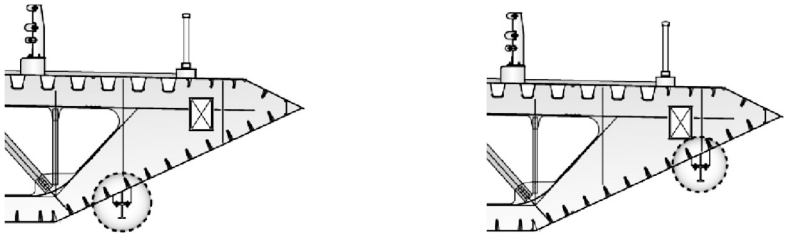


Figure 8. Location of inspection car rail.

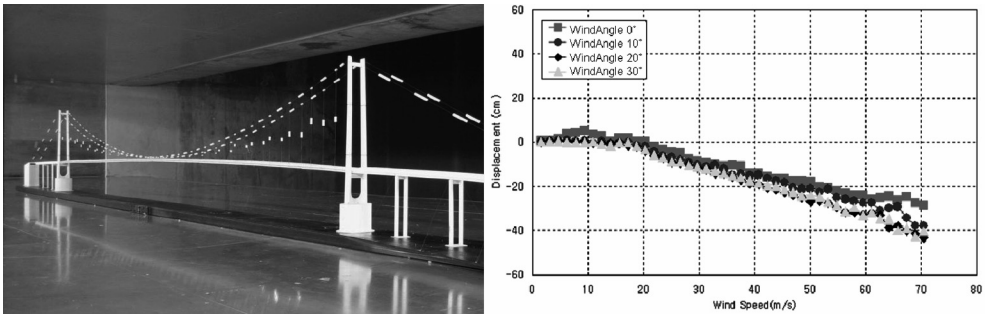


Figure 9. Three-dimensional wind tunnel test: test setup and results.

To establish reliable shape factors for the girder cross-section, wind tunnel tests were also carried out on cross section models with and without future walkways. Since the wind tunnel test revealed that the location of the inspection car rail affects the aerodynamic stability, the rail was determined to be installed at the vicinity of the fairing as shown Figure 8.

The shape factors of lift, moment, and drag for the selected section are 0.14, 0.91, and 0.01, respectively. In order to verify the dynamic analysis, wind tunnel tests of a 1/200 scale bridge model were carried out as shown in Figure 9. Uniform flow test was implemented at the University of Tokyo to find the static deflections according to the wind direction. The maximum horizontal deflection under the design wind speed appeared to be 6.3 m at mid-length of the suspended span. The boundary layered laminar flow test considering aerodynamic effect was performed at Hyundai Institute of Construction Technology. The maximum horizontal deflection under the design wind speed consisted of the static deflection (4.2 m) and buffeting (1.35 m). This buffeting response satisfied the serviceability recommendations of ASCE (1981).

9 CONCLUSION

A brief overview of various design aspects of Jeokgeum Bridge like the design loads, the design of the main structural elements and the wind design has been addressed. Regard to the scale and characteristics of the bridge, the design applied not only the design procedure of the Korean standard specifications but also various foreign design methods like as GRatbelt East, Askey Bridge in Europe, Okland Bay Bridge in USA, Kurusima, Akynada Bridge in Japan. In view of the structural safety, considerable efforts were endeavored during the design to provide solutions able to ease the construction and realize economic efficiency.

The design and construction of previous suspension bridges offered the opportunity to gain valuable experience for the Korean bridge engineers and helped to complete successfully the

design of Jeokgeum Bridge. Jeokgeum Bridge is currently under erection according to the schedule and no particular problem has been reported to date. We are now looking forward the achievement of the bridge that is the fruit of our efforts.

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5 Rehabilitation and preventive techniques

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Chapter 14

Rehabilitation of the lower roadway of the Manhattan Bridge

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ABSTRACT: Once completed, the Reconstruction of the Manhattan Bridge Lower Roadway will finalize the rehabilitation of all roadway levels on this famous East River suspension bridge. The advanced deterioration of the deck, and steel framing required a complete replacement of these elements, which created complicated construction staging/maintenance of traffic issues. If the Lower Roadway were closed to traffic, only two lanes would be available to motorists in the peak direction for the morning and evening rush hour traffic, a configuration which had never been used on this structure previously. A study was undertaken with two separate construction staging/MPT alternates; Complete Closure of the Lower Roadway, and Partial Closure of the Lower Roadway using Temporary Ramps. A full traffic study was undertaken to determine the effect that each of these alternates would have on the traffic flow and possible measures which could be used to mitigate the adverse impacts. This paper will present the some of the complicated planning issues associated with each of the two concepts studied.

The Manhattan Bridge, constructed in the early 1900s and opened to traffic in December 1909 was the last of the three great suspension bridges constructed across the East River. Today, this bridge is one of the most heavily traveled East River crossings carrying more than 78,000 vehicles and 350,000 people over the bridge's seven vehicular roadways and two subway lines which connect Canal Street in lower Manhattan to Flatbush Avenue in Brooklyn. Engineered by Leon Moisseiff (1872–1943) and fitted with a splendid set of approaches designed by the renowned architectural team of Carrere and Hastings, the Manhattan Bridge is one of the most aesthetically pleasing of New York City's transportation structures, Figure 1.

The bridge has three vehicular roadways: the North Upper Roadway which carries two lanes of traffic to Manhattan, the South Upper Roadway which carries two lanes of traffic to Brooklyn and the Lower Roadway which carries three lanes of reversible traffic (i.e. traffic can be switched to either direction dependant upon traffic demands). Under Contract 8C the South Upper Roadway and south transit level were rehabilitated; under Contract 10 the reconstruction of the North Upper Roadway and North transit level were completed. The reconstruction of the Lower Roadway is to be completed under Contract 11.

1 THE PROBLEM

Similar to the other East River Bridges, the Manhattan Bridge has undergone numerous major rehabilitation contracts during the past 25 years. Once completed, the rehabilitation of the Lower Roadway will finalize the reconstruction of all roadway and transit levels on this famous suspension bridge. However, unlike the other construction contracts undertaken on this structure, the reconstruction of the Lower Roadway presented structural and operational difficulties that were not present during the reconstruction of the North and South Upper Roadways. The advanced deterioration of the existing grid deck, floorbeams and stringers required a complete replacement of these elements, which created complicated construction staging and maintenance of traffic issues. The replacement of the existing structural steel framing in-kind would require a complete closure of the Lower Roadway specifically for the replacement of the floorbeams which span from Truss B to Truss C,

Figure 1. Lower roadway of the Manhattan Bridge.

thereby crossing the entire width of the Lower Roadway. Two construction staging options were initially discussed; Complete Closure of the Lower Roadway during Construction and Partial Closure of the Lower Roadway through the Use of Temporary Ramps. It was not readily apparent which of these options offered the best short and long term benefits to the NYCDOT. If the Lower Roadway were permanently closed during construction, only two lanes on the Upper Roadways would be available to traffic in the peak direction for the morning and evening rush hours – a traffic configuration never previously used on this bridge. Under the previous construction contracts, a minimum of three lanes of traffic were made available to peak hour traffic using one of the upper roadways and a reversible lane on the Lower Roadway. If the partial closure option were to be instituted, the construction duration and initial construction cost would be substantially increased, and the disruption to the traveling public would be significantly longer in duration.

2 PRELIMINARY EVALUATION

The NYCDOT asked Weidlinger Associates to prepare a preliminary evaluation of the two construction staging options noted above. During the initial screening process, the NYCDOT Office of Construction Mitigation and Coordination (OCMC) requested an additional partial closure option be evaluated. Thus, the partial closure option was divided into two sub-options; the maintaining of two

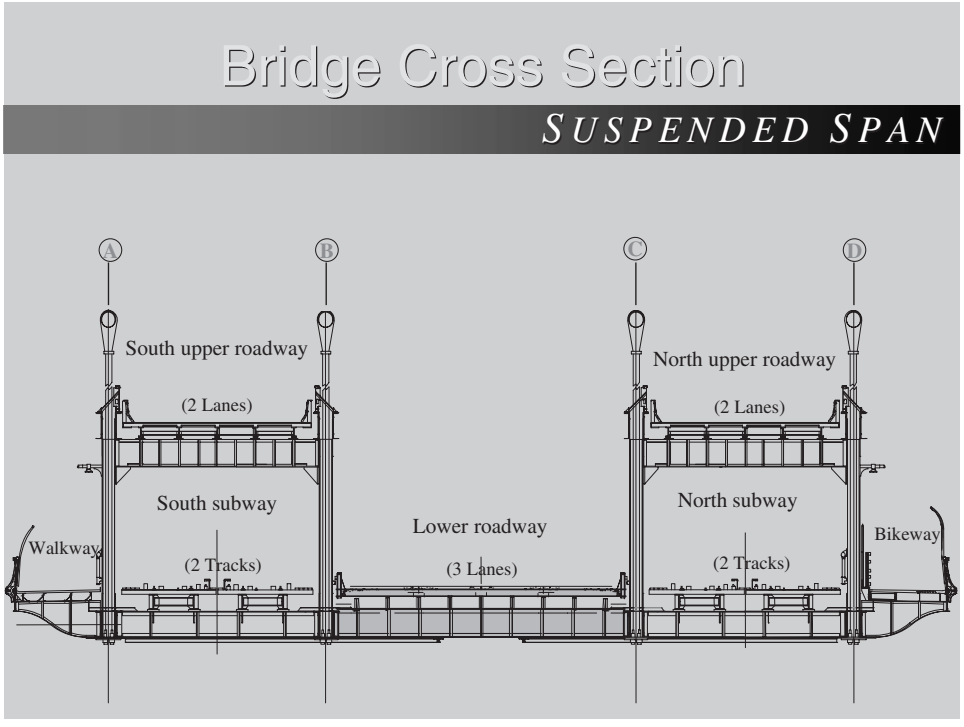

Figure 2. Typical bridge cross section – suspended spans.

lanes of traffic on the Lower Roadway during peak hours and the maintaining of one lane of traffic during peak hours. Therefore, the preliminary evaluation considered the following three options:

Alternate 1 – Reconstruction of the Lower Roadway with complete closure.
Alternate 2 – Reconstruction of the Lower Roadway while maintaining a minimum of two lanes of traffic during peak travel periods on that roadway (Manhattan-bound during the morning and Brooklyn-bound in the afternoon).
Alternate 3 – Reconstruction of the Lower Roadway while maintaining a minimum of one lane of traffic on that roadway during these peak travel periods.

3 STRUCTURAL CONFIGURATION OF THE LOWER ROADWAY

The key element driving the construction staging evaluation was the structural configuration of the Lower Roadway and the ability to replace the entire deck and framing system without complete closure. The configuration of the Lower Roadway in the suspended spans is depicted in Figure 2. As shown in the figure, the typical Lower Roadway framing system is as follows:

- A 3-1/2" deep concrete filled grid deck supported on 10" crossbeams.
- The crossbeams are supported on nine – 18" deep stringers spanning longitudinally between floorbeams, which are located at each truss panel point.
- The floorbeams, which are built-up riveted plate girders with a variable depth web, are spaced at 18' on-center. The floorbeam depth is variable to provide a circular roadway crown.
- The floorbeams are framed into Truss B and Truss C at each truss panel point; into the stiffening truss on the suspended spans and into the similar longitudinal trusses on the approach spans.

The proposed construction calls for the replacement of the entire Lower Roadway framing system. The new deck and framing system will consist of the following:

- A 5-1/16" half-fill grid deck with a 3/8" microsurfacing overlay.
- Eight new W 21 stringers with elastomeric bearings on the floorbeams.
- New welded plate girder floorbeams – same size and configuration as the existing.

4 CONSTRUCTION STAGING ALTERNATES

Replacement of the existing framing system presented many difficulties. The existing floorbeams span from Truss B to Truss C – the entire width of the Lower Roadway. Removal of a single floorbeam from this stringer-floorbeam framing system will require either removal or temporary support of the stringers from the adjacent bays. This leads to the following construction staging alternates.

4.1 *Alternate 1 – reconstruction of the Lower Roadway with a complete closure*

Under this alternate the entire Lower Roadway would be closed to vehicular traffic, Figure 3. All of the Manhattan-bound traffic would use the North Upper Roadway and Brooklyn-bound traffic would use the South Upper Roadway. The Williamsburg and Brooklyn Bridges would be used as alternate routes to "siphon off" traffic thereby creating less congestion on the upper roadways of the Manhattan Bridge.

If this alternate were selected, the contractor would be free to remove spans of the bridge without having to work nighttime shifts, which are usually less productive and more costly than daytime shifts. The construction materials could be delivered to the work zone using either the Brooklyn or Manhattan Approach and construction equipment could be placed (and stored) on the Lower Roadway for the erection of the framing. The floorbeams and stringers could be erected from above which means that, for the most part, the lower lateral bracing system could remain intact during reconstruction. The grid deck could be installed with the main bars running transversely without the need for splicing the deck in the strong direction (fewer construction joints). Rather than having precast panels with numerous construction joints that may lead to future maintenance

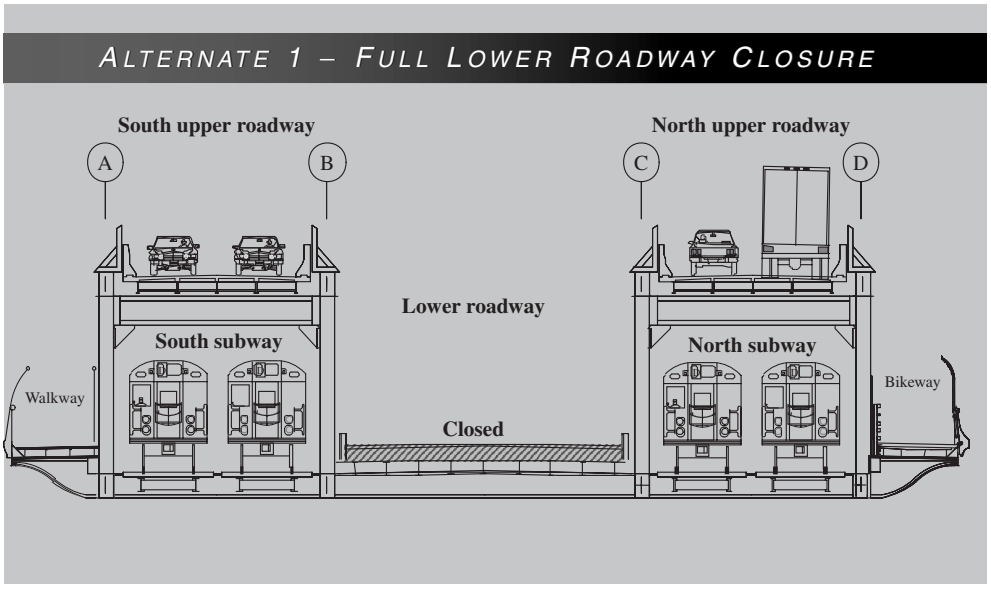

Figure 3. Alternate 1 – full closure option.

issues, the unfilled grid deck could be erected and then the concrete fill could be poured in the field (which would result in a seamless deck). It would also be possible to eliminate every other transverse expansion joint by making the stringers continuous over two spans.

The duration of construction would also be greatly impacted by this staging scheme. The following issues would minimize the duration of construction:

- The contractor would have complete and uninterrupted access to and from the work zone. The Lower Roadway could be used as a construction staging area.
- The contractor, if given an incentive, could work two productive shifts thereby significantly decreasing the construction duration.
- Maintenance and Protection of Traffic would be minimized. Once the Lower Roadway closure is in effect there would be no need to change the traffic pattern. The contractor would not be required to work at off-peak or nighttime hours. They would not be hampered by the need to open the roadway to traffic after a short nighttime construction closure.

The major disadvantage to this scheme would be the necessity to provide a complete closure of the Lower Roadway. When the Lower Roadway is closed for construction, the remaining four travel lanes may have insufficient capacity to accommodate the existing traffic volume, and delays would occur. Traffic would divert to alternate crossings to avoid the delay.

4.2 *Alternate 2 – Reconstruction of the Lower Roadway while maintaining a minimum of two lanes of traffic on the Lower Roadway during peak travel periods*

Under this alternate, Figure 4, it is assumed that at least two lanes of the Lower Roadway would be opened to traffic from 5 AM to 9 PM (peak travel periods are 5 AM to 10 AM Manhattan-bound and 3 PM to 9 PM Brooklyn-bound). It was also assumed that the ramp would remain in place during the non-peak period between 10 AM and 3 PM since it was not practical to move the ramp for such a short work shift. At all other times (9 PM to 5 AM nightly and on weekends) it is

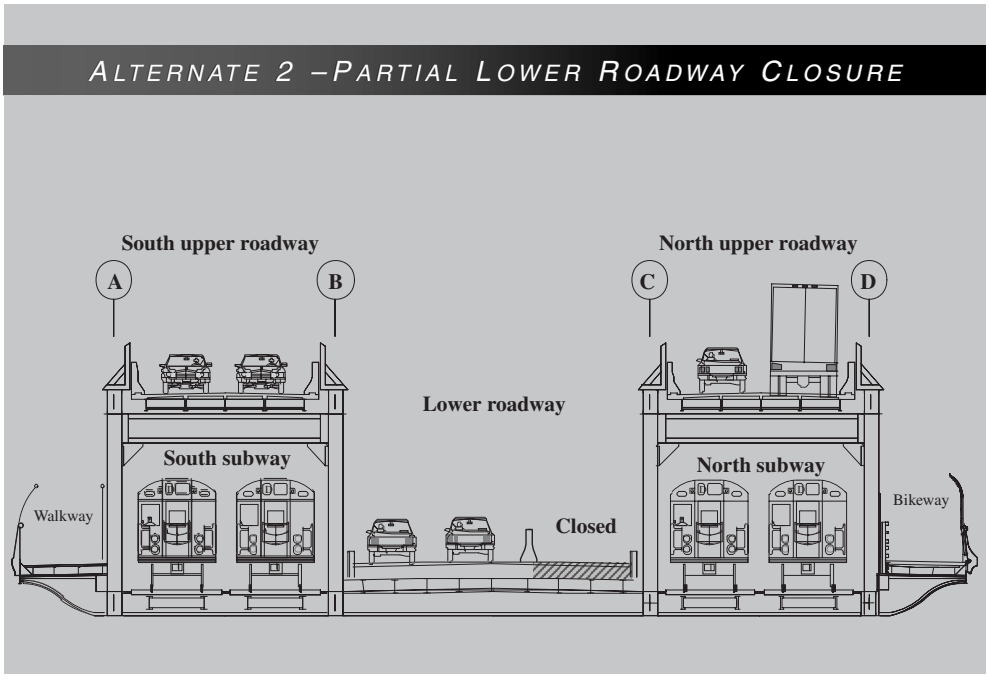

Figure 4. Alternate 2 – partial closure option; maintain two lanes.

assumed that the entire Lower Roadway would be closed and that all traffic would be detoured to the North and South Upper Roadways. The main benefit of this scheme would be the provision of two additional lanes of traffic during peak periods. There would be no need to detour traffic away from the bridge since four lanes of traffic could be maintained in the peak direction.

After reviewing various alternates to stage construction in this manner, it was decided that the only practical way to maintain traffic and perform the necessary reconstruction would be to use a temporary ramp system. The use of a temporary ramp would enable the contractor to perform work at night without the worry of completing an entire section of roadway prior to the morning rush hour. The temporary ramp would be erected during a weekend closure of the Lower Roadway. The reconstruction work would be undertaken during the following periods:

- Weeknights (9 PM to 5 AM) when the Lower Roadway would be closed to traffic. Replacement of the floorbeams could only occur under complete closure periods.
- The stringers and deck could be replaced at any time, however this work is best accomplished during nighttime and weekend closures when the temporary ramp will not hinder construction operations. During the daytime off-peak hours, 10 AM to 3 PM, work could be performed from the closed lane and two lanes of traffic could be maintained on the temporary ramp. Although limited, some construction operations could be accomplished from below the bridge during all daytime hours.
- Movement of the temporary ramp system would occur at nights and on weekends for construction operations. It is anticipated that the major construction operations would occur during these complete closure periods.

The ramp should be approximately 235 feet long consisting of 90-foot ramped section at each approach and a 55-foot long level section over the work zone (the length of the middle section of the ramp could be adjusted to meet the contractor's needs). It is assumed that the ramp would be manufactured as a movable system which can be rolled (moved) out of the way during construction operations. All of the steel removal and replacement work would be accomplished from above by construction equipment placed on the Lower Roadway. This way, there would be no need to disengage the lower laterals in the suspended spans.

As stated earlier, the replacement of the floorbeams would be accomplished when the entire roadway is closed to traffic. Although difficult, the stringers and deck under the ramp could be replaced during daytime hours (off peak) using the closed lane as a staging area. The existing stringers and deck panels could be shifted over to the closed lane, picked up by small cranes positioned in the same lane, and removed from the site. In a similar fashion, the new stringers and grid deck could be installed in the closed lane and shifted under the temporary ramp.

The main benefits of this scheme are as follows:

- A total of six lanes, four on the upper roadways and two on the Lower Roadway, would be available weekdays from 5 AM to 9 PM.
- Complete closure of the Lower Roadway during peak travel periods would not be required. There would be no need to detour traffic away from the bridge. Experience during other construction contracts had shown that six traffic lanes available during peak periods would be sufficient to handle the traffic volume.

The major disadvantages of this scheme are as follows:

- The contractor would be forced to work at night which is generally not as productive as daytime construction.
- The contractor must move the ramp each night and on weekends prior to the commencement of construction operations during the complete closure periods.
- Maintenance and Protection of Traffic will be much more difficult and involved under this alternate. The contractor will be required to open and close roadways and shift traffic on a continual basis.
- Since the temporary ramp is to be shifted on a weekly or biweekly basis, it is probable that the new deck will be precast.

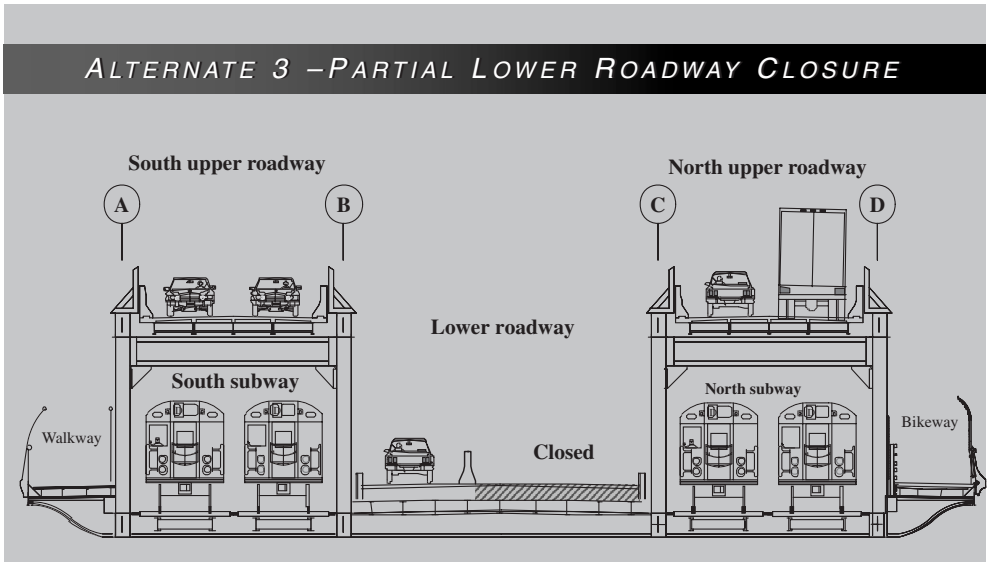

Figure 5. Alternate 3 – partial closure option; maintain one lane.

4.3 *Alternate 3: – Reconstruction of the Lower Roadway while maintaining a minimum of one lane of traffic on the Lower Roadway during peak travel periods*

Under this alternate, Figure 5, it is assumed that at least one lane of the Lower Roadway would be opened to traffic from 5 AM to 9 PM (peak travel periods are 5 AM to 10 AM Manhattan-bound and 3 PM to 9 PM Brooklyn-bound). It was also assumed that the ramp would remain in place during the non-peak period between 10 AM and 3 PM since it was not practical to move the ramp for such a short work shift. At all other times (9 PM to 5 AM nightly and on weekends) it is assumed that the entire Lower Roadway would be closed and that all traffic would be detoured to the North and South Upper Roadways. The main benefit of this scheme is the provision of one additional lane of traffic during peak periods. The three lanes available for peak hour traffic should be able to handle the majority of the traffic volume, however a provision for alternate routes should be provided.

As with Alternate 2, the only practical way to implement this scheme would be to provide a temporary ramp on the Lower Roadway, but in this case it would be only one lane, Figure 6. The use of the ramp would enable the contractor to perform work at night without the worry of completing an entire section of roadway prior to the morning rush hour. The temporary ramp would be erected during a weekend closure of the Lower Roadway. The reconstruction work would be undertaken during the following periods:

- Weeknights (9 PM to 5 AM) when the Lower Roadway would be closed to traffic. Replacement of the floorbeams would occur under complete closure periods.
- The stringers and deck could be replaced at any time, however this work is best accomplished during nighttime and weekend closures when the temporary ramp would not hinder construction operations. During the daytime off-peak hours, 10 AM to 3 PM, work could be performed from the closed lanes and one lanes of traffic could be maintained on the temporary ramp. Although limited, some construction operations could be accomplished from below the bridge during all daytime hours.
- Movement of the temporary ramp system would occur at nights and on weekends. It is anticipated that the major construction operations would occur during these complete closure periods.

As stated earlier, the replacement of the floorbeams could only be accomplished when the entire roadway is closed to traffic. Although difficult, the stringers and deck under the ramp could be

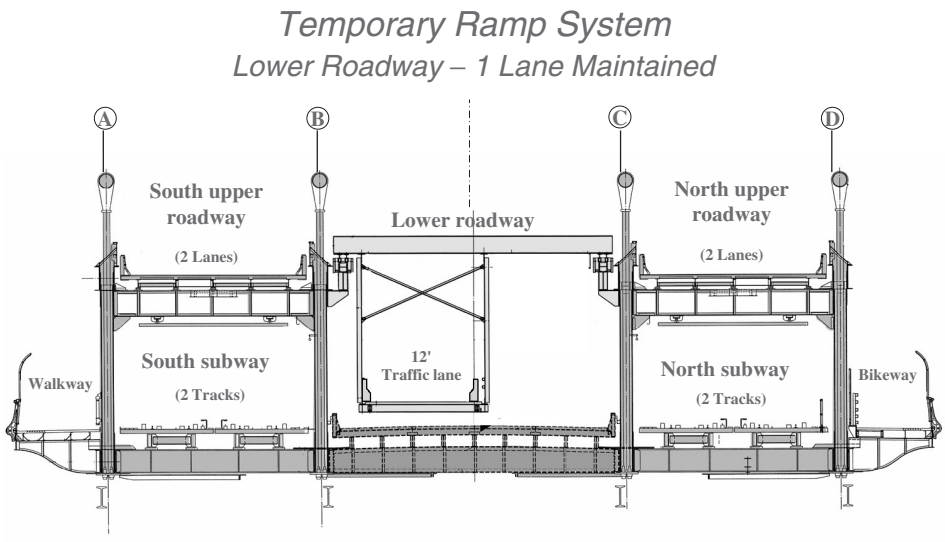

Figure 6. Temporary movable ramp system.

replaced during daytime hours (off peak) using the closed lane as a staging area. The existing material (stringers and deck panels) could be shifted over to the closed lane, picked up by small cranes positioned in the same lane, and removed from the site. In a similar fashion, the new stringers and grid deck could be installed in the closed lane and shifted under the temporary ramp.

The main benefits of this scheme are as follows:

- A total of five lanes, four on the upper roadways and one on the Lower Roadway will be available weekdays from 5 AM to 9 PM. This should be sufficient to handle most of the traffic volume without causing major delays. As with Alternate 1, alternates routes could be established to provide capacity relief if needed.
- The contractor would have two lanes of the Lower Roadway available for his use at all times. This additional space would allow him greater flexibility to perform the necessary construction operations. Since two lanes are available at all times, approximately 2/3 of the deck and stringers could be replaced without impacting traffic or requiring the closure of the Lower Roadway. During the nighttime and weekend closures, the temporary ramp would be moved to provide access for the floorbeam replacement operations. With this flexibility the reconstruction would be accomplished more quickly than under Alternate 2.
- The temporary ramp required would accommodate one lane of traffic and therefore would be half the size of the ramp required for Alternate 2. The movement of the ramp would be much easier and the ability to move sections of the ramp out of the way at night would be greatly enhanced. The production rate for floorbeam replacement would be greatly improved over that of Alternate 2.

The major disadvantages of this scheme are as follows:

- The contractor would be forced to perform some of the work at night which generally would not be as productive as daytime construction.
- The contractor must move the temporary ramp each night and on weekends prior to the commencement of the construction operations during the complete closure periods.
- Maintenance and Protection of Traffic would be much more difficult and involved under this alternate. The contractor would be required to open and close roadways and shift traffic on a continual basis.

5 RECOMMENDATIONS FROM PRELIMINARY EVALUATION

After the preliminary evaluation was completed, it was decided to progress at least two of the alternates to the final evaluation process. Each of the three alternates had distinct advantages:

- The full closure option would provide a superior final product with limited future maintenance concerns, would have a much shorter construction duration, and would be less costly.
- The partial closure options, Alternates 2 and 3 would have less impact on traffic during construction since a minimum of three lanes would be provided for peak hour traffic, as opposed to two lanes under the full closure option.
- Alternate 2 has the additional advantage of providing four lanes of traffic in the peak direction, and would easily accommodate the peak hour traffic volumes.

A decision was made to progress Alternates 1 and 3, the full closure and one lane options to the final evaluation process. Since traffic impacts would be a critical factor in the decision making process, the NYCDOT decided that a comprehensive traffic study should be undertaken to evaluate impacts that these two staging alternates would have on the surrounding area, including the Brooklyn and Williamsburg Bridges.

6 TRAFFIC STUDY

In order to obtain the traffic data and analysis needed to support the decision of which alternate to recommend, Weidlinger retained the services of SIMCO Engineering, PC, to perform the required traffic study (Sim 2002). The scope of work for the traffic study included the following:

- Automated Traffic Recorder (ATR) counts on the Manhattan, Brooklyn and Williamsburg Bridges, for a period of two consecutive weeks.
- ATR counts at 29 critical intersections on the streets systems in Manhattan and Brooklyn adjacent to these three East River Bridges, for a period of one week.
- Manual turning movement counts at 37 intersections, 10 of which included detailed vehicle classification counts. In addition, travel time/delay runs along six routes and lane usage counts at the three bridges and the two neighboring tunnels (Brooklyn Battery and Queens Midtown) were obtained.

Baseline volumes obtained during the study year (2001) were projected to the construction analysis year (2005) by growth rates provided by the New York Metropolitan Transportation Council (NYMTC). In addition, traffic impacts caused by other planned infrastructure projects in the immediate area were included in the analysis. This resulted in the 2005 No-Build traffic condition, of which hourly capacities and levels of service (LOS) were determined. In summary, the three analyses performed were as follows:

- The No-Build Condition – performed for purposes of comparison to the two proposed construction alternates.
- Alternate 1 – complete closure of the Lower Roadway, thus diverting all existing lower level vehicles to the excess reserve capacity of the Upper Roadways (during peak hours) and the neighboring East River Bridges.
- Alternate 3 – partial closure of the Lower Roadway (two lane closure); the lane which remained open on the lower level would be reversible based on peak directional flows. This reversible lane would provide additional capacity to the Upper Roadway (in the peak direction) thus providing three peak hour lanes.

With this 2005 No-Build volume and analysis in place, SIMCO investigated the potential traffic diversion volumes related to the two alternates into the neighboring roadway network, taking into account potential congestion, delay and capacities. For Alternate 3 (partial closure), no traffic diversion was assumed based on the sufficient capacity of the Manhattan Bridge Upper Roadway

Table 1. 2005 No-Build queue lengths.

Facility	Location	Direction	Period	Queue length
Manhattan Br.	Chrystie/Canal Sts.	Manhattan-bound	AM Peak	0.49 miles
Brooklyn Br.	Cadman Plaza West	Brooklyn-bound	PM Peak	0.71 miles

with the single Lower Roadway lane. For Alternate 1 (complete closure), the Upper Roadway has been assigned the additional volume up to its capacity, with a percentage of the excess volume diverting to alternate routes. Based on professional experience and past practices at other major bridge reconstruction projects, up to 10% of the Manhattan Bridge vehicles can be expected to divert. The nearer facility (Brooklyn Bridge – passenger vehicles only) was assigned the diverting passenger vehicles, while the further facility (Williamsburg Bridge – passenger vehicles and trucks) was assigned the diverting commercial vehicles. The combination of the 2005 No-Build volumes and this anticipated diversion traffic volumes resulted in projected 2005 Build volumes expected in the area based on the alternates under review. SIMCO was then able to determine 2005 Build capacity analyses for the two proposed alternates.

One of the most critical traffic performance measures for the alternates being considered is the extent and magnitude of potential vehicle queues and delays incurred by the motorists during the various lane closure scenarios on the Manhattan Bridge, as well as the Brooklyn and Williamsburg Bridges. To establish a future baseline against which to measure the potential impacts of the proposed MPT alternates, the 2005 No-Build traffic volumes with normal two open lanes on the Lower Roadway were used in the queue and delay analysis. Two critical locations in the study area were selected for a detailed queuing analysis, one Manhattan bound and one Brooklyn bound. The locations were:

- Inbound Manhattan Bridge Upper Roadway approach to the Chrystie Street/Canal Street intersection in Manhattan.
- Outbound Brooklyn Bridge exit ramp approach to the unsignalized Cadman Plaza West right turn movement at Middagh Street.

In summary, the following are the expected queue lengths for the two locations during the 2005 No-Build condition.

A detailed queuing analysis for the 2005 Build scenario was performed to determine the extent of congestion queues expected during the Manhattan Bridge construction period. Alternate 3 (partial closure) provides one less lane in the peak direction than the No-Build condition, however, the No-Build scenario has sufficient capacity to process the peak hour volume in one less lane, thus the queue lengths are virtually the same for Alternate 3 as the No-Build (shown in Table 1). Alternate 1, with only the Upper Roadway available for processing vehicles, produces longer queues, summarized as follows.

In an attempt to reduce the queue expected on the Manhattan Bridge North Upper Roadway during the AM Peak, various mitigation measures were investigated at the terminus in Manhattan (Chrystie Street). Northbound vehicles on Chrystie Street between Canal and Hester Streets are vehicles exiting from the Manhattan Bridge North Upper Roadway; southbound vehicles are only destined for Canal Street, where access is also available at Hester and Grand Streets. It was anticipated that if traffic directions were altered during construction, queue lengths would be reduced. In addition, it was recommended to eliminate parking along the Bowery, along with the placing of traffic agents at 10 intersections on Bowery and Chrystie Street. Detailed analysis showed that by closing Chrystie Street between Canal and Hester Streets from 6 AM to 9 AM, (thus eliminating the traffic signals during these hours and allowing free flow, and incorporating these other measures, queue length on the North Upper Roadway in the AM peak would be reduced by 80% (from 0.49 to 0.1 miles) for Alternate 3, and reduced by 72% (from 1.09 to 0.3 miles) for Alternate 1.

In addition to the noted improvement in queue lengths that can be anticipated with these mitigation measures on the Manhattan side, capacity analyses for critical intersections (which worsened with

Table 2. Alternate 1 – full closure of lower level – queue lengths.

Facility	Location	Direction	Period	Queue length
Manhattan Br.	Chrystie/Canal Sts.	Manhattan-bound	AM Peak	1.09 miles
Brooklyn Br.	Cadman Plaza West	Brooklyn-bound	PM Peak	0.76 miles

the two alternates) improved with the introduction of these mitigating measures. For example, at the intersection of Canal and Chrystie Streets, the AM Peak period value was calculated as LOS D, which worsens to an unacceptable LOS F for both alternates. With the mitigation measures proposed on Chrystie Street and the Bowery included in the analysis, the results improve conditions during this AM Peak period back to the LOS D.

7 FINAL REVIEW

Once the traffic study was completed, the two viable alternates were again compared. Issues such as quality of final product, sensitivity to inclement weather, difficulty of construction operations, initial construction cost, maintenance of traffic, community impacts, and safety were all considered during the final selection process. The following is a brief summary of the evaluation process.

7.1 *Duration of construction*

In the partial closure option, the duration of the Lower Roadway construction was greatly increased due to the restrictions imposed on the contractor. The need to maintain traffic on the Lower Roadway would limit the work which can be accomplished during daytime hours. The majority of the construction operations would need to be completed during nighttime and weekend closures of the Lower Roadway. The total estimated construction duration for the partial closure option was 26 months.

The full closure option allowed for much more flexibility during construction. The contractor would have full use of the Lower Roadway for the entire construction duration, and therefore would not have to worry about delivery of materials, equipment placement, concrete pours, installation of prefabricated elements, working nighttime shifts (which can lead to safety concerns and unproductive shift work) and opening the bridge to traffic for the morning rush hour. The estimated duration for the full closure option was 12 months.

7.2 *Construction costs*

The full closure option has a major advantage when evaluating construction costs. The restrictions caused by the partial closure option would significantly increase the construction costs. The increased costs of working at night would have a twofold disadvantage; nighttime work tends to be less productive thereby increasing the duration of construction and increasing unit costs and construction workers must be paid a nighttime differential. In addition, the extended construction duration would increase the contractor's overhead costs. The contractor must purchase and erect a costly temporary ramp system. It was estimated that the complete closure option would result in a $30 million cost savings to New York City.

7.3 *Quality of final product*

The full closure option would allow the contractor to place the grid deck panels the full width of the roadway with no need for the installation of transverse splices in the main load carrying elements of the grid deck, which can be problematic. The concrete can be poured over the entire width of the roadway from deck joint to deck joint with no transverse cold joint required. This is in contrast to the partial closure option where a precast deck system would be used. The connection of these precast panels to the steel framing requires numerous cold joints in the concrete deck surface,

which could eventually leak and lead to deterioration and corrosion, thereby decreasing the service life of the deck and steel framing.

7.4 *Construction operations*

Again, the advantage lies with the full closure option. The full closure option has many distinct advantages including the most important – the contractor has unlimited access to the Lower Roadway for the entire duration of Construction. The disadvantages of the partial closure option would be:

- The temporary ramps must be moved on a daily basis in order for the contractor to perform the required construction activities. Breakdown of the movable ramp system may create delays in opening the Lower Roadway to traffic the following morning.
- Either two 3-month winter shut downs or winter concreting would be required during the duration of the contract if the partial closure option was selected. Winter shutdowns and/or winter concreting are not required under the complete closure scheme.
- Due to the restrictions placed on the contractor under the partial closure scheme, major construction operations will be undertaken on a 24 hour a day basis on the weekends.

7.5 *Safety*

Safety of the traveling public and the construction workers is a major concern of the NYCDOT. The use of a temporary ramp system raised some safety concerns that impacted negatively on its evaluation. To achieve a 26-month schedule, some construction activities would have to be accomplished during daytime hours when the temporary ramp would be in service. There was a concern that the use of construction equipment adjacent to the temporary ramp would be more likely to lead to construction-related accidents, and if they resulted in a failure of the ramp, could potentially cause serious injury to the motoring public on the ramp. Inclement weather could cause motorists to lose control of their vehicles, specifically since the vertical grades required to make the ramp system work are greater than 9% at some locations.

Due to clearance and structural limitations, the moveable ramp system cannot be designed to accommodate trucks. However, with the possibility that trucks could find their way onto the ramp, these vehicles could damage both the existing structure and the ramp system.

The partial closure option creates situations where both the City and the contractor would have greater risk during construction.

7.6 MAINTENANCE AND PROTECTION OF TRAFFIC

The partial closure option has a distinct advantage from a maintenance of traffic point of view. The traffic study shows that the partial closure option can accommodate traffic using the bridge without significant delays. This is backed up by actual experience on the bridge in that this traffic configuration has been used successfully in the past. The disadvantages to the partial closure option are:

- The temporary ramp lane is a cattle chute which could lead to delays if a vehicle breaks down.
- The potential for delays caused by a failure of the driving mechanism of the moveable ramp.

The disadvantages of the full closure option are straightforward. Traffic diversions to other East River crossings are required to accommodate the volume. Mitigation measures are necessary if this option were selected.

8 SELECTED OPTION

After a thorough review of all the issues noted above, the decision was made to select the full closure option. Quality of the final product, safety concerns, a much shorter construction schedule,

and a $30 million cost savings were key elements in the selection process. To mitigate the negative impacts caused by the closure, the NYCDOT provided the following mitigating measures:

- Creation of an extra lane on northbound Chrystie Street and the Bowery for motorists entering Manhattan during the AM Peak Period.
- Providing on-site 24-hour tow truck service to ensure quick removal of disabled vehicles.
- Stationing NYPD traffic agents on both the Bowery and Chrystie Street from Canal Street to Delancey Street to help with the traffic operations.
- Providing an incentive/disincentive clause in the contract that will hopefully cut the duration of closure from 12 to 10 months.

9 CONCLUSIONS

As an owner, the NYCDOT has to weigh many factors in planning its rehabilitation projects. Each project has to be reviewed with an eye towards the total rehabilitation program and how that project will impact adjacent transportation facilities. The many factors considered in the planning of this project – maintenance of traffic/staged construction, quality of final product, contract duration, initial construction and future maintenance costs – are all pieces of the puzzle that have to be placed in the proper position to result in a successful project. Because of the congestion in the area of the Bridge, NYCDOT decided that the recommended mitigative measures are worth the cost and should be implemented. These measures include the liberal use of traffic agents at the bridge approaches, the use of an incentive/disincentive clause to speed the pace of construction, and the reconfiguration of the parking lanes and traffic flow from the North Upper Roadway onto Chrystie Street and the Bowery in Manhattan. Once these measures are implemented, both construction options show traffic improvements over the "no-build" condition, with the partial closure option producing slightly better results than the full closure option. However, the period of disruption for the full closure is significantly shorter (12 months vs. 26 months) than the partial closure. By significantly decreasing the duration of the project, a substantially decrease in the construction costs will be realized. In addition, a full closure also provides for a safer construction operation and gives an end product of higher quality, which ultimately reduces frequency and magnitude of repair work. In NYCDOT's judgment, the negative impacts of full closure are overshadowed by the benefits gained, and as a result, this option was selected.

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Chapter 15

Forth Road Bridge – first internal inspection, strength evaluation, acoustic monitoring and dehumidification of the main cables

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ABSTRACT: The Forth Road Bridge was opened to traffic in 1964, and at the time of construction was the fourth longest span in the world. It now carries over 65,000 vehicles per day and is a key link in the Scottish transport network.

Following work carried out as part of the development of the NCHRP Guidelines, the Forth Estuary Transport Authority (FETA), the owners and operators of the crossing, took the decision in 2003 to carry out a first internal inspection of both main cables. The inspection revealed that there was significant corrosion within the main cables.

This paper describes that inspection, results and the evaluation of the current and predicted future strength of the cable. Following the inspection, a decision was taken to install an acoustic monitoring system and implement cable dehumidification and the current status of this work is also described.

1 INTRODUCTION

When opened in 1964, the Forth Road Bridge was the longest span suspension bridge outside the USA and the fourth longest in the world. It was the most northerly bridge with an aerially spun cable. The main span is 1006 m with equal side spans of 408 m. The cable comprises 11618 galvanized steel wires 4.98 mm in diameter and was installed with the assistance of the John A Roebling's Sons Company to the UK bridge contractors. It was compacted and coated in red lead paste, wrapped and coated with conventional paints.

The bridge has been heavily used since it was opened and now carries 24 million vehicles per year (65,000 per day) and forms a vital link in the Scottish road network. The nearest highway crossing over the Forth is some 32 km inland, and the Forth Road Bridge sits adjacent to its famous neighbour, the magnificent 521 m span cantilever truss Forth Rail Bridge (Figure 1).

The design of the suspension bridge clearly follows the practice then adopted by American designers and uses a stiffening truss rather than a box girder as subsequently used on the Severn and Humber bridges. The bridge carries two lanes in each direction, with wide segregated footway/cycletracks mounted on cantilevers outside of the cables.

The bridge has a dedicated maintenance unit, which carries out routine inspection and maintenance activities. Major tasks are usually undertaken by external contractors. Further information is given in Andrew & Colford (2006).

Over the life of the bridge regular maintenance of the main cable has been undertaken, which has included periodic repainting. Regular external inspections have been carried out with no significant deterioration or egress of water noted.

The investigative work carried out in the USA into the condition of main cables during the last 15/20 years has been closely watched, and FETA staff have joined in and contributed to various

Figure 1. The Forth Bridges.

conferences and workshops. Whilst there was some comfort in the fact that the Forth Road Bridge was somewhat younger than many of the American bridges investigated, FETA took a conscious decision to follow the recommendations in the NCHRP Guidelines (NCHRP, 2004) and to carry out a first internal inspection of the main cables.

The role of consultant was tendered on a quality and price basis, with Faber Maunsell of London being appointed in May 2003 in association with Weidlinger Associates Inc. of New York, who were the authors of the NCHRP Guidelines.

2 SCOPING STUDY

The first task for the consultants was to undertake a scoping study. This involved the collection and review of all available records from construction and subsequent maintenance, plus a detailed inspection of the current external condition of the cables.

Whilst a number of records from construction had been retained, it is unfortunate that many potentially useful documents and drawings had not survived. However, a set of very comprehensive papers describing the design and construction had been published by the Institution of Civil Engineers (Anderson et al., 1965), and this gave some very useful data. Also a number of detailed Resident Engineer's reports dating from the time of construction were found – these included spinning and wrapping, but not compaction (Mott, Hay & Anderson and Freeman Fox & Partners, 1962 and 1964).

During the course of the investigation a number of staff employed on the original construction came forward with some interesting information. Amongst these was a suggestion that the amount of red lead paste used was less than theoretically required to fill the gaps between the wires and that wrapping continued in poor weather conditions (ice and rain).

One particular incident during construction that came to light was a severe storm early on during spinning when only a few strands had been spun. Extensive tangling occurred in one of the side spans, which took a long time to comb out and reform the strands. This event reinforces the exposed northerly location of the bridge.

The cable inspection walks showed that the paintwork was overall in a good condition, although there were some small and widespread defects and that some of these were exacerbated by defects either in the wrapping wire or crossing main cable wires below (Figs. 2–3). This was frequently noted at the end of a panel where the wrapping process had held an exterior wire in a position along the panel, but as it approached the far end of the panel it had to cross over to where it passed through the band. Bulges in the wrapping occurred in these locations and were suspected as potentially containing water. However, there was no evidence of water coming out of the cable at any of the cable bands.

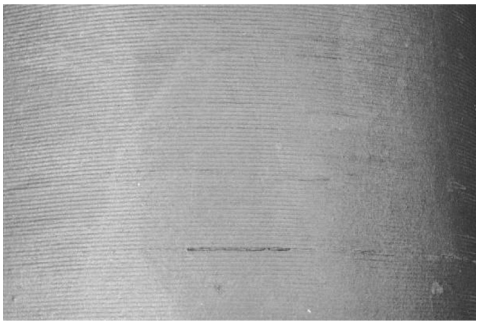

Figure 2. Cracks in paint.

Figure 3. Bulge in wrapping.

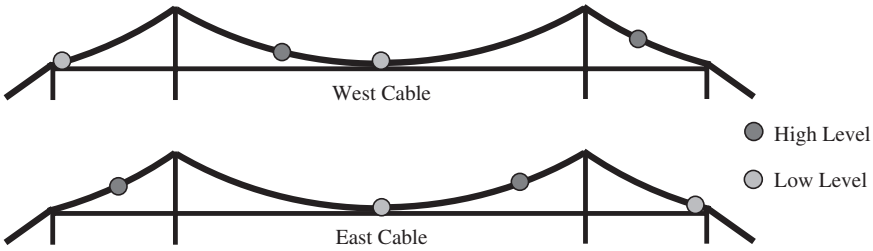

Figure 4. Planned cable inspection locations.

Although the NCHRP Guidelines were being followed, the recommended number of inspection locations was increased from 6 to 8, partly as the bridge was 10 years older than the minimum age of 30 years recommended for carrying out the first inspection, and partly to give a greater representative geographic coverage of different areas of the bridge. Four low level locations were recommended (two at mid main span, two diagonally opposite at ends of side spans) and four high level (two main span, two side span), as shown in Figure 4.

3 CONTRACTOR PROCUREMENT

A contractor was appointed to carry out the work associated with facilitating the inspection, and like the consultant the selection was based on quality and price. As this was the first such internal cable inspection in Europe there were no U.K. or European contractors with direct experience and so the tender list was extended to include one U.S. contractor. The successful contractor was C Spencer Ltd.

4 INSPECTION PROGRAMME

Due to high winds experienced during winter, the contract was planned to be carried out over the summer period during 2004.

The inspections followed typical procedures adopted on U.S. bridges, however, the very strict UK health and safety legislative environment along with working within strict traffic management rules and the new nature of work with cables, all presented challenges to the contractor.

The low level access platforms were conceived as simple scaffolding, with the only difficulty being their location above the aerodynamic ventilation gap between carriageway deck and footway/cycletrack deck. Site work commenced with the low level platforms and these allowed the contractor to learn the required techniques.

The high level platforms were initially planned to be made from specialist aluminium scaffolding components and a trial platform was built on the ground below the back stays. However, this did not prove satisfactory and contractual difficulties with the access supplier led to significant delay. It soon became apparent that the high level panels could not be accessed within the summer period and it was agreed that these would be postponed until the following spring, by which time purpose designed steel platforms had been fabricated.

As will be seen in the next section, initial results from the inspection were worse than anticipated and so it was decided to carry out further low level inspections over the winter period where the effects of high winds were less severe.

At each panel, generally eight groove lines were sequentially driven and one sample wire removed and replaced. The cable was recompacted and wrapped using specially built machines. Red lead paste was used to coat the cable following traditional practice.

5 INSPECTION RESULTS

The first panel to be inspected was on the east cable at mid main span. The general expectation was that the cable would be in a good condition with the wires being mostly Stage 2 with perhaps a few Stage 3. There was considerable surprise when broken wires were found at the bottom of the cable and examination of the cable surface showed many Stage 4 wires were present. At the top of the cable Stage 4 wires were found just below the surface (Figure 6).

The condition of the wires in the cross section is shown in 5. It can be seen that there is a significant population of Stage 4 wires with many located towards the top of the cable, but more importantly, most of the remaining wires have already reached Stage 3, leaving only a few Stage 1 and 2 wires. Hence significant deterioration has already taken place with the inevitable future transition of the Stage 3 wires to Stage 4.

Also of interest was the finding of a piece of wood deep inside the cable (Figure 7). This was believed to have been part of the strand adjustment equipment that became "lost" within the cable. It had caused the fracture of one wire and a local area of Stage 4 wires.

The second panel inspected was on the same cable, but at the end of the northern side span. Given the poor condition of the first panel, it was therefore expected that similar conditions may exist here too. However, the findings were totally contrary with the vast majority of wires being in good condition at Stage 2 (Figure 8). No broken wires were found.

The third panel was located diagonally opposite at the end of the south side span, west cable. The amount of wire deterioration was somewhat greater than the previous, but the particular feature

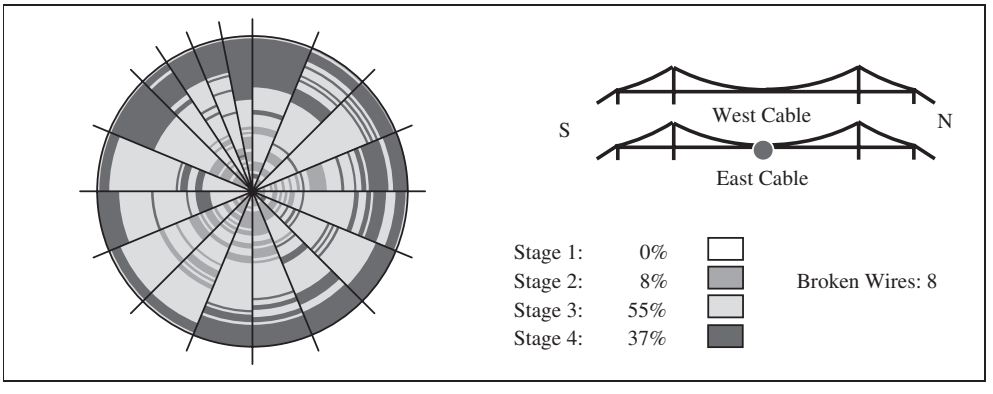

Figure 5. Inspection result for first panel (E100S – 100N).

here was the large number of broken wires, 31 in total (Figure 9). About one third were found to have broken within the adjacent side tower saddle and could not be repaired.

The fourth panel at mid main span on the west cable completed the initial low level inspections. Conditions similar to the first panel were found, but not so severe (Figure 10).

The planned low level inspections were now complete, with the high level ones anticipated the following spring. This pause enabled some thought to be given to the results obtained so far and to try to account for the radically different results.

Figure 6. Stage 4 wires below surface.

Figure 7. Piece of timber inside cable.

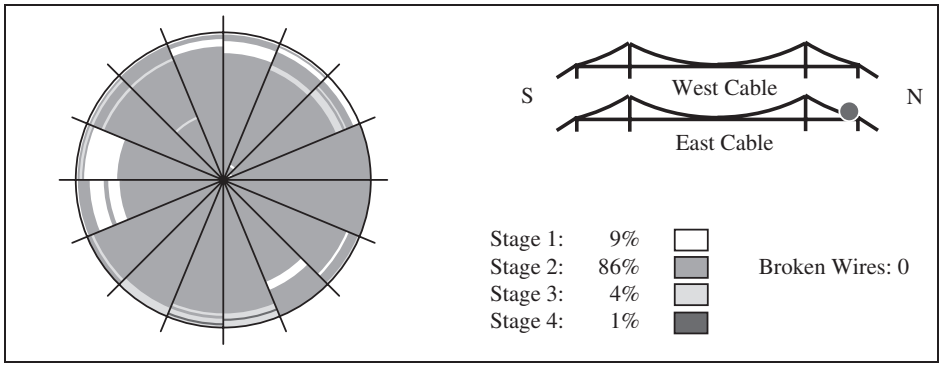

Figure 8. Inspection result for second panel (E00N – 02N).

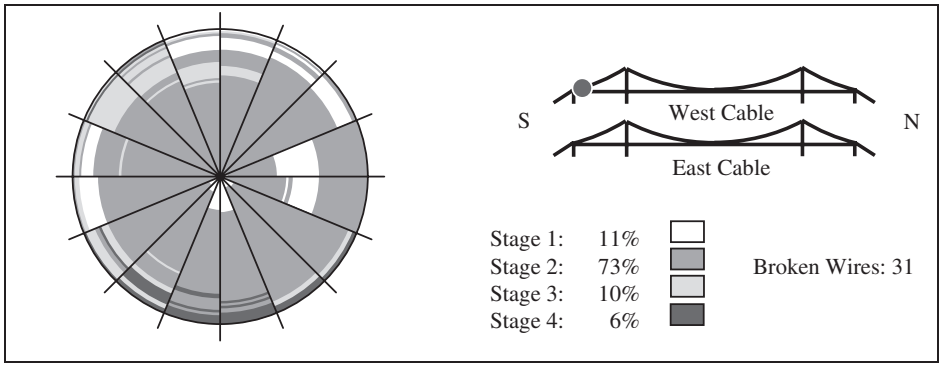

Figure 9. Inspection result for third panel (W00S – 02S).

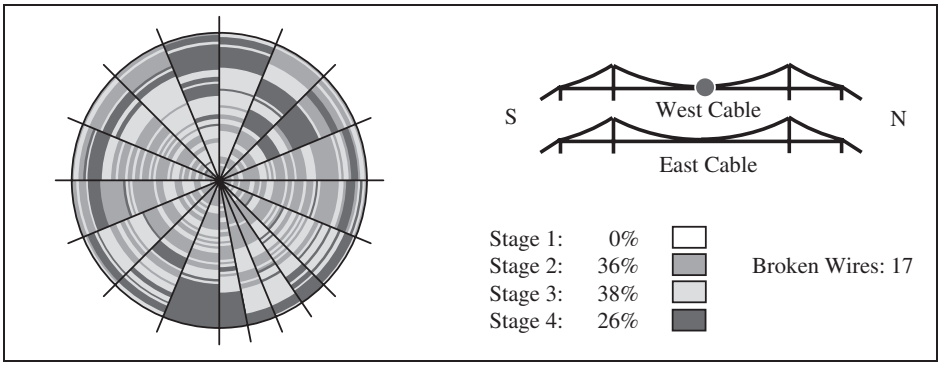

Figure 10. Inspection result for fourth panel (W100S – 100N).

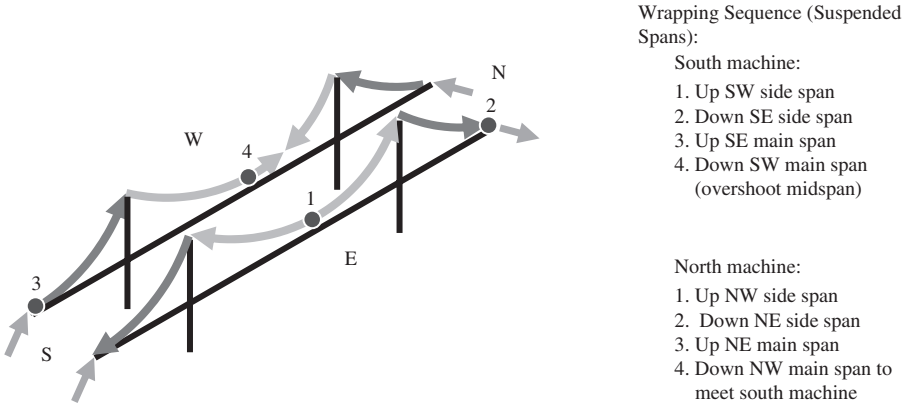

Figure 11. Wrapping sequence.

Factors such as cable slope, orientation and prevailing wind direction were considered, but these could not adequately explain the variation found. Another idea developed concerned the direction the cable was wrapped. It was thought that uphill wrapping would have tended to hold any water in the cable, whereas downhill wrapping would tend to squeeze the water out. The origin of this idea came from experience of wrapping the cables on a major bridge. Here they had been wrapped uphill, but with a few panels left unwrapped at the low points to permit water to drain out. Following heavy rains, water poured out at mid span and was still coming after a week. Obviously over the life of the bridge this initial water should have found its way out a long time ago, but it would have created an initially wet cable.

Construction records were re-examined and the following diagram determined.

The first panel inspected at the east mid main span was wrapped uphill in both directions away from it, and the second panel was wrapped downhill and was found to be in a much better condition. The third panel with the many broken wires was also wrapped uphill. The fourth panel was mostly wrapped downhill but there was some uphill wrapping as the wrapping machine passed through midspan and met the other machine further along the cable, thus there was some correlation with the wrapping direction theory, but it was not conclusive.

It was decided to make use of the winter period and carry out inspection of two further low level panels. One was at the bottom of the north-west side span (uphill wrapping). Conditions similar to the other uphill wrapped side span panel were found, but the number of broken wires was less. The other additional panel inspected was adjacent to the first panel (again uphill wrapping). Here the results were similar to the adjacent panel, but with a lower proportion of Stage 4 wires. To complete the

Table 1. Summary of inspection results.

			Corrosion level				
	Cable panel	High/Low	Stage 1	Stage 2	Stage 3	Stage 4	Broken
1.	E100S-100N	Low	0%	8%	55%	37%	8
2.	E00N-02N	Low	9%	86%	4%	1%	0
3.	W00S-02S	Low	11%	73%	10%	6%	31
4.	W100S-100N	Low	0%	36%	38%	26%	17
5.	W00N-02N	Low	11%	75%	6%	8%	8
6.	E100N-98N	Low	0%	16%	58%	26%	8
7.	W22N-24N	High	0%	45%	42%	13%	5
8.	E18S-20S	High	19%	60%	19%	2%	0
9.	W76S-74S	High	7%	58%	25%	10%	2
10.	E60N-58N	High	22%	43%	26%	9%	7

inspection of the six low level points, the cable sleeve was removed at the bottom of the south-east side span. Conditions were found to be good, as expected.

The following spring inspections of the four high level panels were carried out. Conditions ranged from average to good, with no particular findings of note. Table 1 above summarises the visual inspections on all ten panels.

6 TESTING

One 6 metre long sample wire was removed from each groove and was tested in a local laboratory. The majority of the testing was tensile, with each sample providing ten tensile specimens. Only four specimens from Stage 1 and 2 wires were tested, making a total number of tensile tests of 704.

Limited information was available on the specification of the cable wire, but it was known to have a tensile strength in the range 100–110 Imp. Tons/sq. in. (1544–1698 N/mm^2) based on gross cross-sectional area. It was manufactured at Dorman Long Wire Works on Teeside, U.K., who went on to supply the wire for Severn and Humber bridges. The 0.400 inch diameter rod had a specified analysis of 0.75 to 0.80% carbon and 0.50 to 0.70% manganese. Following drawing and galvanising it was collected on a 5 ft diameter coiler. This diameter was still apparent on the sample wires removed.

A summary of the results is shown below in 12, separated into the four corrosion stages plus Stage 4 wires that were found to be cracked.

It can be seen that a few wires exceeded the upper specified strength limit. It is known that the specification for Severn and Humber was changed to 100–115 Imp. Tons/sq. in. (1544–1774 N/mm^2) owing to experience at Forth, and it must be presumed that the overstrength wire was accepted at Forth.

More significantly, a large proportion (31%) of the test samples lie below the minimum specified tensile strength of 100 Imp. Tons/sq. in. No doubt the small loss of steel cross-section due to corrosion on some of the wires would account for falling below specification, however, as some of the Stage 2 wires lie in this area it seems likely that some marginally substandard wire was included in the cable.

It was interesting that no cracks were found in sample wires greater than 5 rings into the cable. This concurred with the visual inspections in the panels, where broken wires were always near the surface, except where there was a local anomaly such as a piece of buried timber.

An unusual feature of the testing was that the stress-strain relationship was measured up to failure in one specimen from each sample wire. This is not usually carried out as the extensometer is vulnerable to damage when the wire breaks, however, the laboratory was able to provide a suitably robust one. Typical full length stress-strain plots are shown below (Figure 13).

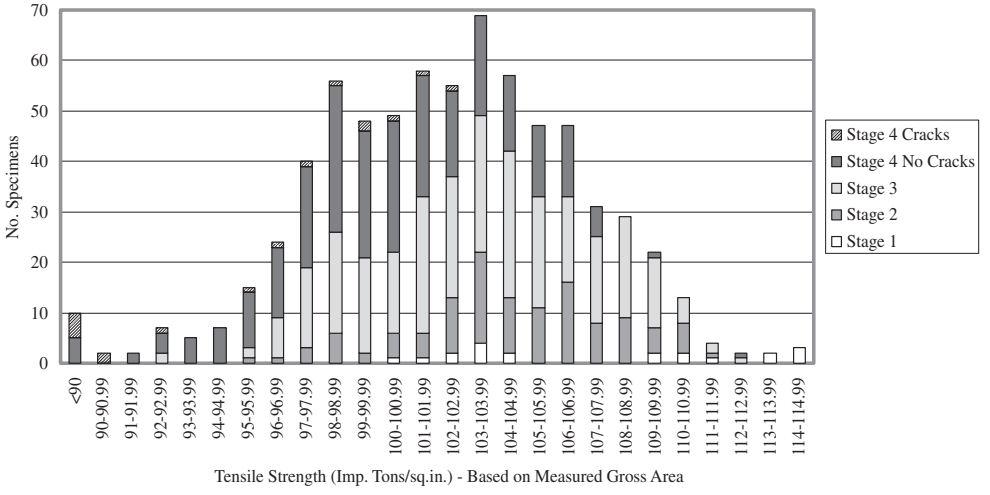

Figure 12. Summary of tensile tests.

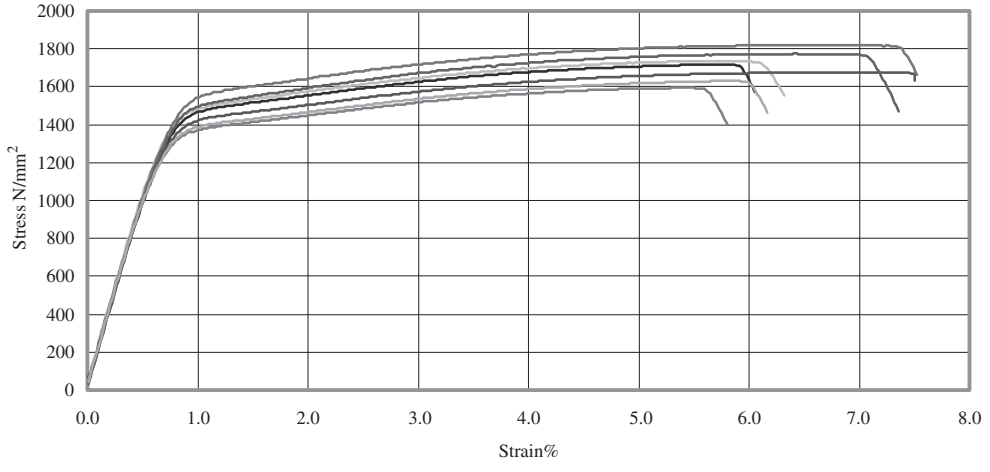

Figure 13. Variation in stress – strain curves.

It was apparent that there was a surprisingly large spread in these curves, which was thought to be due to varying carbon content. Further chemical tests were carried out and the following graph was produced, demonstrating that there was indeed a relationship between tensile strength and carbon content (Figure 14).

It will be noted that many of the wires are outside the specification of carbon content of 0.75% to 0.80%. However, it should be noted that these are not representative of the general population of wires as some of these were specifically selected for chemical analysis based on their extreme (high or low) stress/strain curves.

Two zinc coating tests were carried out on Stage 1 and 2 wires, the weight of zinc coating test and the Preece test. The weight of zinc coating test measures the average weight of zinc remaining on the wires, and most tests gave a weight in excess of the likely minimum specification of 275 g/m^2. The Preece test gives a qualitative indication of the local thickness of the galvanising and can highlight the presence of thin spots and holidays in the coating. Less than one third of the wires satisfied the criteria for a new wire, and hence the majority of Stage 2 wires were considered soon to become Stage 3.

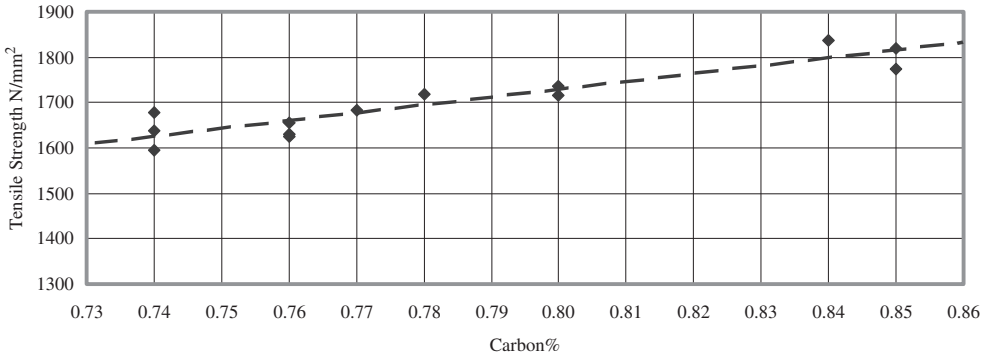

Figure 14. Variation of UTS and proof stress with carbon content.

7 STRENGTH EVALUATION

The cable strength was evaluated using the NCHRP Brittle Wire model. Of major interest and importance in the calculation is the proportion of the wires that have cracks. Of the 46 Stage 4 wires tested, 11 were found to have cracks, which gives a nominal fraction of cracked wires of 0.24. Applying a one-sided confidence limit of 97.5%, the maximum fraction of wires was found to be 0.38.

An examination of the location of the cracked and broken wires led to the observation that they all lay within the outer 6 rings of wires. Therefore calculations were carried out both on the basis of nominal and maximum fraction of cracked wires and also considering either all Stage 4 wires to be potentially cracked, or those only in the outer 6 rings.

These results are summarised in Figure 15 and establish a bandwidth of strength loss of between 8% and 15% from that calculated assuming the cable to comprise new wires.

It was considered that the assumption that all Stage 4 wires in the cable may be cracked was overly conservative for this bridge, but that the assumption that wires were cracked only in the six outer rings was based on opening only 10 panels out of 200. It was therefore recommended that the current strength loss was between 8% and 10%.

In addition to the estimation of current strength, the owner was very interested in the projected future strength over a period of 15 years. Whilst a method for future strength estimation is provided in the NCHRP, it is somewhat tentative and is only intended to predict a further 10% of the bridge's life. In the case of the Forth Road Bridge this equals just 4 years. To achieve a satisfactory model, the cable cross-section was divided into three areas (upper, lower and both sides together), and it was assumed that cracked wires only exist in the outer 6 rings.

This model gave a similar, but slightly smaller current loss in strength, but this was thought to be due to a greater potential error caused by splitting the test data into smaller groups. It is appreciated that there is considerable uncertainty in future strength calculations, as the relevant data from long term tests is not available yet. The assumption of the same deterioration coefficient being valid beyond Stage 4 was made, as was one of twice as large. These calculations showed a similar long term decline in strengths, but at different rates.

8 DETERMINATION OF MAIN CABLE LOADS

A detailed review of the loads in the main cables was undertaken. A full build up of the suspended span weight on completion of construction and the current situation was established. It showed that at opening the bridge was somewhat lighter than anticipated, but that subsequent additions and modifications had brought the weight back up to near the original intention. The dead load is about 158 kN/m in the main span and 210 kN/m in the side spans.

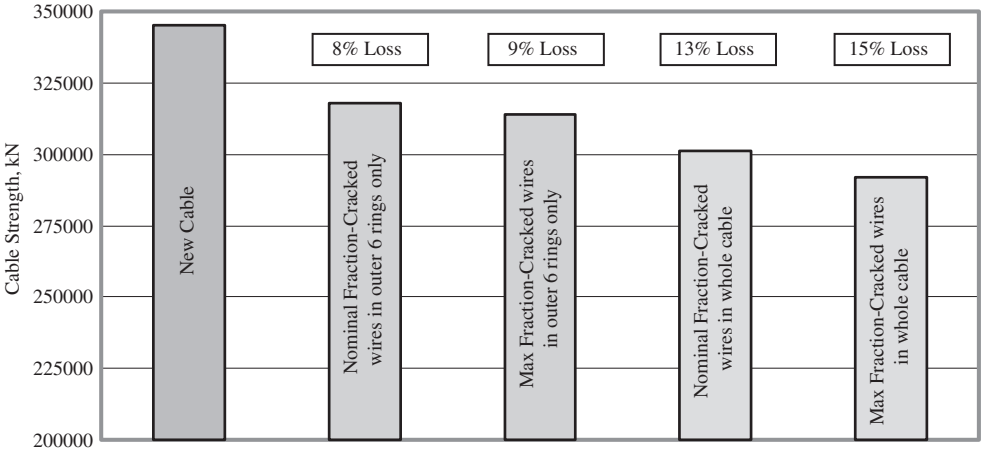

Figure 15. Summary of calculated cable strengths.

Live load has been assessed as a "bridge specific load", using weigh-in-motion equipment and statistical techniques to generate maximum loadings at serviceability limit state. The original design live load was to the then standard BS 153 (BSI, 1954) and assumed a maximum vehicle weight of 22 Imp. Tons. The maximum cable tension is achieved by loading all three spans and for this loaded length the lane loading intensity was 5.84 kN/m. The original design assumed two full lanes plus the other two lanes carrying one third of a full lane, giving a total bridge load of 15.6 kN/m. This was found to have increased by about 75% for current loadings, but as with bridges of this span the cable force is dominated by the dead load.

Interestingly, the footways were assumed to carry the standard design loading and the cycleways half of this, but given the length and width, this equals a significant load (4.46 kN/m). However, the length and location of the bridge means that in practice usage by pedestrians and cyclists is very low, and even allowing for use by FETA's maintenance vehicles and site cabins, the envisaged design loading was unrealistically high. A bridge specific footway loading of 0.3 kN/m was therefore developed reflecting real usage. This reduction helped offset the large increase in vehicular load.

9 CABLE SAFETY FACTORS

The cables were originally designed using a working stress approach with a factor of safety against tensile strength of 2.5. This corresponds to American practice of the day. It appears that the cables were originally slightly overdesigned and including the final weight on completion being less than expected gives an original safety factor of 2.64.

The current factor of safety has been evaluated using the cable strengths determined using the Brittle Wire Model. These are shown in Figure 16 and include the calculation of cable strength in the as-new condition as well as the range of estimated loss of strength of between 8% and 10%.

Some thought has been given to the minimum acceptable factor of safety that is required to keep the bridge open to the public. There appears to be a consensus in the US that traffic should not be run on a suspension bridge for any period with a factor of safety of less than 2.0, although it is known that a few bridges have been open below 2.0 for a short time whilst measures were taken to reduce loads or increase strength. However, these examples have occurred as a result of a severe unexpected problem being discovered during an inspection. The present situation at Forth Road Bridge is quite different as the safety factor is still well above 2.0.

UK bridge codes adopted semi-probabilistic limit state approaches many years ago, but as there are so few major suspension bridges in the UK, these were not covered. Consideration was given to the derivation of appropriate partial safety factors and even reliability methods, but given the

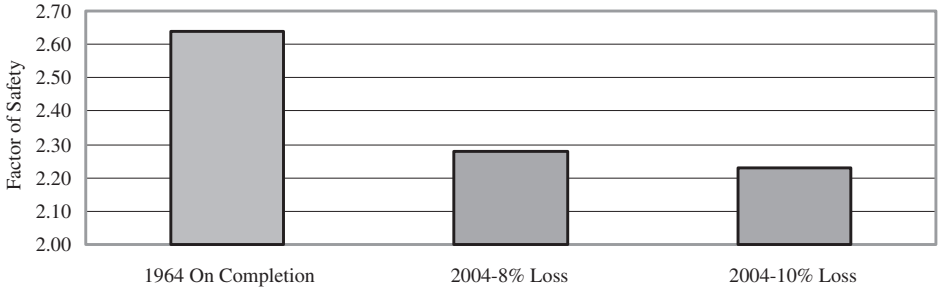

Figure 16. Current factor of safety.

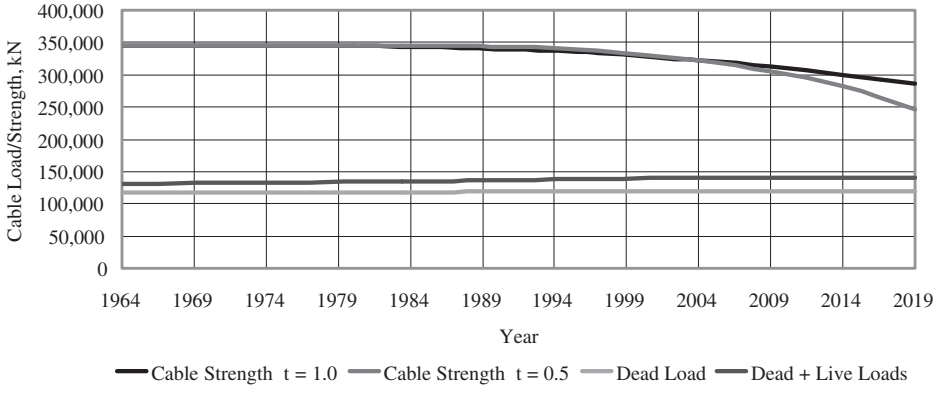

Figure 17. Predicted cable loads and strengths.

limited nature of a first inspection and the inherent lack of certainty in the results, it was agreed to set the minimum acceptable factor of safety as 2.0.

Having confirmed that the bridge cables were safe today, attention focussed on future predictions. Whilst it was appreciated that the future strength predictions could be considered as conjecture, it is nevertheless a vital piece of information for the Authority to help plan future cable management strategy, with associated budgetary requirements.

The predicted changes in cable loading and strength are given in Figure 17, with the corresponding factor of safety in Figure 18.

The more conservative case predicts that traffic restrictions would be required in 2014 (eight years time), and the other 2020. It is not feasible to reduce dead weight on the Forth Road Bridge. Indeed other studies have shown that the stiffening truss is currently theoretically understrength and will require some major strengthening soon. In the worst case it is envisaged that live load restrictions would be phased in, for example by firstly restricting heavy trucks. In this way it may be possible to continue for 5 more years.

The conclusion from this for the owner was very simple. If nothing is done (other than routine work similar to that previously carried out) then the bridge faces premature closure and the likely political and economic consequences would be serious.

A second crossing of the Forth has been contemplated for some time and detailed studies were carried out in the early 1990s, but the project foundered at that stage. The continuing increase in traffic and increasing maintenance challenges (including the main cables) has brought the project back on the agenda. FETA is committed to the principle of a new multi-modal crossing of the Forth and is consulting with the Scottish Executive's Agency, Transport Scotland on the matter.

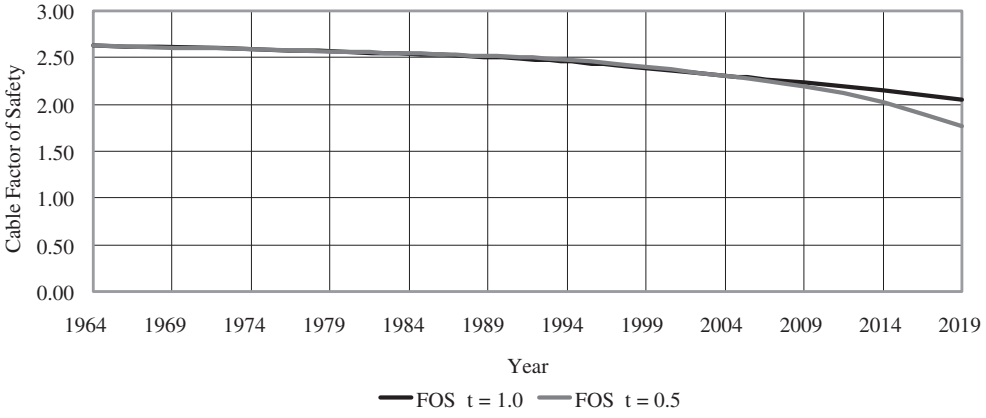

Figure 18. Predicted future factor of safety.

10 ACOUSTIC MONITORING

As this was a first internal inspection, only a relatively small proportion of the cables were subject to detailed intrusive inspection. It could not of course be assumed that the worst panel had been found.

Further, the only data on the long term deterioration of the cables known with certainty are the two points comprising firstly when the cable was completed as-new, and secondly the time of the recent inspection.

The rate of deterioration of the cable is essentially the rate at which wires pass from one stage to the next. Part of this will be the rate at which wires break. The latter is a mechanism that can be measured using acoustic monitoring techniques, and this has been done with increasing levels of confidence on a number of other suspension bridges. A further aspect is that the entire length of the cables can be monitored.

It was therefore recommended that an acoustic monitoring system be fitted. Initial ideas centred on a fully wireless system, as this was thought to minimise work at high level. However, it was realised that the northerly location and consequent low light levels in winter, plus the north/south orientation of the bridge would force the solar panels to face east or west, or both (to enable easy access along the cable), and hence did not offer the best solution.

The tender was therefore based on a fully wired system (i.e. wired power and signal) but contractors were in addition permitted to offer alternatives. Like the cable inspection, the tender comprised quality and price elements. The winning bid was an alternative hybrid system comprising wired power and wireless signal transmission, submitted by Advitam SAS using the Pure Technologies "Soundprint" system. This proposal scored the highest quality mark and it was also the most economic.

Work started on site in April 2006, but it became apparent that the UK radio telecommunications authority would not grant licences for the frequencies required by the system, and so the design has been record as a fully wired system.

11 DEHUMIDIFICATION

The future strength analysis predicts that the bridge will become unserviceable within a relatively short timescale. It was therefore deemed unacceptable for FETA to continue to maintain the cables in the same manner they had done to date. Even repainting would not extend the cable life by much, as it has been realised that the cables must be protected with a "perfect" paint system to prevent water ingress.

The authors visited the U.S. in the fall of 2005 and met with a number of bridge owners. One of the aims was to understand the current thinking on prolonging cable life. The process of oiling was well

known, with the general understanding that it provides protection to individual wires by coating them and also by filling the interstices between them. This has been used extensively in the U.S., however, we are not aware of any conclusive evidence of its long term success. It has a number of drawbacks including leakage of excess oil at low points; it is an expensive operation as the entire length of cable needs to be opened to permit oil to be introduced (followed by compaction and re-wrapping), and the oil will over time gradually dry out making future re-oiling a challenging task. There is also a concern that the oil will not completely cover every wire, and that bare wire may corrode at a faster rate.

A more recent development is cable dehumidification, which has been used on all the Honshu Shikoku suspension bridges in Japan. Systems have been fitted to new structures as well as to existing bridges. Similar systems have been retrofitted to three European bridges, although two of these have stranded cables. There is strong evidence that these systems can significantly reduce the amount of moisture held within the cable. If the relative humidity is reduced below a critical threshold, the rate of corrosion will diminish to a very low level or may even cease.

The authors believe that dehumidification offers the most promising future for the cables of the Forth Road Bridge, and Faber Maunsell has commenced work to study the feasibility, with the intention of installing a system during the summers of 2007 and 2008.

12 REPLACEMENT OR AUGMENTATION OF THE MAIN CABLES

Whilst the Authority are optimistic that the proposed dehumidification will halt the corrosion of the cables, there is no absolute guarantee that this will be the case. Therefore, the Authority had to give consideration to replacing or augmenting the main cables should it become necessary in the future. However, there are several significant engineering difficulties including:

- the main towers, including existing cable saddles, have already been strengthened and new load paths need to be identified at the tower tops
- potential problems with the integrity of the existing anchorages, and the study would need to investigate methods of determining the condition of the post-tensioned strands
- new anchorages will be required
- the existing truss is overstressed and will also require to be strengthened

A critical question is the extent of the carriageway restrictions that will be required to carry out replacement or augmentation of the main cables.

Given the above, FETA decided that a feasibility study should be carried out and a procurement process is currently being undertaken for this work. The study is likely to cost between £1 M and £1.5 M and could take up to 18 months to procure and complete.

13 INDEPENDENT AUDIT

Given the significance of the findings of the first internal inspection, in November 2005 the Scottish Executive appointed Flint & Neill Partnership to audit the results. The purpose of the audit was to carry out a desk study of the findings and to advise the Scottish Executive whether those findings were reached using a process of appropriate rigour and whether the conclusions were reasonable. Flint & Neill appointed New York based Ammann & Whitney to act as sub-consultants providing specialist advice using experience gained from inspections and assessments carried out in America.

In January 2006, a comprehensive audit report to the Scottish Executive (Flint & Neill, 2006) was submitted which concluded that FETA's consultant "has performed the initial internal inspection and cable strength calculation in accordance with accepted practice in the United States and in general conformance with NCHRP guidelines. Certain assumptions have been made that differ from the NCHRP methods but they are reasonable and appropriate considering the particular characteristics of the Forth Road Bridge versus the older US bridges upon which the NCHRP study relied on as case studies".

Flint & Neill note that the initial investigation by FETA "was not prompted by the discovery of any concerns with the Forth Road Bridge cables but as a prudent response to the results of findings in the USA. When the original scope for this initial investigation was determined, the severity of the findings was not anticipated."

Assessment of future strength loss beyond a few years cannot be definitive and the audit report suggested that loading restrictions could be required from 2013. The report continues, "while the corrosion is of a serious nature if left unchecked, there is no urgency to limit loading on the bridge at this time. However, it is strongly recommended that action be taken to protect the cables from further deterioration as soon as practical".

The audit report confirmed FETA's previous decision to install acoustic monitoring equipment to both main cables as soon as possible and to develop a dry air injection system (dehumidification) for construction in the near future. It also recommended that further internal inspections be carried out within five years.

14 CONCLUSION

A relatively young major European suspension bridge has been subject to its first internal cable inspection. The thought that the cables would still be in a good condition was quickly found not to be true. The apparent overall good condition of the paint hid the deterioration that had been taking place within the cable.

The investigation discovered widely varying levels of corrosion in different parts of the bridge, and it was difficult to fully explain the discrepancies.

The current loss of strength is believed to be in the range of 8% to 10%, and the cable factor of safety is between 2.2 and 2.3. However, future projections highlight the likelihood of an accelerating loss of strength and in the worst case traffic restrictions would be required from 2014.

This result has prompted a number of actions including provision of an acoustic monitoring system to help establish and monitor the deterioration rate, and a plan to install a dehumidification system, plus a study on replacement or augmentation of the cables.

The cable investigation project has involved significant expenditure to date (£2.2 m) and a requirement for a greater future commitment.

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Chapter 16

Prevention of main cable corrosion by dehumidification

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ABSTRACT: Corrosion of main cables on suspension bridges is a major problem on a worldwide basis. The load-carrying wires are covered by wrapping wire and corrosion is therefore hidden and often progresses to a very serious level before it is detected. Many examples have illustrated that traditional corrosion protection systems for main cables do not prevent corrosion, but merely slows it down. This has led to the development and application of dehumidification of main cables, which truly prevents corrosion. A dehumidification system blows dry air through the main cables and keeps the atmosphere in the cables so dry that corrosion can not occur. This paper describes the details of such a system, design considerations, results of Life Cycle Cost Analysis and experience from systems in operation.

1 INTRODUCTION

Corrosion of main cables on suspension bridges is a major problem on a worldwide basis. The load-carrying wires are covered by wrapping wire and corrosion is therefore hidden and often progresses to a very serious level before it is detected. There have been some cases where it has been necessary to replace the cables at enormous cost to the owner. In some cases it has even been necessary to close the bridge.

Many examples have illustrated that the traditional protection system for main cables, the so called Roebling system, does not prevent corrosion, but merely slows it down. Older suspension bridges have been able to survive despite corrosion, as the safety factor for the main cables has been quite high, so a decrease of the capacity is acceptable to a certain level. Main cables on newer suspension bridges are generally designed with a much lower safety factor, which makes them more sensitive to corrosion. Relatively serious corrosion attacks on bridge cables have been found on bridges as young as ten years. It is therefore imperative to prevent corrosion of main cables, no matter the age of the bridge. Only one method has been proven to completely prevent corrosion – dehumidification. A dehumidification system blows dry air through the main cables and keeps the atmosphere in the cables so dry that corrosion can not occur.

This paper presents:

- A description of the dehumidification concept
- A description of systems for main cables
- Considerations for designing a system
- Results of Life Cycle Cost (LCC) Analysis demonstrating that dehumidification is much more economical than a traditional system
- Case stories/experience from systems in service

2 DEHUMIDIFICATION CONCEPT

Many of the main components of major bridges are steel structures. In order to ensure a long service life and provide an appropriate level of safety, these structures must be protected from corrosion.

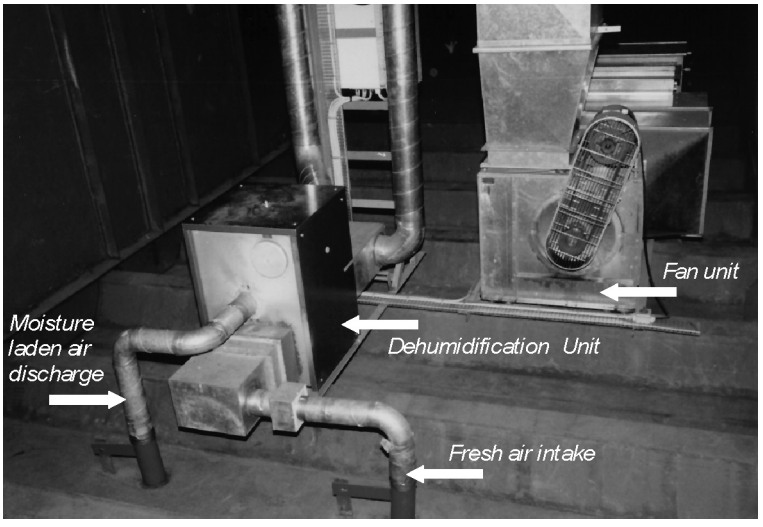

Figure 1. A dehumidification plant in the box girder of the Faroe Bridges, Denmark.

Corrosion protection has traditionally been provided by means of surface treatment, i.e. blasting and painting. In the course of the last 35 years an alternative method – dehumidification – has been developed, implemented and proven. Dehumidification has been proven to be superior to painting in all respects, i.e. technically, economically and environmentally.

The most widespread application for bridges is the protection of the internal surfaces of closed box bridge girders. Dehumidification systems are implemented in new bridges and in existing bridges, which may have insufficient protection or need renewal of corrosion protection.

Dehumidification systems have also been designed and installed in numerous other bridge structures, including:

- Anchor houses on suspension bridges
- Cables saddles on suspension bridges
- Abutment rooms
- Anchorage boxes on the top of cable-stayed bridge pylons

It has been proven by experience that corrosion does not take place if the relative humidity in the vicinity of a steel structure is kept below 60%. This is the basic concept upon which corrosion protection by dehumidification is based.

The concept of dehumidification has been known for many years and it has been successfully applied to a wide range of applications. A dehumidification plant, such as used in bridge structures, is a well known and reliable means of keeping the relative humidity in a closed room under an acceptable level. The use of dehumidification for the corrosion protection of enclosed steel surfaces in bridges has been developed by COWI over the last 35 years. During this time the dehumidification concept for bridges has won international acceptance and is a standard requirement in many countries.

The traditional alternative to dehumidification as a means of corrosion protection for the inner surfaces of a steel box girder is surface treatment by blasting and painting. It has been proven that dehumidification by means of sorption is in all respects the superior method. The main advantages of dehumidification are:

- Dehumidification is virtually 100% effective, providing a much higher level of protection than painting.
- The initial cost of dehumidification is only a fraction of the cost of painting.
- The maintenance costs of dehumidification are much lower than for painting.

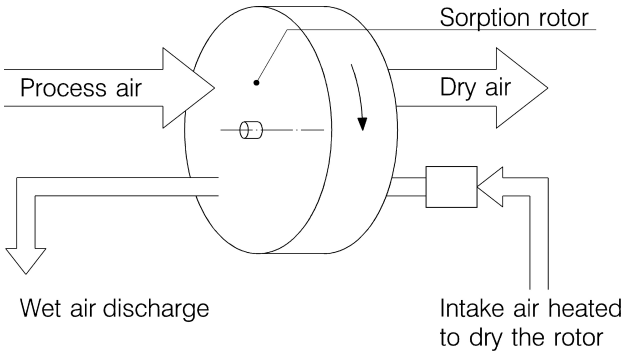

Figure 2. Dehumidification by active sorption.

- The life cycle cost of dehumidification is much lower.
- It is easy to monitor and verify the effectiveness.
- Dehumidification is environmentally friendly, as it does not have the environmental problems which are caused by blasting and painting.

There are two principally different methods of dehumidification: condensation and sorption. Sorption can be either passive or active, the main difference being the use of forced air stream. For bridge structures active sorption is applied, as it is the most effective method.

The passive means of dehumidification by sorption is well known, for example the small sachets of silica gel, which are usually enclosed in packages with electronics or other moisture sensitive products. Larger portions of sorption materials are also used for example for reducing the relative humidity in a damp basement. The passive means is not suitable for corrosion protection of steel bridge structures, where the more effective active means is applied.

The principle for dehumidification by sorption is illustrated in Figure 2. This method works by binding the moisture in the process air to a hygroscopic material (a sorbent).

A sorption system contains a rotor which is built up of many small pipes, coated with a sorbent, most commonly lithium chloride. The process air is forced through the rotor and its moisture is absorbed under this process, resulting in dry air. The rotor turns very slowly, allowing time for the process. On the opposite side of the rotor heated intake air is blown through, which dries out the sorbent coating. This air becomes moisture laden and is subsequently discharged.

This method is efficient for all air conditions, i.e. there are no temperature and relative humidity limits as with the condensation method. This method is generally applied to various bridge structures with excellent results.

3 SYSTEMS FOR DEHUMIDIFICATION OF MAIN CABLES

Systems for dehumidification of main cables are generally based on the same technology as the above mentioned applications, as well as many years of experience with these. There are three main components:

- A sealing system for the main cables, including cable bands, saddles and other connected components.
- A dehumidification system capable of producing and blowing dry air through the main cables.
- A control and monitoring system.

3.1 *Sealing system*

We have carried out extensive research, development, workshop testing and on-site testing to determine the best systems for sealing the main cables, cable bands, saddles and other connected

Figure 3. Wrapping and heat bonding of the two layers.

components. We have concluded that the best system to seal the cable sections from band to band is the Cableguard™ Wrap System from the D.S. Brown Company. This is an elastomeric wrap with a thickness of 1.1 mm and a width of 200 mm. It is applied with a 50% overlap, so the total thickness is 2.2 mm. It is applied under tension with a special wrapping machine. After wrapping a section it is heat bonded with a special heat blanket, which melts the two layers together and shrinks the material slightly, giving an even tighter fit. Special details have been developed to ensure sealing at the transition to the cable bands and to give a uniform appearance. The main advantages of this system are:

- Lower life cycle cost
- Able to withstand sufficient overpressure
- Environmentally friendly, no paint products to remove or apply
- Can be supplied in numerous colors – requires no paint
- Good working environment, no fumes and no blasting work
- Execution is less sensitive to poor weather
- Shorter construction period
- Virtually maintenance free
- Long lifetime – UV and weather resistant
- Easy to remove and replace – does not bond to the cable

For sealing of cable bands, saddles, injection and exhaust collars and other components special details have been developed. These are generally based on a double barrier system with a combination of sealer strips and adhesive caulk. Materials which have been applied for similar usage over many years on bridges are generally applied.

3.2 *Dehumidification system*

The dehumidification system produces dry air and blows it through sections of the main cables. The system assures overpressure inside the sealed cable system. While the sealing system may have minor imperfections in the form of small leaks, no water or moisture will enter the cables, as the overpressure will prevent this.

The dehumidification system is made up of the following main components:

- Dehumidification plants
- Injection points
- Exhaust points

A layout, such as shown below, is developed for the dehumidification system. The layout defines the positions of the dehumidification plants, injection and exhaust points as well as the flow sections.

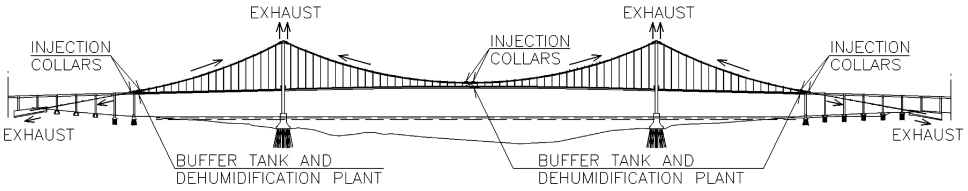

Figure 4. Dehumidification system layout, Little Belt Bridge, Denmark.

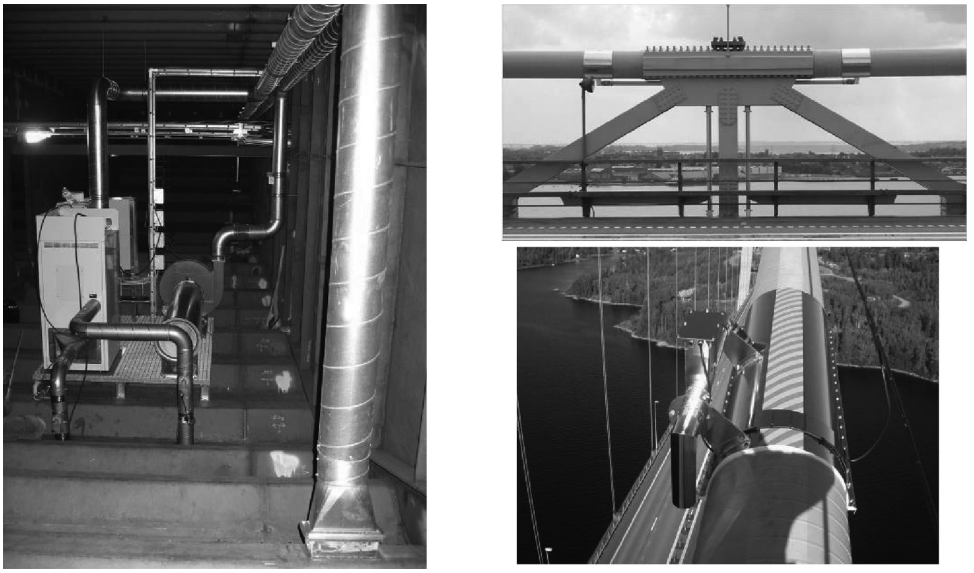

Figure 5. Dehumidification plant (left), injection (upper) and exhaust points (lower).

3.3 *Control and monitoring system*

The dehumidification plants are placed in buffer tanks for several reasons:

- This provides a protected atmosphere.
- Good access.
- Electrical consumption is minimized as the air is mixed up to 40% RH instead of the app. 0% RH that the dehumidification units produce.

The main components of a dehumidification plant are a dehumidification unit, a fan, an electrical board, filters and ducting, such as illustrated below.

Injection points are established by either modify existing bridge components, such as the saddles or by designing purpose suited injection collars, such as illustrated below. Exhaust points are established in the same manner.

3.4 *Control and monitoring system*

The control and monitoring system allows adjustment of the system and data from the system documents that the system is performing properly. Instrumentation is arranged at the dehumidification plants, in the buffer tanks and at injection and exhaust points. These instruments and plants are connected to local PLCs (Programmable Logical Computer), which in turn are connected to a

central computer, which stores all data. From the central computer it is possible to adjust the system and monitor the readings from all the instruments. Key data to be monitored includes system functionality, relative humidity, temperature, flow, pressure.

4 KEY DESIGN CONSIDERATIONS

The key design considerations for a system for dehumidification of main cables are basically the same considerations that are generally applicable to suspension bridge design:

- Optimization of design – lowest Life Cycle Cost (LCC)
- Accessibility
- Durability
- Verifiability
- Minimal traffic disruption

A system for dehumidification of main cables is in itself the optimal corrosion protection system for main cables, cf. chapter 5. The system for each bridge should however be optimized. Optimization starts with the layout, which should be suited to the cable lengths and utilize the bridge's existing elements as integrated parts of the system. An example of this would be to utilize the enclosed room of a box girder as a buffer tank. Electrical consumption should be kept at a minimum by utilizing features such as buffer tanks and minimizing the overpressure in the cables. Utilization of well documented materials with a long lifetime should also be included in the design. When new details are developed these should be proven and further optimized by full scale testing.

All components which require service should be easily accessible without disturbing the traffic. Dehumidification plants and instrumentation can for example be placed inside the box girder and pylons. As much as possible no components should be installed along the main cable, as this would be detrimental to access to the cables and be an obstruction when carrying out other maintenance activities.

The above mentioned placement of service requiring components also improves the durability of these components and minimizes the LCC. A monitoring system should be designed such that at all key data to verify the systems functionality and corrosion protection of the main cables is well documented.

5 LIFE CYCLE COST ANALYSIS

A Life Cycle Cost (LCC) Analysis should be carried out to determine the correct methods to be applied to the details of a bridge structure. In such an analysis all costs connected with each detail are considered, including construction, maintenance, repair and replacement. Traffic disruptions due to the works should also be capitalized. The present values for all costs are calculated by use of a compound interest rate, usually in the range of 4 to 6%. The detail with the lowest net present value (sum of present values of all costs) is the optimal detail.

Such analysis has earlier been carried out for corrosion protection of the inner surfaces of a box girder by means of dehumidification contra traditional protection by painting. The conclusion was that the net present value of dehumidification was only a few percent of the net present value of a paint system, resulting in enormous savings during the lifetime of the bridge.

In connection with completed projects we have carried out LLC Analysis for the following cases:

- Renewal of corrosion protection for main cables on existing bridge
- Corrosion protection of new main cables

The conclusion in both cases is that a dehumidification system including elastomeric wrap is by the far optimal solution. The results of these analyses are included in the following two sections.

In the following examples we have not included the costs for:

- Traffic disruptions (extra driving time for users) as this is not standard practice in all countries.
- Traffic regulations (signs, barriers and the like), as this varies greatly from country to country.

If these costs were included a dehumidification system would be even more advantageous, as there are fewer and shorter periods with traffic regulation and disruption.

The costs in the examples are based on European experience from actual projects. The costs are given as reference costs (not in monetary values), as local conditions may give substantial variation. The purpose of the results is to illustrate the relative difference and the economic advantage of a dehumidification system.

The calculations are carried out for a period of 60 years, such that a sufficient amount of major repairs and replacements are included in all strategies, hence the results are representative for the entire lifetime of the bridge.

5.1 *Renewal of corrosion protection*

In this example we have calculated the life cycle costs for the following strategies:

1. Application of elastomeric wrap and a dehumidification system
2. Renewal of paint

The following lifetimes and assumptions for strategy 1 are utilized:

- Elastomeric wrap – lifetime 30 years
- Exposed ducts and details – lifetime 30 years
- Dehumidification system – lifetime 60 years
- Electrical consumption and yearly service included in costs

The following lifetimes and assumptions for strategy 2 are utilized:

- Removal of paint and application of new paint system – lifetime 20 years
- Spot repairs – every 5th year
- The paint system is not red lead

An alternative to strategy 2, strategy 2a, has also been calculated. In strategy 2a the lifetime of the new paint system is more optimistically assumed to be 30 years, such that the following lifetimes and assumptions for strategy 2a are utilized:

- Removal of paint and application of new paint system – lifetime 30 years
- Spot repairs – every 5th year

The results of the LCC analysis – net present values – are shown in Figures 6 and 7. The strategy with the highest LCC for each interest rate has been given index 100 and the LCC for the other two strategies are given relative to this.

The results of the analysis clearly show that strategy 1, dehumidification and wrap, is the optimal strategy. As the initial investment for all three strategies are calculated to be of the same order (strategy 1 is app. 10% lower) the difference between strategy 1 and 2 in Figure 6 is not as substantial as for operation and maintenance alone, as shown in Figure 7. The results in Figure 7 also illustrate

Strategy	Compound interest rate		
	4%	5%	6%
1. Dehumidification incl. wrap	70	72	75
2. Paint 20 years	100	100	100
2a. Paint 30 years	84	86	87

Figure 6. Results of LCC including initial investment and operation and maintenance costs over 60 years.

that dehumidification is much more maintenance friendly, as there is a substantial difference from the other strategies.

If the following costs were taken into account dehumidification would much more advantageous:

- It is a well known fact that the main cables will deteriorate to a certain degree when a painting strategy is applied. This results in enormously expensive intrusive inspections and rehabilitation works. These costs and the lost value due to deterioration (reduced load carrying capacity) are not included in the analysis.
- If red lead paint is to be removed/applied the painting strategy would be even more expensive due to environmental costs.
- A painting strategy requires many long traffic disruptions and the costs of traffic regulations and lost time for users are not included.

5.2 *Corrosion protection of new main cables*

In the case of new main cables there are even greater savings to be made by applying dehumidification. Besides the savings mentioned above there are also the following savings:

- The zinc paste and wrapping wire can be omitted.
- Shorter time schedule due to these omissions – indirect savings.
- Less dead load to carry – indirect savings to structures (main cables, pylons, anchorages and foundations).

In this example we have considered a large suspension bridge and calculated the life cycle costs for the following strategies:

1. Traditional system with zinc paste, wrapping wire and paint
2. Dehumidification system with elastomeric wrap (no paste or wrapping wire)

The results of the LCC analysis – net present values – are shown in Figure 8. For illustrative purposes the LCC for strategy 1, traditional system, has been given index 100 and the other life cycle costs are given in relation to this. A compound interest rate of 4% has been applied and a representative period of 60 years is considered.

The results of the analysis show very clearly that strategy 2, dehumidification and wrap, is by far the optimal strategy. If the additional costs concerning strategy 1, traditional system, as described

Strategy	Compound interest rate		
	4%	5%	6%
1. Dehumidification incl. wrap	47	44	42
2. Paint 20 years	100	100	100
2a. Paint 30 years	66	65	63

Figure 7. Results of LCC for operation and maintenance costs over 60 years (excl. initial investment).

Life Cycle Cost	Strategy	
	1. Traditional	2. Dehumidification
Initial construction	40	8
Operation and maintenance	12	4
Indirect construction*	48	0
Total LCC	100	12

Figure 8. Results of LCC for a 60 year period with interest rate 4%.
* Cost due to dead load of zinc paste and wrapping wire on structures.

in Section 5.1 (traffic disruption and cable deterioration) are also taken into the account the conclusion would be even stronger.

6 CASE STORIES

6.1 *Little Belt Bridge, Denmark*

The Little Belt Suspension Bridge carries the E20 motorway and connects east and west Denmark. The bridge was opened in 1970 and it carries three lanes of traffic in each direction. The suspension bridge has a main span of 600 m and two side spans of 240 m for a total length of 1,080 m. The approach spans have a total length of 620 m, giving a total length of 1,700 m for the entire connection.

The superstructure is a steel box girder with a width of 33.3 m. It is the first bridge structure in the world to be protected from corrosion internally by means of dehumidification.

The main cables are each approximately 1,500 m long with an outer diameter of 580 mm and are made up of helical strands. The original corrosion protection is a modified Roebling system, made up of: 1. Galvanized wires in the strands, 2. Zinc paste on the bundle of strands, 3. Galvanized wrapping wire and 4. Paint on the surface of the wrapping wire.

In 1996 the lifetime of the surface paint was nearly depleted and a series of activities was instigated to determine the optimal strategy for the future corrosion protection. This included in-depth inspections, testing of paint and wrapping systems to seal the cables and pressure and flow tests. Based on the experience from these investigations a technical and financial analysis was carried out, including Life Cycle Cost Analysis. The optimal solution was determined to be wrapping with elastomeric wrap (Cabelguard™) combined with dehumidification.

The works were carried out within the time schedule during 6 months in 2003. The layout of the system is illustrated in Figure 4. The system has been in service for 3 years and is performing well, as confirmed by data from the monitoring system and inspections. Data from this period indicates that the relative humidity of the dry air supply in the buffer tanks is generally about 40–45% and that the corresponding exhaust air is generally in the range of about 35–55%, see Figure 10. No leakage of the sealing system has developed, which is documented by data from the monitoring system, as the ratio between exhaust and injection flow has been constant.

The buffer tanks ensure a minimal running time of the dehumidification units, which are the only components with a relatively high electrical consumption. The total electrical consumption of the entire system is very low, app. 20,000 kWh per year.

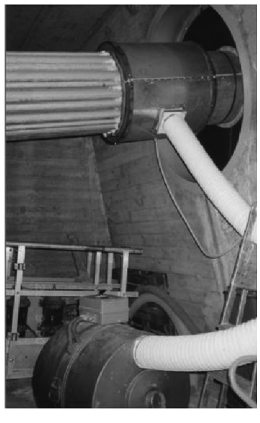

Figure 9. Little Belt Bridge, overview and on-site injection and flow test.

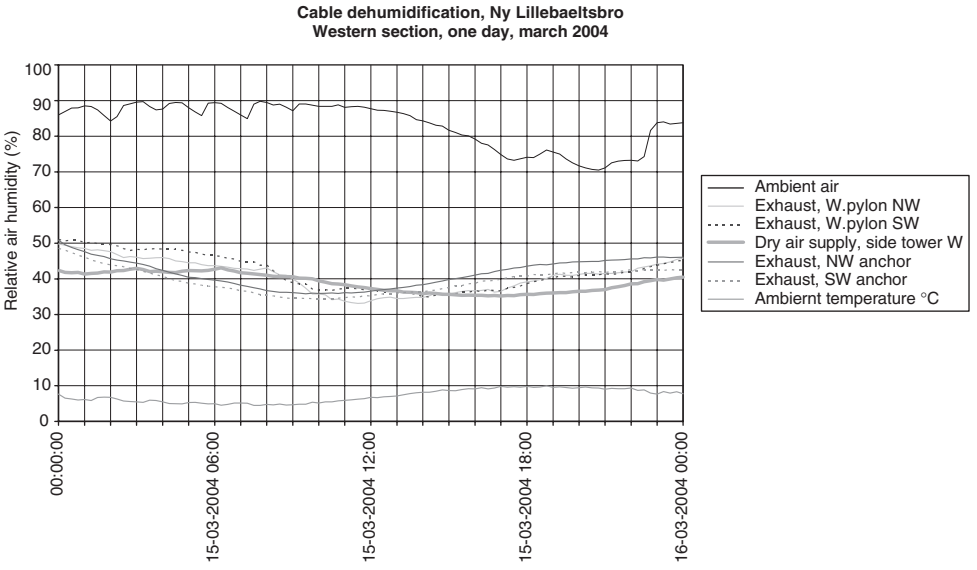

Figure 10. One day graph: relative humidity of injection air, exhaust air and ambient air and temperature.

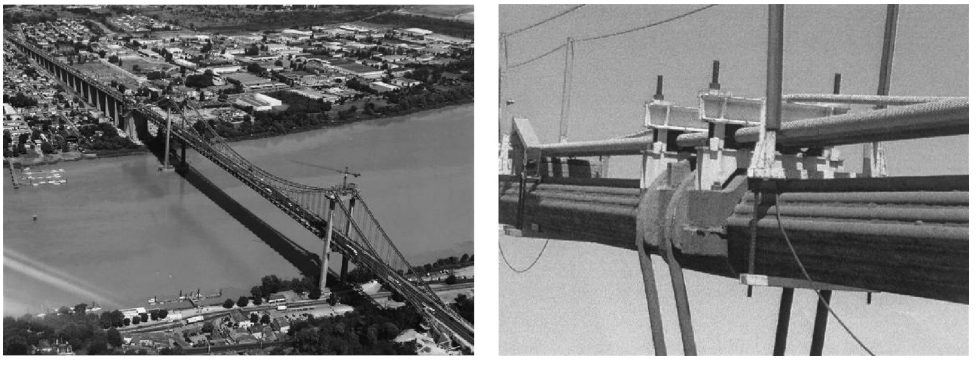

Figure 11. Aquitaine Bridge, overview and temporarily strengthened main cables.

6.2 *Aquitaine Bridge, France*

The Aquitaine Suspension Bridge crosses the Garonne River in Bordeaux, France and was opened in 1967. The suspension bridge has a 394 m long main span and two 143 m long side spans for a total length of app. 680 m.

The original main cables were each composed of 37 locked coil strands, see Figure 11. Due to tradition in France at that time the wires of the strands were not galvanized. The only corrosion protection for the main cables was therefore the paint applied to the surface of the strands, which was only maintainable on the outer surface of the outer stands on the stretches between cable bands. These circumstances resulted in serious corrosion problems, which were already detected at a very early stage. As early as 1979 a number of ruptured wires were detected. The cable deterioration continued over the following years and was closely monitored. The worst damages were located at the cable bands, where moisture could accumulate and it was not possible to maintain the paint. Ruptures occurred as deep as 4 layers into the strands. Due to the serious condition of the main cables the owner (La Direction Départementale de l'Equipement de la Gironde) decided

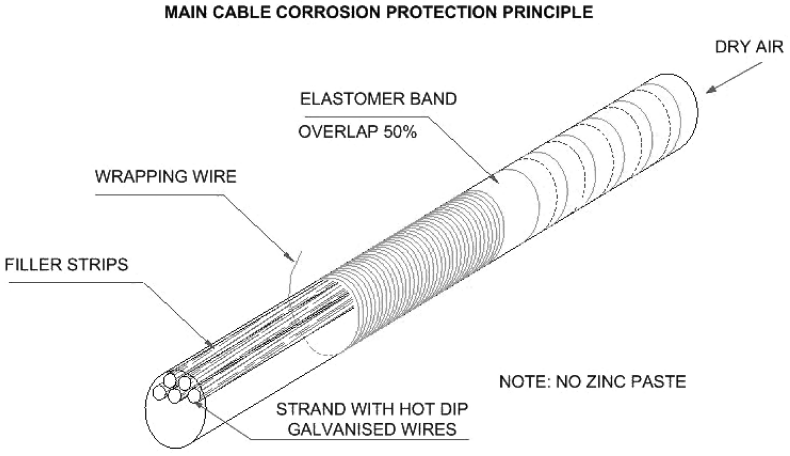

Figure 12. New corrosion protection system.

in 1999 that the entire cable system should be replaced. The carriageway should at the same time be expanded from four to six lanes. The corrosion protection system for the new main cables should obviously be upgraded to the present state of the art.

The corrosion protection system for the new main cables is composed of: 1. Hot dip galvanization of all wires in the strands 2. Hot dipped galvanized wrapping wire, 3. Elastomeric wrap and 4. Dry air flow, see Figure 12. The normal zinc paste layer has been omitted, as it is not necessary. This gave substantial savings and allowed a much faster installation of the wrapping wire, which was performed in the course of just one week per main cable.

A dehumidification plant is located at the top of each pylon and feeds dry air into the main cables, which flows to exhaust points at the middle of the main span and at the anchorages. The control and monitoring system includes:

- Ambient temperature and relative humidity
- Relative humidity, temperature and pressure at injection and exhaust points, as well as at intermediate points

The construction was completed in 2003 and the system has been performing well. The relative humidity of the exhaust air is quite constant at an average level of app. 25%. Based on operation experience the system can be further optimized to allow a more economical operation. The dehumidification plants can be controlled by the relative humidity of the intermediate points and thereby only run when it is necessary. The target level for a possible optimization is 40% RH with brief peaks of up 50% RH allowed.

6.3 *Högakusten Bridge, Sweden*

The Högakusten Suspension Bridge carries the E4 over the Ångerman River about 400 km north of Stockholm, Sweden and opened in 1998. The main span is 1,210 m long and the side spans are 310 m and 280 m long, giving a total length of 1,800 m. The main cables are made up of parallel wires with an outer diameter of 650 mm. Each cable is approximately 1,900 m long. The corrosion protection system for the main cables was design as a traditional system with galvanized wires, zinc paste, wrapping wire and paint.

From the very beginning it was observed that large amounts of water flowed through the cables and especially after rainy periods water could be seen dripping out of the open lower joint of the cable bands and the drain holes in the central node. Water ingress was due to several conditions. During construction it was decided to omit the zinc paste. This fact combined with a somewhat defective paint system and other details allowed water to enter the cables and led to an accelerated

Figure 13. Högkusten Bridge, cable works commenced and condition of wires on bottom of cable.

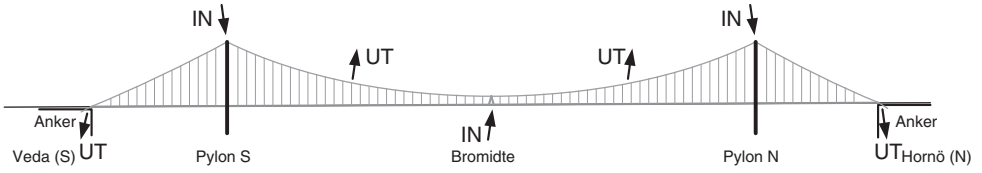

Figure 14. Layout of dehumidification system.

deterioration of the corrosion protection. The Swedish Corrosion Institute inspected the cables and analyzed the drainage water. The conclusion in 2003 was that the zinc galvanization on the cable wires would be depleted within about ten years. The wrapping wire was removed over a 2 m section at the low point by the central node in 2004, see Figure 13, and the condition here confirmed the prediction. The zinc was depleted and there were signs of ferrous corrosion on the bottom wires of the cable and the inside of the wrapping wire.

In early 2004 COWI was awarded the project for a new corrosion protection system to be based on dehumidification and experience from the two above mentioned bridges. Based on the outline design studies a layout with flow sections of app. 310 m was chosen. The system is arranged with buffer tanks of dry air in the top of the pylons and in the bridge box girder at mid span. From the pylons the dry air flows to exhaust points at the anchorages and half way to the mid span. From the box girder at mid span dry air flows to the mutual exhaust points, half way to the pylons.

The control and monitoring system includes measurement of: 1. Buffer tanks: relative humidity, temperature and functionality of plant, 2. Injection points: dry air flow, 3. Exhaust points: relative humidity, temperature and dry air flow and 4. Ambient: relative humidity and temperature.

Despite 25% downtime due to poor weather, the works were completed in just six months, from April to October 2005. The system has now been in operation for about 7 months and data from the monitoring system indicates that the system is functioning well and that the sealing of the cables has been done very well. As expected the exhaust air is so far quite moist, as it will take some time to dry out the large amounts of water in the cables. The monitoring system also calculates the water content in the injection and exhaust air and the difference between the two levels has been on the average app. 1.5 g/kg. This corresponds to an average of app. 1 litre water per day, which is removed from each cable stretch. When the drying out process is completed it will be possible to calculate the total amount of water removed from the cables. Further analysis of monitoring data is currently underway.

Figure 15. Overview of the Messina Bridge.

6.4 *Messina Bridge, Italy*

The Messina Bridge, which will connect Calabria with Sicilia in Italy, has a main span of 3,300 m and 380 m tall pylons. There are four main cables, each with a diameter of 1.2 m.

The bridge authority Stretto di Messina S.p.A. required in the 2005 tender project that dehumidification of the main cables be analyzed with regards to feasibility and Life Cycle Cost and compared with a traditional corrosion protection system. Dehumidification was to be applied if found advantageous. Based on experience from the dehumidification systems on other bridges, a dehumidification system for the main cables was developed. Dehumidification plants are placed in buffer tanks in the top of both pylons and at two positions in the bridge girder. Dry air from the buffer tanks is injected in the main cables and flows to exhaust points. The results of the Life Cycle Cost Analysis indicate that the construction cost for a dehumidification system is 77% less and that the net present value of operation and maintenance over the first 60 years is 71% less than a traditional system. There are also substantial indirect savings in construction costs due to lighter cables.

7 CONCLUSION

A dehumidification system, composed of a sealing system, a dry air system and a monitoring system, is in all regards the state-of-the-art method for protecting main cables from corrosion. This is the only system which completely prevents corrosion, whereas other systems at best can only slow it down.

A dehumidification system can be integrated in the design of a new bridge or new cables, such as the Aquitaine Bridge, or it can be designed to suit the needs of an existing bridge such as the Little Belt Bridge or the Högakusten Bridge. Furthermore, it is equally well suited for main cables made up of parallel wires, such as the Högakusten Bridge, and cables made up of strands such as the Little Belt Bridge or the Aquitaine Bridge.

The major advantages of dehumidification system contra traditional systems are:

- Corrosion is completely prevented.
- Monitoring ensures that this is well documented.
- The Life Cycle Cost is much lower, app. 50–80% lower.
- Environmentally friendly – no toxic waste from removal or application of paint or paste.
- Safer for workers – no exposure from removal or application of paint or paste and generally better working conditions.
- Construction is less sensitive to weather conditions and much faster.
- As zinc paste is not necessary the wrapping wire can be applied much faster than usual, giving direct savings as well as extra time savings.
- Much less traffic disruption due to a shorter construction period and much less maintenance.

The major advantages of a dehumidification system for main cables are clear and the technology is well proven. Many bridge owners have already wisely decided to implement such a system and prevent corrosion of the main cables so they will serve the bridge and its user for many generations to come. All bridge owners should follow suit and ensure the safety of their cables and thereby their bridges.

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6 Wire rope and strand applications

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Chapter 17

Electromagnetic inspection of wire ropes – vertical lift bridges

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ABSTRACT: Recent visual inspections of vertical lift bridges performed by various companies have raised concerns that wire ropes used in the systems contain established internal and external corrosion and severe external wear where the ropes rub and vibrate against spacers and attachments. The majority of these lift bridges exceed 40 to 50 years in age with some as old as 90 years. With the exception of a few bridges, only visual inspections have been conducted on these wire ropes. Improvements and reductions in the physical size of existing instrumentation have made it possible to inspect these wire ropes nondestructively on a periodic schedule. This same type of inspection can be conducted on static wire rope or structural strand up to 5 inches in overall diameter.

1 INTRODUCTION

The continuing deterioration of the aging infrastructure, including the corrosion and degradation in the wire ropes used on vertical lift bridges is a problem of great importance that needs to be addressed. The so called dressing used in the past, on these wire ropes is a simple preservative and has no inhibiting affect on the internal corrosion or abrasive wear. Older technology greases and dressings were used to coat the external wires of the rope structure and had no penetrating properties that protect the wires inside the structure. Consequently, the structural integrity of these ropes is compromised. Electromagnetic inspection can assist in locating loss of cross sectional area, due to corrosion in wire ropes. Broken wires, both internal and external can be detected along with localized areas of damage cause by abrasion and wear. Wire ropes are manufactured with wires that have been drawn to diameters with tolerances of .0005 of an inch (Wire Rope Technical Board 1993). These tolerances are closer than those called for in turbine designs. When corrosion and/or damage occur, these "high tolerance" wires cannot slide or work within the rope structure. This phenomenon causes the rope structure to deteriorate at an accelerated rate.

Electromagnetic inspection (EM) of wire ropes has been utilized in the mining industry since the early 1950's in gold mines in South Africa. These mine shafts are extremely deep, in excess of 8000 feet. Some method of advanced wire rope inspection was necessary to help prevent sudden hoist rope breakage. As many as 200+ mine personnel ride these rope hoisted cages to the work-place on a daily bases. Wire ropes in this application are virtually "life lines". Wire rope hoists are also used for placing and extracting mining materials and equipment. Broken ropes cause costly downtime and damage to mine equipment and possible fatal injuries to mine personnel. EM of wire rope is also widely used for the inspection of guy strands on towers and flare stacks, lift and tram way ropes in the ski industry. Ropes on vertical lift bridges work in the same manner as ropes other applications, just much more slowly.

Bridge ropes remain in static positions for long periods of time and in many situations, this is more damaging, due to improper dissemination of the lubricants, to the internal rope structure than movement. Different types of wire rope lubricants (Anderson 1988) have been studied extensively, but, old lubricants must be removed, at least partially, before new lubricant is applied. The length of a rope's serviceable life can be greatly affected by the type(s) of lubricants applied.

In order to pinpoint defects and corrosion explained in the previous paragraph, electromagnetic inspection must be employed using approved instrumentation and qualified inspection personnel (ASTM E-1571–01).

2 VISUAL INSPECTION OF WIRE ROPE AND STRAND

The simplest, but not necessarily the cheapest, method of inspection of bridge ropes is visually. However, visual inspection is not the easiest. Typically, a rope is coated with heavy, hard asphalt based grease, usually applied sometime in the last 5–10 years. The external rope structure cannot be seen through this layer of grease and dirt. Therefore the ropes must be cleaned thoroughly prior to the visual inspection. Cleaning can be very expensive and time consuming. In addition, the old lubricants must be disposed of and, by today's standard, lubricants and preservatives are considered hazardous waste. In the case of a vertical lift bridge, the grease must be removed over a waterway. This problem can cause the price of inspection to sky rocket. It would make more sense to inspect the rope to determine its condition prior to removing the coatings and dealing with disposal problems. If the rope is clean, visual inspection can be performed on the external wires of the rope. Diameter measurements can be taken frequently over the body of the rope. The data from the measurements can give an indication of extreme external wear or stretching of the rope structure.

Visual inspection must be performed adjacent to, or around end terminations and attachments. EM inspection will not give accurate readings adjacent to these attachments, due to the proximity of large masses of ferrous, magnetic material.

3 REASONS FOR DETERIORATION

There are many factors relating to the deterioration of wire ropes and strands that can be accurately detected or depicted by using an approved method of electromagnetic wire rope (Haynes & Underbakke 1980) inspection:

3.1 *Loss of Metallic Cross Sectional Area (LMA)*

Loss of Metallic area in a rope structure can be caused by many factors including corrosion, internal wear, interstrand nicking and abrasion. Vibration at end attachments and spreaders can cause LMA from abrasion. Dragging a rope along the ground or across structures will result in loss of metallic area in the form of external wear. Interstrand nicking is caused when the wire rope core, for many reasons, no longer supports the strands in the rope. Corrosion occurs when the wire rope is not protected from outside influences such as a saltwater environment, splash of road salt from passing vehicles and nearby emissions of foreign substances such as plant emissions and hydrocarbons. These areas of deterioration can be highly localized or spread evenly throughout a wire rope. Typically, the cause of corrosion, wear and interstrand nicking can be avoided by the use of a good lubrication/preservative program over the life of the rope or strand.

3.2 *Local Faults (LF)*

Local faults include broken wires, missing wires, pitting, localized corrosion and/or wear. Broken wires are caused by fatigue, mechanical damage, abuse and Martinsite or hydrogen embrittlement and several other factors. Crushing and damage due to outside forces and abuse are also referred to as local faults. Pitting is caused by corrosion.

Some newly manufactured wire ropes and, to a lesser degree, strands, contain local faults. During the manufacturing process, several complicated steps are carried out and, therefore, an insignificant number of broken internal and/or external wires are allowed to be passed on to the consumer. These defects do not necessarily affect the structural integrity of the rope or strand. The cost of a planned,

"perfect" rope would be prohibitive. That's not to say that many ropes exceeding 10,000 feet in length have no defects whatsoever. The Quality Control measures adhered to in the manufacture of domestic wire rope and strand are extremely stringent.

4 NDT METHODS

Many NDT methods for the inspection of wire rope and strand have been explored over the years, including acoustic emissions, radiography at end terminations and attachments and others, but the use of the electromagnetic inspection method is the only proven practical way to efficiently inspect these products. EM type wire rope testers have been successfully used for several decades in a number of countries, including several in Europe, North America and South Africa (Geller et al. 1994). In many countries, including the United States and Canada, their use has been mandated by appropriate governmental legislation. In addition, the visual method of inspection must be used as an important aid to electromagnetic testing.

5 LIFT BRIDGE CONFIGURATIONS

There are several basic designs of vertical lift bridges, but most contain a counterweight system consisting of massive concrete counterweight blocks suspended in towers at both ends of the bridge span. While the bridge is in the lowered position, the blocks hang in the tops of the towers situated at each end of the span. While the span/bridge is being raised, the blocks are kept from rotating by the use of simple guide beams. When the road span is in the raised position, the blocks are near street level.

5.1 *The vertical lift bridge is typically powered by one of two methods*

In a pure counterbalanced situation there are several balance wire ropes which are attached to the four corners of the span. This method of operation is designed to use small, low horse power motors. The ropes are run over large sheaves mounted in the tops of the towers at the ends of the bridge. The sheaves are mounted on pillow block types of idler bearings. Electric motors are installed at the center of the bridge span. These motors are run at synchronous speeds in order to the keep bridge level during operation. The size of these motors need be only enough to overcome the friction in the counterweight sheaves mounted in the towers. When the motors are activated they, through various gear reductions, rotate a drum which pulls or pays out a wire rope that is reeved from the drum, at the center of the span to one end of the span, through a sheave and up to the top of the tower through another sheave and back down to dead end on the end of the span deck. The only load subject to these "activation" wire ropes is the weight of any excess ice or dirt on the bridge deck or the counterweight blocks, the change in weight of the counterweight ropes and friction. Typically, the up and down haul ropes range from 1 to 1½ inches in diameter. The counterweight ropes range from 1½ to 2¼ inches in diameter and number as high as 24 ropes on each corner of the bridge.

In a counterweight driven situation the movement of the bridge while requiring higher horsepower motors, is quite simple. The wire ropes are run over large grooved drums in the tops of the two towers. Electric motors, typically one for each set of drums rotate the drums and the ropes are moved because of friction between the rope and the drum. The motors in both towers have to be somewhat synchronous in order to keep the bridge level. These ropes typically run from 1½ to 2¼ inches in diameter and number as many as 24 on each corner.

Other wire rope systems utilized in vertical lift bridges are passenger and vehicle barriers that raise and lower with the operation of the bridge. These barriers are designed to stop a vehicle that may have lost its brakes, etc.

If any of these ropes fail, the bridge, at a minimum, would be placed out of service for an extended period of time. If a counterweight rope within the system fails during operation drastic

circumstances could result. Typically, all ropes in the system are placed in service at the same time; therefore, the condition of all ropes may be somewhat equal depending on their function. If one of the balance ropes failed, it is possible that the others would fail in a "zipper" type of an effect. The total load would be transferred to other ropes in the system. One way to prevent this situation is to perform electromagnetic inspection on the ropes on a reasonable periodic schedule.

6 ELECTROMAGNETIC INSPECTION – THEORY OF OPERATION

A magnetic circuit is created around and through a specimen of wire rope as depicted in Figure 1. The magnets oriented in the ends of the sensor are arranged so that the magnetic current is directed from a positive to a negative pole. The flux bar carries the magnetic field current from one pole to another, the magnetic current, following the path of least resistance is directed through the wire rope. The flux bars must be designed so that the total cross sectional area of the bars are equal to or greater than the total cross sectional area of the wire rope or stand under test. See Figure 1.

The sensor head is constructed in a manner that it can be placed onto and removed from a wire rope easily. (See Figure 2).

6.1 LMA-Loss of Metallic Area

Once the unit is placed on a rope, the hall sensors in the ends of the sensor head sense the magnetic current within the rope and convert it to voltage. Voltage is then converted to percent of Loss of Metallic Area, LMA or weight of the rope in pounds per foot. The rope is moved through the head or the head is moved over the rope. The LMA is relative from the best part to the worst part of the rope. The original cross sectional area of the rope can be entered into the test data, giving

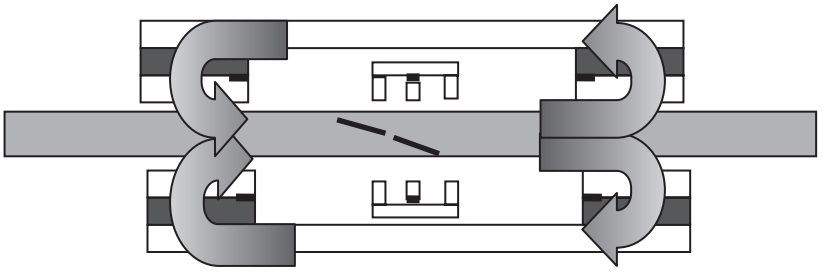

Figure 1. Sensor Head Diagram.

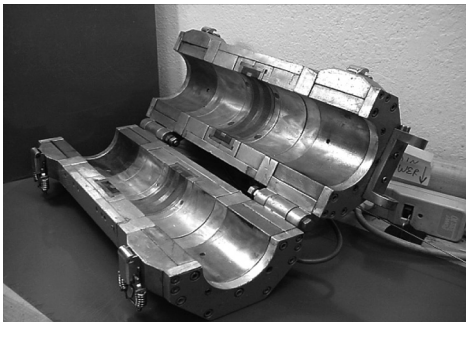

Figure 2. Sensor Head.

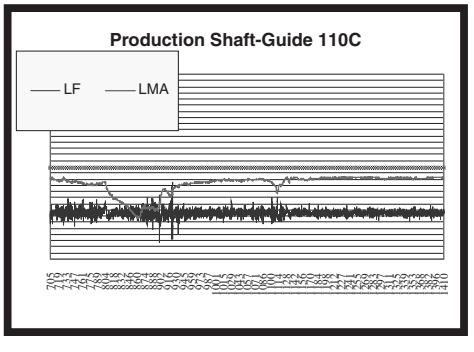

Figure 3. Data Chart.

metallic area content as new. The data is recorded digitally and placed in a strip chart presentation displaying the LMA in percent from original. (See Figure 3).

7 RETIREMENT CRITERIA

Retirement data is mandated and provided by various agencies including the mining industry, the ski industry, various States and Provinces in Canada and other governmental agencies like OSHA and MSHA. The most widely used retirement criteria pertaining to the NDT of wire rope is CFR 30 Part 75 (Code of Federal Regulations, CFR 30, Part 75) which is associated with mining hoist ropes and may be utilized as a guide line for other wire rope use.

§ 75.1434 Retirement criteria:
Unless damage or deterioration is removed by cutoff, wire ropes shall be removed from service when any of the following conditions occurs:

(a) The number of broken wires within a rope lay length, excluding filler wires, exceeds either –
  (1) Five percent of the total number of wires; or
  (2) Fifteen percent of the total number of wires within any strand;
(b) On a regular lay rope, more than one broken wire in the valley between strands in one rope lay length;
(c) A loss of more than one-third of the original diameter of the outer wires;
(d) Rope deterioration from corrosion;
(e) Distortion of the rope structure;
(f) Heat damage from any source;
(g) Diameter reduction due to wear that exceeds six percent of the baseline diameter measurement; or
(h) Loss of more than ten percent of rope strength as determined by nondestructive testing.

8 INSPECTION PROCEDURES

8.1 *Vertical lift-balance ropes*

Typically, lift and balance ropes are configured as shown in (Figure 4).

The vertical ropes on vertical lift bridges are typically very close together, sometimes almost touching as shown in Figure 5. In most cases, it is necessary to de-tension and inspect one rope at

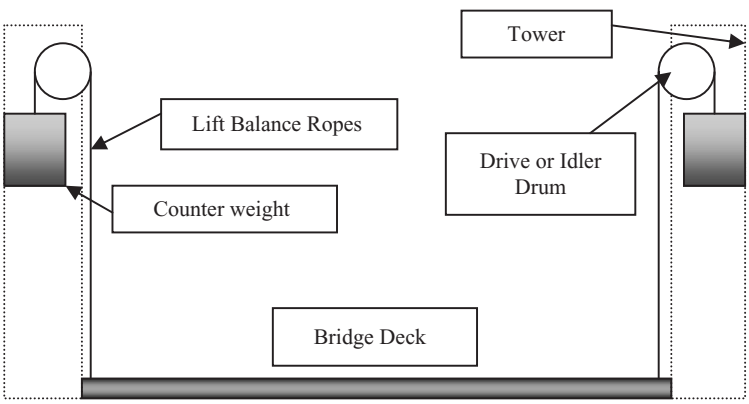

Figure 4. Typical Design for Lift and Balance.

Figure 5. Close Proximity.

Figure 6. Take up bar attachments.

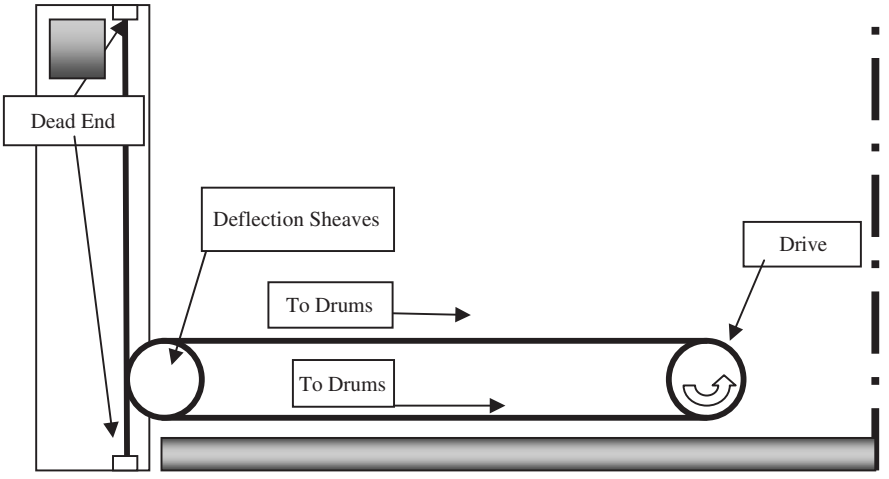

Figure 7. Typical Design Drive on Bridge.

a time. The bridge should be placed at the lower limit and the counterweight blocked in the top of the tower if possible. This is accomplished by attaching connector links in the top of the tower to the top of the counterweight. Many lift bridges are designed in this manner.

The rope is de-tensioned, the take-up rod removed from its attachment point and swung out so as to be able to place the sensor head on the rope. See Figure 6. The sensor head is then hoisted to it's the upper limit or until it touches an adjacent rope. The sensor head is then lowered then removed. The take-up rod is reattached carefully so that the tension is the same as before it was removed. The same process is carried out for all ropes on all four corners of the bridge.

Some vertical lift bridges are designed so that the ropes are far enough apart to allow them to be electromagnetically inspected. In this case, the sensor head is placed on the rope and hoisted to a point as far above the bridge deck as possible. The point is marked and the remaining ropes are inspected from the bridge deck. The counterweight and is then lowered to it's lower limit and the ropes are inspected from the counterweight to the mark made when inspecting the ropes above the bridge deck, overlapping as much as possible with the data from the bridge deck test.

The sensor head is placed in two locations on each rope to insure 100% coverage; on the bridge span at the drum and on the bridge span over or under the deflection sheave. The bridge is moved up and own for inspection of these ropes. The sensor head remains in one location during the test(s).

9 CONCLUSIONS

Nondestructive inspection of wire rope has been used successfully for almost five decades. This method can accurately measure the loss of cross sectional area in a wire rope or strand caused by wear, corrosion or other factors. Local faults such as broken wires, localized damage or pockets of corrosion can be depicted. Using these to factors, LMA and LF's and the loss of strength can be estimated to within plus or minus 3% from the original strength, when new, of the rope under test.

One training institute offers a five day class in NDT of wire rope so personnel can be adequately trained for operation of test equipment and interpretation of test data.

A new, sophisticated computer program has been developed by Acuren, and is now widely used in wire rope and strand inspection.

NDT wire rope inspection, when conducted with properly calibrated equipment, meeting the requirements of ASTM E-1571, and conducted by properly trained and qualified inspection personnel, can prevent accidents and costly downtime. NDT of wire rope can assist operators in determining when wire ropes should be removed from service based on present results. After three inspections have been conducted and data accumulated, the data can be extrapolated into the future to help determine a replacement date for the wire ropes.

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Chapter 18

Wire rope and strand assemblies in bridge applications

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ABSTRACT: The revitalization of historic landmark bridges is an important topic for discussion in many circles today. This paper compares the current and past engineering knowledge and technology used to design bridge wire rope and strand assemblies. This paper looks at strand and rope development including socketing and fabrication procedures for assemblies. Major advancements in these fields along with advanced manufacturing technologies provide higher strength materials for assemblies. Topics discussed will be the advancements in corrosion inhibitive materials available for hangar assemblies along with their advantages and disadvantages. The reader will also be informed of the importance of proper assembly fabrication including prestretching and proofloading techniques and why this must be done to ensure proper installation of the completed assembly. Socket selection and attachment style will affect the fabrication tolerances and provide simple installation and tensioning onsite.

1 INTRODUCTION

The use of carbon steel wire in the fabrication of long span bridges has been practiced for roughly 150 years. The basic concepts and ideas incorporated into those structures are still practiced today in the construction of modern bridges. The same holds true to the structural wire rope and structural strand used today. These materials are distant cousins to the materials first incorporated in the developing bridge world. While combining higher strengths, better abrasion, and greater resistance to corrosion one item that has remained a constant is the use of carbon steel in the wire making process. The requirements of the steel industry today take full advantage of the available technologies while pushing the limits of the current manufacturing capabilities.

2 WIRE PRODUCTION

The building block of structural bridge rope and bridge strand is galvanized wire. The current (ASTM A603 & A586) specifications for these items mandate the physical requirements for the wires. The requirements include the mechanical properties, stress under load, elongation, tensile strength, ductility, zinc coating weight, coat adherence, and wire surface finish. Typically to improve on one property another must be sacrificed.

The process of producing galvanized wires has seen several improvements and refinements over the past ten years leading to increased strengths and improvements in quality. Some of the processes once thought to be necessary have been removed or replaced to reduce cost and stream line the manufacturing. The manufacturing begins with the rod production. Carbon is the major steel element in the rod used for wire production. Higher carbon elements directly correlate to higher tensile strengths as well as the resistance to abrasion. The negative side of this relates to the elongation and ductility of the wire. The rod production begins with continuous cast blooms rolled to diameter and control cooled to provide a clean homogenous pearlitic grain structure suitable for direct drawing. The controlled cooling eliminates the need to patent the rod prior to drawing the wire. The drawing process

is next. Drawing is the process to reduce the diameter of the rod by passing through a number of successive dies. The drawing process aligns the grain structures into a fiberous geometry substantially increasing the tensile strength of the wire. This process requires clean steel without inclusions to ensure cupping and ductility to not become a factor. Since the drawing process is a cold working process the amount that can be performed is limited by the steel. Tensile strength increases dramatically but the ductility of the wire becomes very poor. With the quality of the steel available some rod maybe reduced up to 95% of its original area. However in some cases the wire must be patented. During patenting the steel grain structure becomes more uniform and ductile making the wire suited for further drawing. Several advancements in the patenting of wire have been introduced and incorporated in to the process. The use of lead to quench the steel is no longer a viable solution in the United States due to stringent EPA regulations. Out these regulations have come new methods using salt, sand, and steam as viable quenching methods. These methods provide superior refinement of the wire grain structures without the harmful environmental effects that are associated with the use of lead.

The last step in the wire production process is galvanizing. The majority of the wire supplied for bridge strand and structural bridge rope are galvanized by the hot dip method. Meaning the wires are submerged in molten zinc to provide the coating. The main drawback to this type of coating method is strength loss due to the annealing of the wire while submerged in the molten zinc. Approximately 10% of the wire strength will be lost during the hot-dipped coating process. Drawing the wires after coating will regain most of the strength due to the cold-working effect. It is possible to produce wires in the 340–350 ksi range by drawing after galvanizing. The other means of coating the wires is electroplating the zinc to the surfaces. This provides a clean coated surface that is acceptable for use in static and dynamic applications.

3 STRAND AND ROPE PRODUCTION

The stranding and closing portions of the cable manufacturing process becomes one of the most critical steps. The ASTM specifications allow each manufacturer the latitude to determine their design as long as the mechanical properties are met as specified. Several of the steps in these processes can help the cable perform better in fatigue application as well as greatly affect the tensile strength.

4 ASTM A586 STRUCTURAL STRAND

The ASTM title for this type of strand is *Zinc Coated Parallel and Helical Steel Wire Structural Strand*. This specification also covers *Zinc Wires for Spun-In-Place Strand*. The process of manufacturing structural strand consists of laying consecutive layers of wires around a center, changing the direction of the lay of each layer. The opposing lay direction increases tensile strength and Modulus of Elasticity while providing a balanced rope. The wires used in each layer may or may not be preformed. The performing of wires consists of mechanically forming the wire. This ensures the wires form in a tight layer around the core or base layer. Some strand constructions will have multiple wires laid to form a unitized center, sometimes referred to as the Parallel Contact Core, to increase the static fatigue of the cable. Tension-tension fatigue testing has shown this construction reduces internal nicking and bending stresses which occur in conventional cross lay designs. This testing shows cracks to initiate at the cross-wire contact point of inner wires, eventually leading to tear in shear. Increasing the number of wires in the strand, smaller diameter wires, has also been shown to increase the fatigue life of structural strand. This can be related to less contact area and ductility of the wires. However it has also been shown that wire grade and mean loading have little or no effect on the fatigue life of the strand. Structural strand is designed to be used in straight tension only. Deflections in structural strand could reduce the strength and severely reduce the fatigue strength of the strand if not tensioned properly.

5 ASTM A603 STRUCTURAL WIRE ROPE

The ASTM title is *Zinc Coated Steel Structural Wire Rope*. This is also commonly referred to as Structural Bridge Rope. Due to the mechanical requirements of the specification structural wire rope is made differently from traditional six or eight stranded running ropes. The individual outer strand lay and rope lays are lengthened to provide increased strength and Modulus of Elasticity. Since they are being utilized in static tension there is a core option for these style ropes. A single strand core or independent wire rope core can be provided without a reduction in the mechanical properties of the rope. The strand core provides a simpler more traditional style rope which on average will have an increased Modulus of Elasticity. This also allows the producer the option of varying the manufacturing process.

6 CORROSION RESISTANCE

By looking around us today at the ageing structures everyone is fully aware of the need to protect the steel from the harsh environment. Countless studies have been conducted to measure the corrosion resistance ability of zinc coated steel. In the (ASTM A586 & A603) specifications there are three levels of zinc coating available for the wire. These levels consist of Classes A, B, and C which increase the amount of zinc weight per foot, Class C being the heaviest. These coatings may be applied by the hot-dip method or electroplating process.

Aside from the zinc coating applied to the wire surface there are numerous additional types of coatings that may be applied to the rope or strand to increase the corrosion resistance of the assembly. Studies have been performed to find alternatives to painting or coating the cables onsite after installation. Greases, pastes, and preservatives applied internally to the cable during the manufacturing process work to protect the wires from corrosion during the life of the cable. Testing done by the State of California Department of Transportation compared wires coated with Gri-Kote Z-Complex 2C, manufactured by Grignard Company LLC, bare wires, and various other coatings after 6 months of accelerated corrosion testing in an (ASTM B117) salt fog chamber (Reis 2004). The obvious signs of severe corrosion are present in Figure 1 of the bare wire while oxidation of the wires in Figure 2 is the result of the wires coated with Gri-Kote Z-Complex 2C.

The coating applied to the wire for testing purposes incorporates a combination of 45% zinc dust and 45% zinc oxide in a blend of vegetable oil, with additional corrosion inhibitors. These types of corrosion inhibitors can increase the life of the cable substantially by protecting the wires as a sacrificial anode as well as repel water away from the surface. They must remain flexible during the life of the cable to adjust and deflect as the cable operates. Grease type corrosion inhibitive

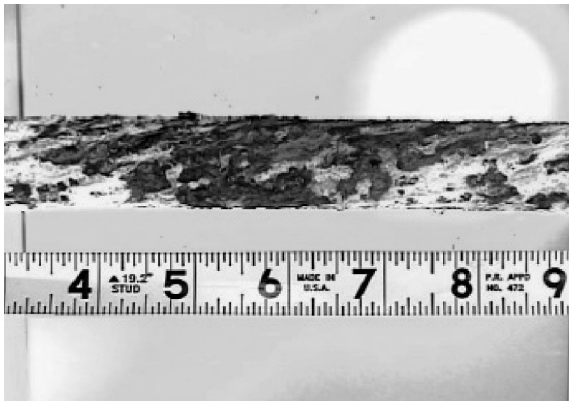

Figure 1. Corrosion of bare wire.

treatments can be used to prevent moisture from entering the cable and provide reduced friction between the wires during its operating life. The main drawback to these types of treatments is the appearance of the material on the exterior of the strand. Figure 3 shows the application of Syncan Grease preservative, supplied by the Grignard Company LLC, to galvanized structural strand during the manufacturing of the cable.

The coating or jacketing of cables as a means of providing corrosion resistance is not recommended. These types of coatings not only conceal the wires from external inspection but could act as a water trap. Although UV inhibitors are included in the coating material these become susceptible to cracking. These cracks will then work to draw in moisture which has no outlet. It is also critical that individual wires be allowed to move and adjust freely during the operating life of the cable.

7 FABRICATION OF ASSEMBLIES

Once the strand has been produced and the socket selected the assembly work begins. Depending on the application and installation procedures the final length tolerance may be become a very critical point. Several procedures need to be done to ensure the assemblies fabricated in a shop environment can be installed in a field environment without adjustment.

Prestretching the wire rope or strand should be the first step. This operation is critical to remove the constructional stretch of the cable providing length stabilization during assembly fabrication

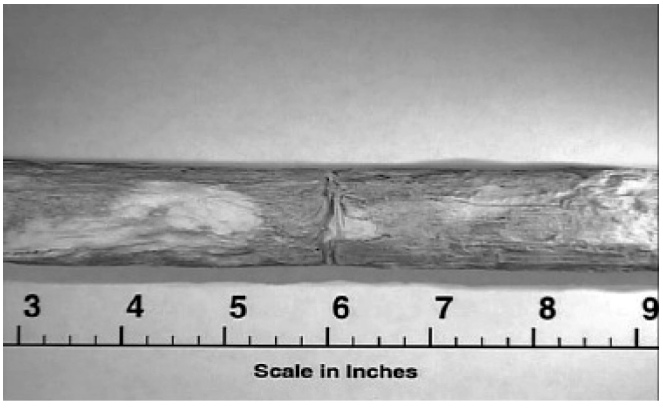

Figure 2. Corrosion of coated wire.

Figure 3. Application of preservative.

process. The ASTM specifications do not state what procedures or steps are to be used to establish this they do specify the minimum Modulus of Elasticity, E, for the cable. The ASTM A586 Structural Strand specifies the limits at 24,000 ksi while ASTM A603 Structural Wire Rope is slightly lower at 20,000 ksi. The prestretching operation is carried out by loading the cable to a specific percentage of the cables minimum breaking force and holding it for a set time period. In some cases the cable may be cycled up and down.

Directly after prestretching the cable it should be measured for assembly length. During the prestretching operation the zinc coated wires will seat onto one another slightly displacing the zinc coating, this is more prevalent with heavier coated wires. Moving or handling of the cable prior to measuring will cause disturbances which will reintroduce some of the constructional factors of the cable. Typically the assemblies are measured for cutting at the calculated dead load tension of the structure. This enables production of the assemblies to the desired tensioned field length of the cables. It is highly recommended that all measurements be performed in controlled environments under cover to ensure undesirable weather or temperature changes do not affect the assembly performance. All measuring should be performed with calibrated tapes. The standard temperature specified by the National Institute of Science and Technology is 68 degrees Fahrenheit. Should the design temperature of the bridge vary widely from this temperature it will be necessary to accommodate these length changes prior to beginning measuring operations.

The prestretching and measuring procedures lead to the final operation of socket installation. The most common type of end termination is the speltered socket. Swaged and wedge style sockets are available but efficiency losses and unfamiliarity with the products are still present. In speltering operations the brooming and cleaning of the cable ends need to be performed to ensure proper adhesion of the wires to the socketing medium. The prepared end is then suspended vertically. This is required to minimize the curvature of the strand and ensure equalization of the stresses in the wires. The larger the diameter of the cable the higher the cable must be suspended to ensure the cable lie true during the socketing operation. Figure 4 shows a multi leveled socketing tower for speltering large diameter structural strand.

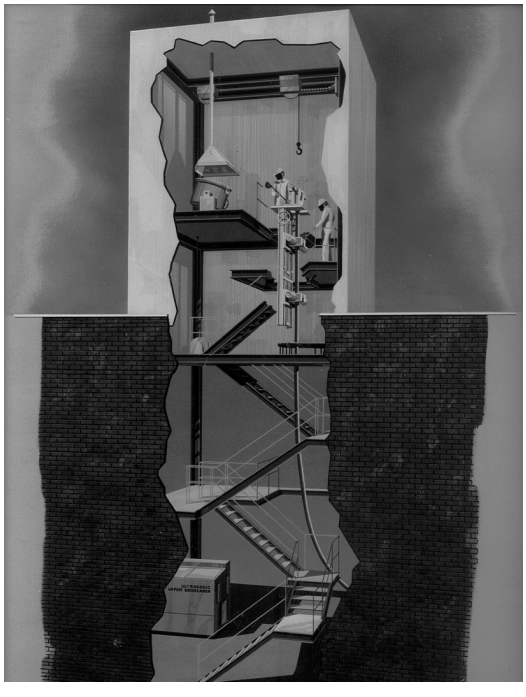

Figure 4. Socketing tower.

The broomed end is then closed and inserted into the socket splaying the wires evenly to allow flow of the socketing medium around each wire. The axial alignment of the socket with the strand should be checked. In addition, the concentricity of the strand within the socket should be checked. Proper socket alignment assures maximum service life for any socketed assembly, by distributing stresses to all wires in the strand in equal proportion. The socketing medium is then added to the socket. The two common types of medium are resin and zinc. With each having positives and negatives to the installation and long term assembly the zinc medium is the most common found today. Several other mediums including Zinc-Aluminum and Zinc-Copper are becoming present on the market. The final operation of the socket attachment process is proofloading the socket. The proofloading of the socket serves to test the pouring operation of the assembly manufacturer. It also serves to seat the socket. Spelter sockets work on a wedge action to develop the strength of the socket. The wedging seats the cone in the socket causing some elongation of the assembly. This elongation should be accounted for in the layout of the assembly.

The previous paragraphs have only spoken of spelter sockets. It should be pointed out to the reader that a swaged style socket may also be applied to the cable types spoken of in this paper. Swaged sockets are mechanically pressed on to the cable. However since these are forged pieces the size and design of the socket may be limited to smaller diameter cables.

8 BRIDGE ROPE FATIGUE

Axial fatigue testing of Galvanized Structural Bridge Rope is a time consuming process. The fatigue data acquired from these types of tests is crucial in determining periodic maintenance and load effects on suspenders. Open strand sockets were speltered onto all of the samples with Prime Western grade zinc. The cyclical frequency of the loading was approximately 100 cycles per minute. The tests were performed on four 2-3/4" diameter 6×25FW IWRC galvanized bridge ropes. The rope was produced in accordance with the (ASTM A603) specification.

Samples 1 and 2 were cycled from 90 to 170 kips during the tests. Sample #1 performed the best of all samples. Table 1 shows the results of all samples tested. It has been known for some time that the failure of wire rope subjected to axial load is fretting. This is a form of fatigue in which the damage is accelerated by the rubbing of surfaces. To reduce the fretting action pure boiled linseed oil was applied directly to the rope during cyclic loading to simulate the lubricity which would be provided by the protective paints applied in a preventative maintenance program. This also occurs naturally due to the oxidation of the zinc coating. Sample #2 was painted will linseed oil only at the beginning of the testing. This sample approached the 40 broken wire at approximately half the cycles of sample #1. Samples 3 and 4 were cycled from 102 to 193 kips. Sample #3 was not painted and was the only sample allowed to be cycled to complete failure. Sample #4 was cycled to 100,000 cycles and removed for destructive testing. Sample #4 broke at approximately 4% over the nominal strength. Open strand sockets were speltered onto the samples with Prime Western grade zinc. The cyclical frequency of the loading was approximately 100 cycles per minute.

The result of the testing was as expected. The number of wire breaks increases as the cycles increase. There is no retirement criteria specified for wire rope and structural strand components. For the tests an initial value of “six broken wires in one rope lay” was established to determine the fatigue failure of the rope. The testing soon pointed out that this number was not a realistic value. Therefore the fatigue failure criterion was increased to approximately “40 broken wires in one rope lay” to establish the actual life of the cable. With the exception of one sample, all were able to carry the loading despite the large number of wire breaks. Due to the nature of fretting fatigue wire breaks tend to raise up and out of the rope, “porcupine”. Due to the increased contact stresses in the wire at the raised point, failure will occur rapidly causing the wire to complete break from the rope and fall out of the sample. However the loss of the wire does not reduce the strength of the sample for two reasons. Wire ropes are composite structures that are redundant at any section. The second is due to frictional forces. A broken wire will regain load carrying ability several rope diameters away from the failure. Figure 5 shows fretting on the suspenders of the Manhattan Bridge in New York City.

Table 1. Wire rope fatigue results.

Sample #1		Sample #2		Sample #3		Sample #4	
# Broken wire	# of cycles	# Broken wire	# of cycles	# Broken wire	# of cycles	# Broken wire	# of cycles
1	1309900	4	1016800	64	456931	20	466000
4	1869400	10	1168860	86	483700		
17	2142900	19	1323400	160	587000		
28	2688500	37	1452900	260	859000		
42	3193263			Failure	1670000		
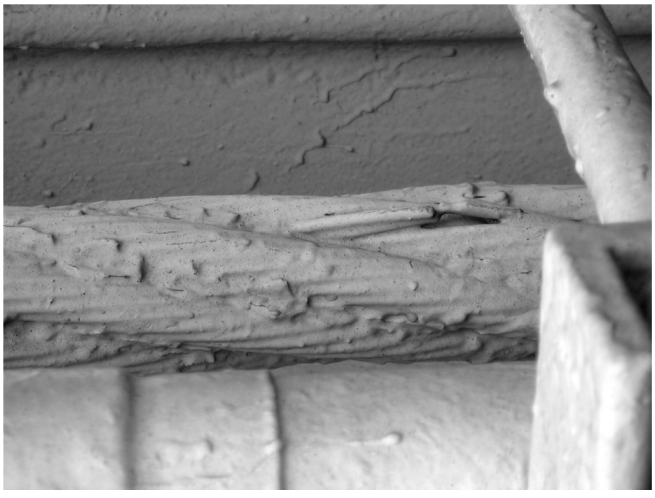
Figure 5. Fretting of suspender ropes.

Because of the lubrication effect, the fatigue life seen of the standard un-lubricated samples as received from a rope mill should not be compared directly with that of samples 1 and 2.

9 CONCLUSION

This paper was written with the intent of educating the bridge community on the advancements and process used to properly manufacture cable assemblies. The amount of time and effort exerted by the steel community over the past 10 years to advance the mechanical properties of galvanized steel wire used in all types of cables has been tremendous. The processes used today to produce these items domestically are very restrictive but with the introduction of fine grain steel rod and advanced manufacturing processes a higher strength galvanized wire is being produced. The current ASTM specification covering these types of cables allows the manufacture the latitude to produce cables without restrictive manufacturing parameters. Innovative materials are now available to increase the corrosion resistance of the cable. Some of the materials are applied during the manufacturing of the strand to individually coat wires and some are applied to the cable once it is installed and tensioned onsite. The fatigue tests presented in this paper show that standard preventative maintenance on bridge cables can significantly extend the life of cable. Variations in the construction of the cable have also been shown to increase the life cycle of the cable.

The amount of information provided in this paper only begins to describe some of the ideas and trends of the wire rope and strand industry. The ultimate goal is higher strength materials that provide safe and efficient structural members for bridges.

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Chapter 19

Epoxy coated and filled prestressing steel strand cables

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ABSTRACT: Corrosion protection for prestressing steel materials used for prestressed and post-tensioned structures is one of the most important factors needed to increase the structure service life. The corrosion protection is usually applied at the construction site by cement grouting or other means, however, it has been discussed for several years that it is sometimes difficult to control the protection quality due to operator's skill variation, grout quality, weather or other factors. On the other hand, several prestressing materials with factory applied corrosion protection have been used in various applications, such as prestressing, post-tensioning, stay cables and ground anchors. In general, this type of prestressing material has several corrosion protection layers around the steel and has the following advantages.

– Highly reliable corrosion protection due to quality-controlled manufacturing processes. The quality can be also maintained after production in the plant, during delivery, during construction and during the structure service life.
– Labor cost saving and rapid construction by eliminating corrosion protection work on site.
– Easier inspection and replacement of cables in the future.

Epoxy coated and filled prestressing steel strand (ECS) is one of the most common materials produced with a factory-applied corrosion protection. This material is perfectly protected by epoxy coating on both the outside and inside of the steel strand. It has been applied to various PC bridges since 1983 such as cable-stayed bridges, extra-dosed bridges and box girder bridges. This paper will introduce several evaluations conducted using ECS in order to verify that ECS performs sufficiently enough to maintain long service life of PC bridges.

1 INTRODUCTION

1.1 *Background*

Recent requirements are for Prestressed Concrete (PC) structures to perform with high durability for more than a 100-year service life with minimum maintenance cost. Several investigations on deterioration of PC Bridges have indicated that serious damages of prestressing steel cables (tendons) inside of bridges are one of the main reasons for the deterioration. According to an investigation conducted in Japan, PC bridge deteriorations due to steel tendon damage have been detected in 46 out of 84 bridges (55%). In addition, corrosion or breaking of tendons due to chloride attack has been detected at 22 out of 84 bridges (26%) and corrosion or breaking of tendons due to improper cement grouting have been detected in 16 out of 84 bridges (19%). Based on this investigation, it is obvious that the important key for long term service of PC bridges is to protect prestressing tendons from corrosion or breaking due to chloride attack or insufficient grouting. This is the main reason why factory applied corrosion protection has been considered as an important option in order to minimize the bridge deterioration problems.

1.2 *General design concepts for ECS*

In general, prestressing steel tendons with factory applied corrosion protection have a corrosion protection layer made of metallic coating, organic coating, plastic sheath or corrosion inhibitor on

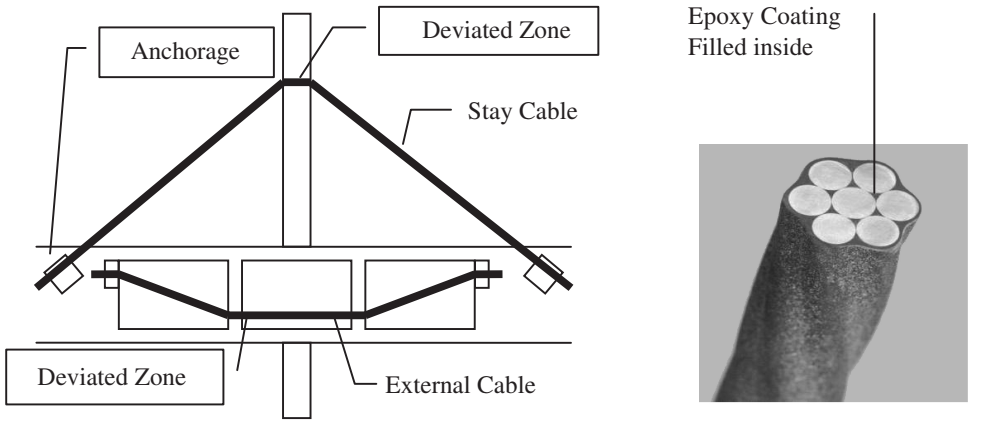


Figure 1. Applications of ECS cable and cross section view of ECS.

the steel surface. When those steel tendons with the protection layer are used at PC bridges as stay cables or external span cables where the highest degree of corrosion protection is usually required, the following requirements are always critical for the protection layer to satisfy, Figure 1:

- Corrosion resistance against water, chloride attack generated by salty environments and other possible chemicals attacks.
- Durability at deviated zone inside of pylons or box girders where high compressive force operates on the protection layer.
- Anti-fatigue performance of anchorage without any additional work to remove the protection layer.

ECS is designed based on the following concepts in order to satisfy the requirements described above.

- Thick epoxy coating layer. ASTM A882 specifies the coating thickness between 0.38 through 1.14 mm (ASTM A882, 2005). In addition to the coating layer outside, the inside of the strand is also filled with the same epoxy coating as shown in Figure 1 to prevent any corrosive atmosphere that could go into the inside of strand from the strand ends.
- Sufficient adhesion between epoxy coating and the steel surface is also required. The adhesion prevents the coating from peeling off the strand due to any mechanical damage. In addition, the adhesive coating prevents relative movement between individual component wires of strand, which can reduce fretting between the component wires and improve the fatigue performance.
- Epoxy coating with enough strength and ductility to maintain resistance against any mechanical damage during construction and any compressive force on the coating layer once the structure is placed in service.

This report introduces several evaluations conducted using 15.2 mm diameter 270 k grade ECS specified in ASTM A882 in order to prove that ECS can satisfy the required corrosion resistance, durability at deviated zone and anti-fatigue performance of anchorage.

2 CORROSION RESISTANCE

2.1 *Corrosion test without stressing*

Several corrosion tests using water, several types of chemical solution ($CaCl_2$, NaOH, $Ca(OH)_2$) and salt spray were conducted to evaluate corrosion resistance of ECS without stressing compared

Table 1. Several corrosion test results using ECS without stressing.

Test conditions		Test solution	Test results		
			ECS	Bare strand	Galvanized strand
Water	ASTM G 20-77	Distilled water	No visible corrosion	Rust	Rust (White)
	Temperature: 20°C	3 mol $CaCl_2$	No visible corrosion	Rust	Rust (White)
Chemicals	Term :1,090 Hr (45 days)	3 mol NaOH	No visible corrosion	No corrosion	Rust (Dissolution)
		Saturated $Ca(OH)_2$	No visible corrosion	Discoloration	Rust
Salt spray	ASTM B117-73 Temperature: 50°C Term: 1,000 Hr	Salt spray	No visible corrosion	Rust	Rust

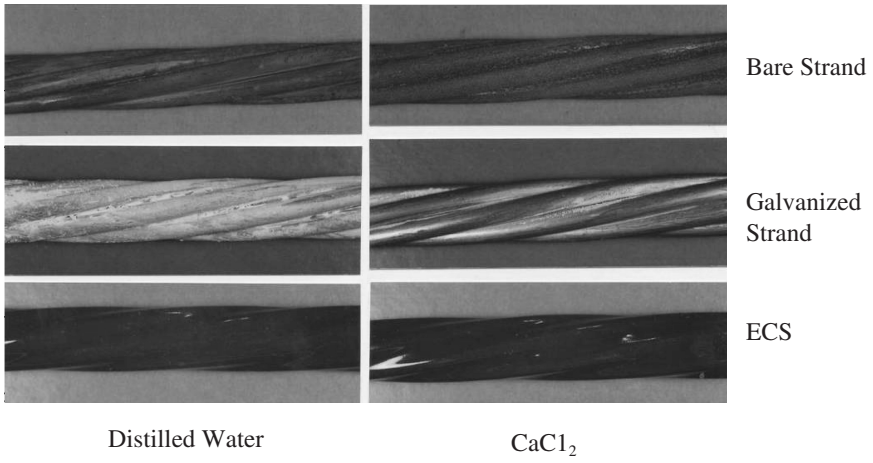


Figure 2. Corrosion test results using distilled water and $CaCl_2$ solution without stressing.

with non-coated strand (bare strand) and Galvanized strand (Zinc coated strand). All of the test results are summarized at Table 1 and all the samples after the tests are shown in Figure 2 through 4. No visible sign of corrosion was observed on any of the specimens of ECS. The same type of corrosion tests were also conducted for ECS with a man-made pinhole at the center of the sample in order to verify that epoxy coating adheres to the steel surface tightly enough to prevent corrosion generated at the man-made pinhole from developing around the pinhole. All the samples except the sample with NaOH had corrosion at the pinhole after the tests however no visible corrosion development around the pinhole was observed as shown in Figure 5.

2.2 *Corrosion test under stressing*

A salt spray test was conducted for ECS under stress with 70% of guaranteed ultimate tensile strength (GUTS). To apply the tension on the sample, it was set in a steel frame and anchored using chucks at the both ends after stressing. Both of the anchorage areas were completely protected from the salt spray attack during the test as shown in Figure 6. No corrosion was observed after the 3000 hour salt spray test as shown in Figure 6.

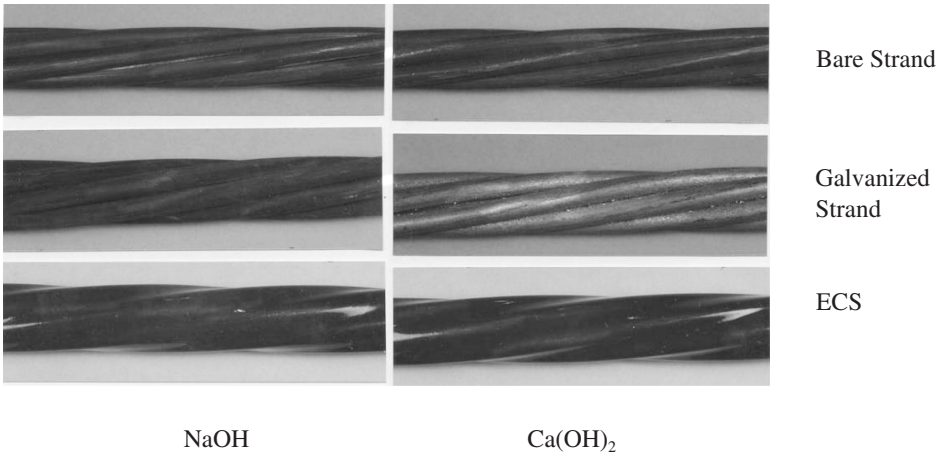


Figure 3. Corrosion test results using NaOH and $Ca(OH)_2$ solution without stressing.

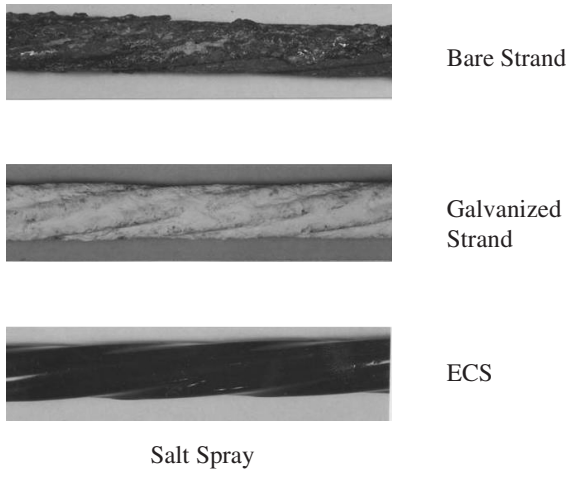


Figure 4. Corrosion test results using salt spray without stressing.

2.3 *Composite cycle corrosion test*

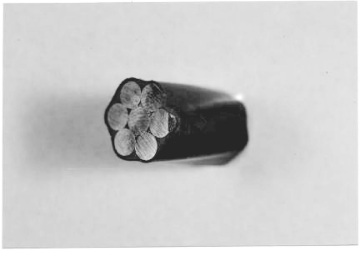
Considering actual practice during construction and the environment around a PC Bridge in service, a composite cycle test (JHS403-1992) was conducted for several types of ECS samples.

Test samples:

- Sample A: ECS under 70% of GUTS tension.
- Sample B: ECS with a man-made pinhole at the center of the sample. Both ends of the sample were sealed by touch-up paint.
- Sample C: Same condition as for B with the exception that the manmade pinhole is repaired by touch-up paint.
- Sample D: Cut sample of ECS without any treatments.
- Sample E: Epoxy coated steel plate, which is coated by electrostatic coating method after brushing the plate surface.

ストランドにφ6mm の孔を設けて腐食試験したところ(CaCℓ2溶液)

View of pinhole area on ECS after a corrosion test using $CaCl_2$ solution

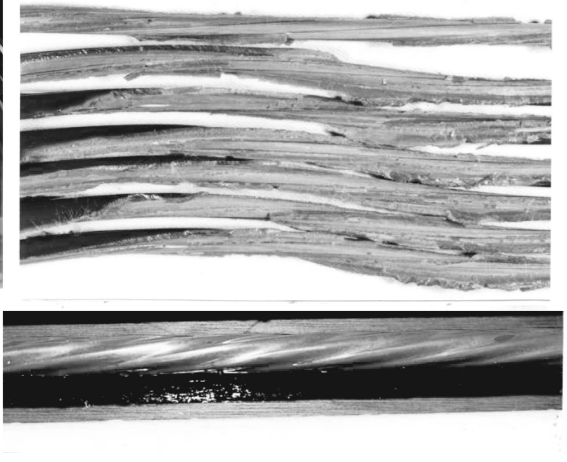


ストランド孔の部分をストランド断面側から観察したところ

Cross section view of pinhole area after a corrosion test using $CaCl_2$ solution

Figure 5. Corrosion tests results using ECS with a man-made pinhole.

Overview of 3,000 hour salt spray test under stressing



<Top> Inside of ECS after 3,000 hour salt spray test
<Bottom> ECS after 3,000 hour salt spray test

Figure 6. Salt spray test using ECS under stress with 70% of GUTS.

2.4 *Corrosion test under stressing*

A salt spray test was conducted for ECS under stress with 70% of guaranteed ultimate tensile strength (GUTS). To apply the tension on the sample, it was set in a steel frame and anchored using chucks at the both ends after stressing. Both of the anchorage areas were completely protected from the salt spray attack during the test as shown in Figure 6. No corrosion was observed after the 3000 hour salt spray test as shown Figure 6.

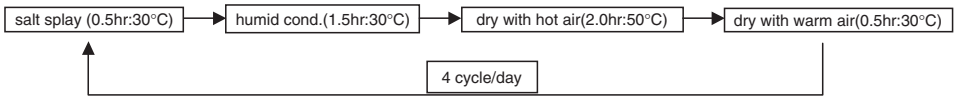


Figure 7. Corrosion cycle procedure of the composite cycle test in JHS403-1992.

Test condition

This composite cycle test was conducted in accordance with JHS403-1992 specified by a Japanese Highway Public Corporation. Test pieces were irradiated in the sunshine-carbon-ark-light test machine for 96 hours prior to the cycle test. After that, the corrosion cycle procedure shown in Figure 7 started. The corrosion test was conducted for sample B without the prior irradiation due to the limitation of the equipment size.

Test term

360 cycles for 90 days, which is specified by JHS403-1992, were applied for sample E. 720 cycles for 180 days were applied for the other samples (A, B, C, D).

Test results

- Sample A: No sign of corrosion was observed.
- Sample B: Corrosion was observed at the man-made pinhole. However, the corrosion did not develop around the pinhole.
- Sample C: No sign of corrosion was observed on both regular coating area and repaired area by touch-up paint.
- Sample D: No sign of corrosion was observed except both ends of the sample where no coating exists. It was also observed that no corrosion was developing from the unprotected ends.
- Sample E: No sign of corrosion was observed except the scratched lines where no coating exists. The corrosion at the scratch lines did not develop to the coated area.

Figure 8 shows all of the samples after testing. In the overall, this observations satisfied the JHS403-1992 requirements where no corrosion, no coating cracks, no softening of the coating and no peeled coating from the steel surface should be observed after the cycle test.

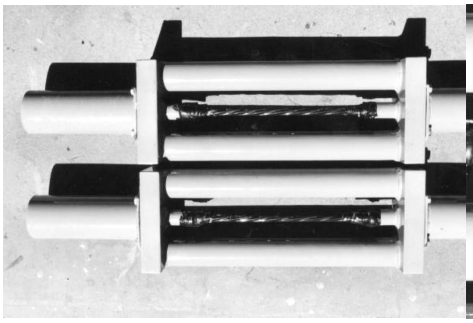
3 DURABILITY AT DEVIATED ZONE

3.1 *Full scale fatigue test at deviated zone*

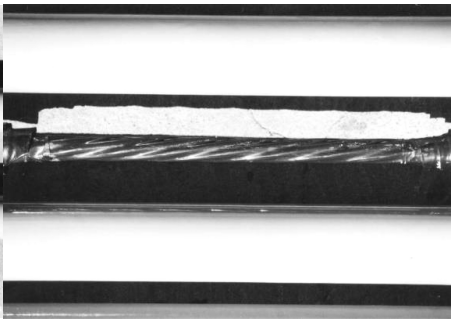
A 3 million cycle full scale fatigue test was conducted for a tendon with 19 cables of 15.2 mm diameter ECS at two deviated zones as shown in Figures 9 and 10. The test conditions are shown as follows.

- Deviator angle: 14 degree
- Radius of deviated zone: r = 3,000 mm
- Grouting: No grout on the entire length of the cable including the deviated zone and anchorages
- Tendon assembly: Twist
- Lower limit load: 2,957.4 kN (60% of GUTS)
- Stress range: 129.2 kN
- Frequency: 0.7 Hz
- Number of cycles: 3 million
- After the fatigue test, each strand at the deviated zone was tensile tested.

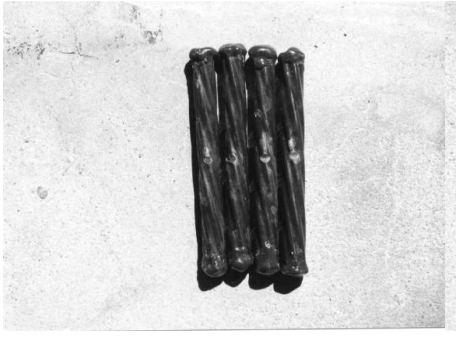
The 3-million cycle fatigue test was completed without any wire failures. Observing the ECS which was located at the deviated zone during the test, it was verified that the epoxy coating remained sufficiently enough to keep the required corrosion resistance as shown in Figure 11.



Sample A
- Under 70% of GUTS tension
- No sign of corrosion

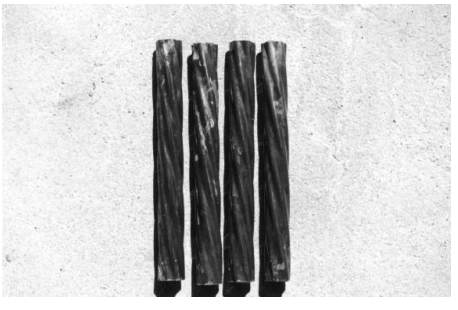


Magnified view of Sample A



Sample B
- With a manmade pinhole
- No corrosion development around the pinhole

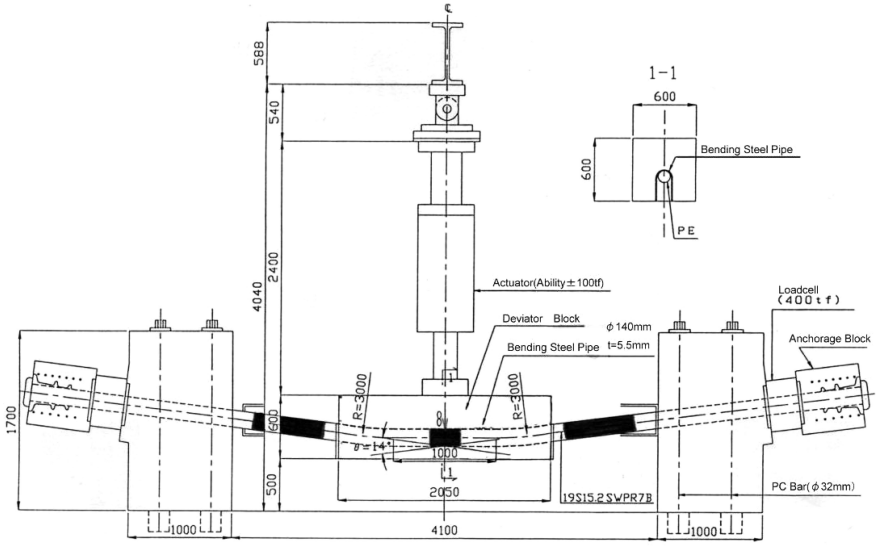
Sample C
- With a manmade pinhole repaired by touch-up paint
- No sign of corrosion



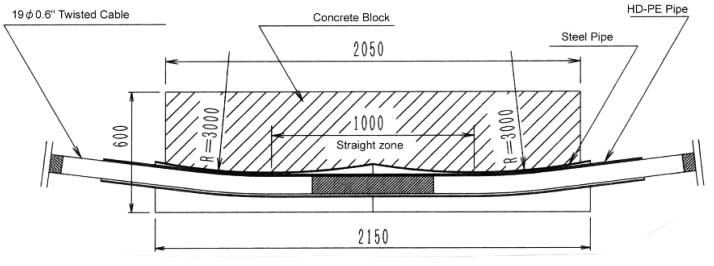
Sample D
- No protection of the cut ends
- No corrosion development from the cut ends

Sample E
- Epoxy coated steel plates with scratched lines
- No corrosion development from the cut ends

Figure 8. Test result of the composite cycle tests in JHS403-1992.



Schematic of the full scale fatigue testing equipment at deviated zone



Magnified view of the deviated zone

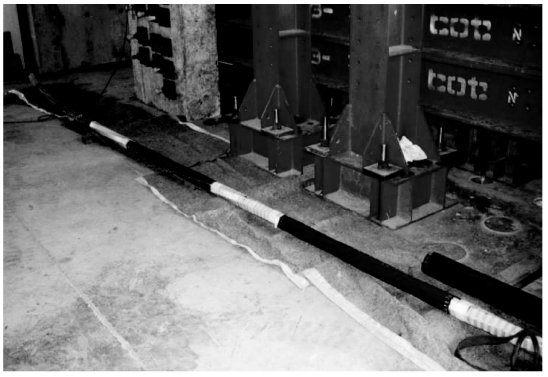
Figure 9. Schematic of the full scale testing equipment at deviated zone.

3.2 *Full scale stressing test at deviated zone*

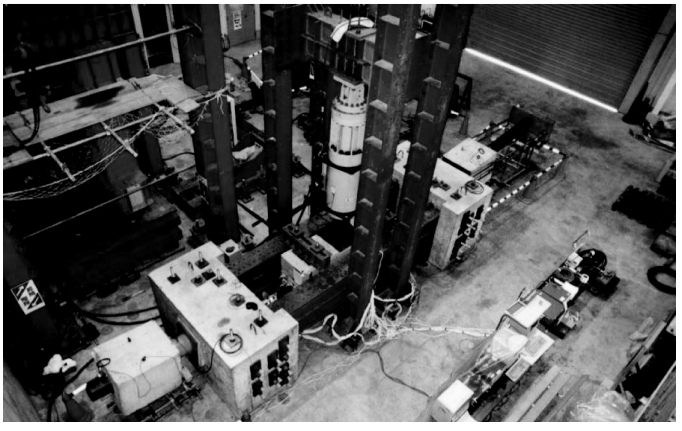
A stressing test was conducted for each of two tendons with 19 cables of 15.2 mm diameter ECS at the deviated zone to evaluate the durability of ECS under lateral pressure which operates on the tendon during stressing. This test was conducted for both a parallel cable tendon and twisted cable tendon because the force direction to the ECS in each of the tendon is different. The test conditions are shown as follows.

- Radius of deviated zone: r = 3,000 mm
- Deviated pipe: A polyethylene pipe installed inside of bent steel pipe which is embedded in a concrete block.
- Stressing load: 3789.9 kN (90% of nominal yield point).

After installing the tendon into the deviated pipe and stressing it up to the required load, the tendon was moved 200 mm as shown in Figure 12 keeping the initial tension. After the procedure was completed, the tendon was detensioned and disassembled to observe the epoxy coating located at the points in Figure 12 where high compressive pressure operated onto the epoxy coating.



ECS cable for testing



Overview of the full scale fatigue testing equipment at deviated zone

Figure 10. Overview of the full scale testing equipment at deviated zone.

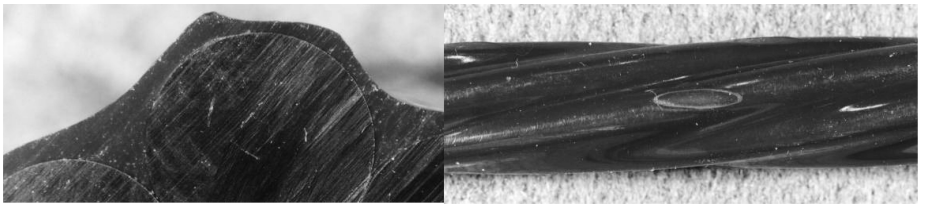
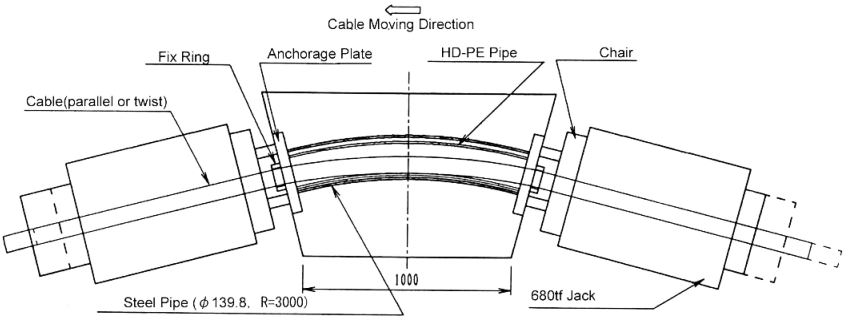


Figure 11. Epoxy coating layer remaining after the 3-million fatigue test at deviated zone.

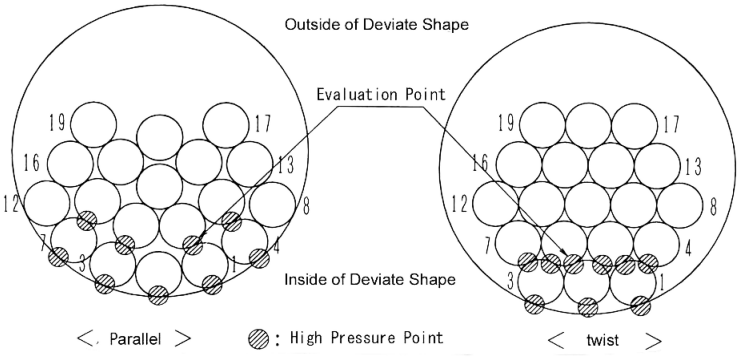
As a result of the observation of epoxy coating at the high pressure points in both the parallel cable tendon and twisted cable tendon, it was verified that the epoxy coating remained sufficiently enough to keep the required corrosion resistance as shown in Figure 13.

3.3 *Creep test using ECS under compressive load*

A creep test with compressive load was conducted for epoxy coating on 15.2 mm diameter ECS in order to evaluate long term durability of the epoxy coating against compressive pressure which always operates on ECS cables in a tendon located at a deviated zone. The general test procedure is as follows.



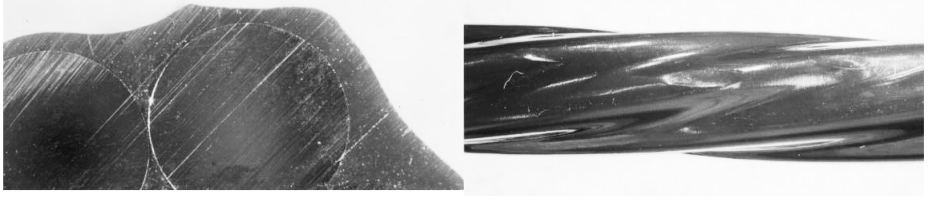
Schematic of the full scale stressing test at deviated zone



High pressure points where high compressive pressure operates onto the ECS during the test

Figure 12. Schematic of the full scale stressing test at deviated zone.

Epoxy coating layer remaining from the parallel cable



Epoxy coating layer remaining from the twisted cable

Figure 13. Epoxy coating layer remaining after the full scale stressing test.

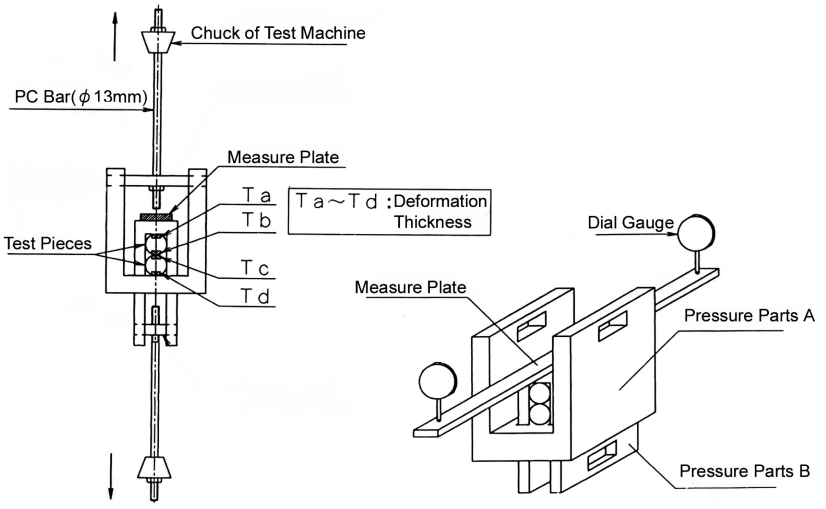


Figure 14. Schematic of ECS creep testing equipment under compressive load.

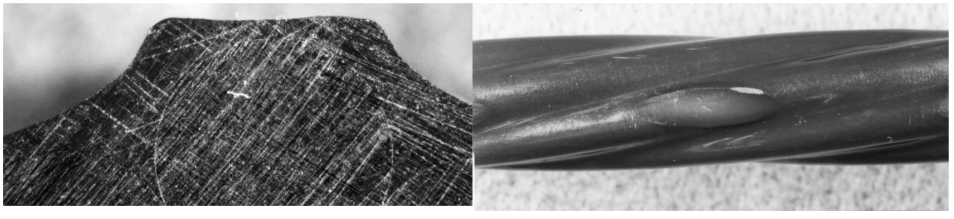


Figure 15. Epoxy coating layer remaining after the creep test.

– Set two pieces of ECS at the specific creep testing equipment shown in Figure 14.
– Start loading up to 3.4 ton force/100 mm that is calculated as the maximum compressive pressure for 15.2 mm diameter ECS in 19 cable tendon located at a deviator zone.
– Measure displacement of the epoxy coating by reading the dial gauge keeping the initial load constant.
– Read the dial gauge every 5 seconds during the first 8 minutes, every 30 minutes during the next 30 hours, every 2 hours during the next 69 hours and every 4 hours at the last 216 hours.

After the test, it was verified that the epoxy coating remained sufficiently enough to keep the required corrosion resistance as shown in Figure 15. In addition, the creep curve at Figure 16 indicates that the reduction of coating thickness seems to be saturated at the last stage of this test. Based on these results, it can be considered that the coating can maintain the required thickness after 100 years, which is the required service life for current PC bridges.

4 ANTI-FATIGUE PERFORMANCE OF ANCHORAGE

Specific anchorages for ECS have been developed using bite through wedges which teeth can penetrate epoxy coating layer and reach the steel surface inside of the coating. Using this bite through wedge, ECS can be anchored without removing the coating at the gripping area. Figure 17 shows 2 million cycle fatigue tests using single ECS and bite through wedges.

This result shows that no wire breaks occurred at less than 250 MPa of stress range which PTI "Recommendation for Stay Cable Design, Testing and Installation" specifies the minimum

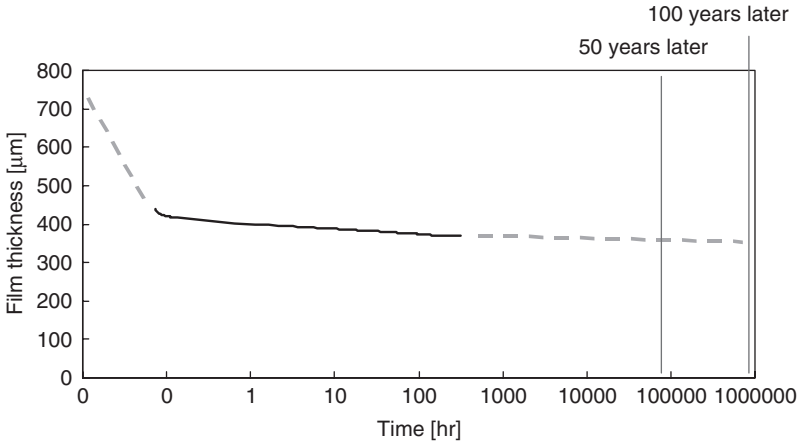


Figure 16. Creep curve of ECS creep test under compressive load.

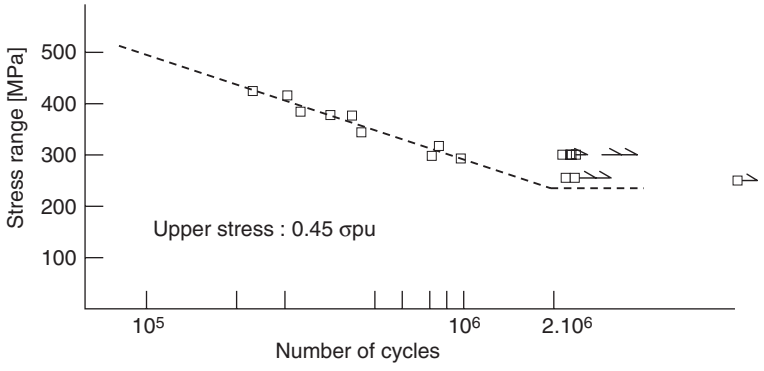


Figure 17. Fatigue performance of wedge anchored single ECS.

requirement as 159 MPa at the same initial load as for this test (PTI 2001). Epoxy coating prevents relative movement between each of the individual component wires and prevents any fretting between the wires. It can be considered that this advantage results in higher fatigue resistance of ECS than regular bare strand. In addition to the single strand tests, several fatigue tests with multiple ECS cable tendon have been conducted as shown in Table 2. No ECS failures were detected during the fatigue tests on the table.

5 CONCLUSION

Based on the all test results introduced in this paper, it can be considered that ECS is capable of satisfying the corrosion resistance, durability at a deviator zone located inside of pylons or box girders and anti-fatigue performance of anchorage without any additional work to remove the protection layer, which indicates that ECS is one of the prestressing materials that can contribute to increase the service life of PC bridges.

In fact, ECS has been used for more than 140 bridges in the world since 1983. Figure 18 shows the construction site of the Waldo-Hancock Bridge in Maine, USA where ECS has been used for the stay cables. In addition, Figure 19 shows ECS used for cables at a box girder bridge in Japan.

Table 2. Fatigue test results using multiple ECS cable tendon.

Number	Number/size of component strand	Grouting	Stress range (MPa)	Maximum angle of deviation	Test results
1	4S 15.2 mm	None	100.0	7.65″	No wire failure after 2 million cycles
2	2S 15.2 mm	None	50.0	2.5″	No wire failure after 2 million cycles
3	9S 15.2 mm	None	160.0	3.8″	No wire failure after 2 million cycles
4	9S 15.2 mm	Done	160.0	3.8″	No wire failure after 2 million cycles
5	9S 15.2 mm	Done	200.0	3.8″	No wire failure after 2 million cycles
6	12S 15.2 mm	Done	100.0	3.6″	No wire failure after 2 million cycles
7	19S 15.2 mm	Done	67.0	3.0″	No wire failure after 2 million cycles
8-1	19S 15.2 mm	None	50.0	14.0″	No wire failure after 2 million cycles
8-2	19S 15.2 mm	None	50.0	14.0″	No wire failure after 2 million cycles
9	48S 15.2 mm	None	80.0	2.5″	No wire failure after 2 million cycles

Figure 18. ECS stay cables used for Waldo-Hancock Bridge in Maine, USA.

Figure 19. ECS cables used for a box girder bridge in Japan.

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