Session C1

General Repair of Bridges II
Aspects généraux de la réparation des ponts II
Allgemeine Fragen der Instandsetzung im Brückenbau II

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Cable Anchorage Repairs on New York City Suspension Bridges

Réparation de l'ancrage des câbles des ponts suspendus à New York

Reparaturen an Kabelverankerungen New Yorker Hängebrücken

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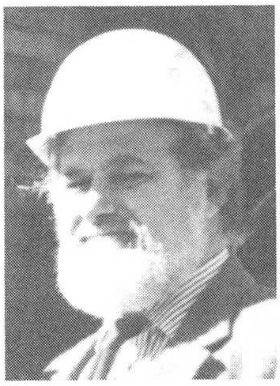

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SUMMARY

Severe corrosion in the eyebars anchoring the main cables of the Manhattan and Bronx-Whitestone bridges led to concern about the reduced level of safety. Innovative steps to strengthen these members by cutting and re-anchoring cable strands or by prestressing are presented. Less extensive repairs to cable strands on the Triborough Bridge and a method of holding strands on the George Washington bridge during repairs are also described.

RÉSUMÉ

La corrosion sévère des ancrages des câbles principaux des ponts de Manhattan et de Bronx-Whitestone suscitait l'inquiétude à cause de la réduction de la sécurité. Des méthodes nouvelles pour renforcer ces éléments, en coupant et ancrant de nouveau les torons des câbles, ou en utilisant la précontrainte, sont présentés. Des réparations moins laborieuses des torons des câbles du pont de Triborough et une méthode pour immobiliser les torons du pont de George Washington durant les réparations sont aussi décrites.

ZUSAMMENFASSUNG

Starke Korrosionserscheinungen an den Oesenstäben, die die Hauptkabel der Manhattan- und Bronx-Whitestone-Brücke verankern, liessen eine Reduktion der Tragsicherheit befürchten. Es werden neuartige Methoden vorgestellt, wie durch Abschneiden und Neuverankerung der Kabel oder durch Vorspannung diese Bauteile verstärkt werden könnten. Ferner werden preiswerte Kabelreparaturen an der Triborough-Brücke und eine Methode zur provisorischen Kabelhalterung während Reparaturen an der George-Washington-Brücke erwähnt.

1. INTRODUCTION

In 1850, the collapse of the suspension bridge at Angers brought to an end suspension bridge construction in France for 20 years. The cause was corrosion of the iron wires inside the concrete encasement below the ground level. [1] Wires had been embedded in this manner for 20 years, based on observations that iron bars embedded in concrete do not corrode. The bundled wires unfortunately are not bars. The mishap led to regulations requiring solid bars embedded in concrete to which to anchor cables. In 1845, Roebling anchored his cables by looping his wires around a strand shoe, much like passing a rope around a thimble, and fastened this to eyebars embedded in the masonry anchorages, a system which has been used in nearly all American suspension bridges since that time.

The steel eyebars anchoring the cables on two important suspension bridges in New York City, the Manhattan Bridge across the East River between Manhattan and Brooklyn, and the Bronx-Whitestone Bridge, between Queens and the Bronx, are the subjects of major rehabilitation projects. During routine inspections, it was noted that the paint on the eyebars just above the face of the concrete in which they are embedded was subject to exfoliation caused by expansion of the rust below. The bars had been painted only a few years before.

When chipping off the layers of loose rust, it was seen that a hard dark gray layer of corrosion product was firmly attached to the steel. Wire brushing only polished this material, and chipping with chisels and hammers was necessary to remove it. In both bridges, it quickly became obvious that the dark corrosion product concealed considerable section loss, and removal by grit blasting was required to clean the bars to white metal. During this process, it was found that the steel bars are indeed protected from corrosion inside their concrete embedment. In both cases, corrosion loss stopped abruptly at the concrete surface. Above the concrete surface, varying degrees of corrosion were found, with losses exceeding 40 percent in some eyebars (Figure 1, in inches).

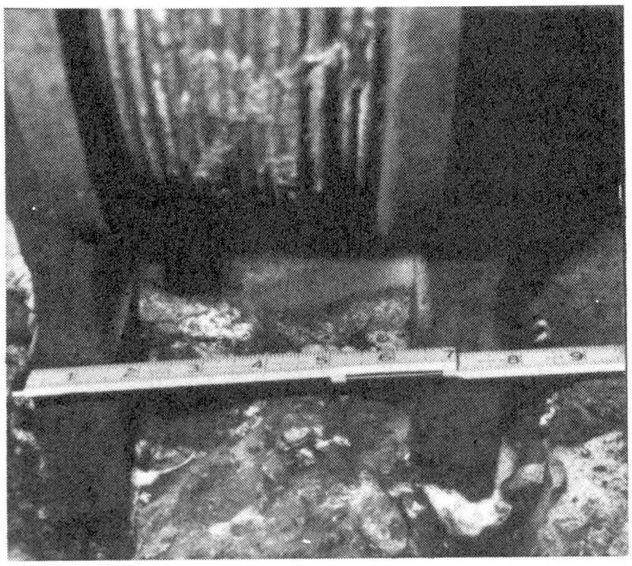

Figure 1. Corroded Eyebars on Manhattan Bridge

In both cases, drainage from the roadway is the major culprit in causing the corrosion. In the Manhattan Bridge, this chloride laden water entered through cracks and porosity in the unreinforced concrete vaults, exacerbated by the presence of an abandoned, but not sealed, system of clay tile drains in the concrete.

On the Bronx-Whitestone. the water entered through a joint between the roadway and -the anchorage and fell onto the concrete just next to the eyebars.

On both these bridges the eyebars emerge only a few feet from the concrete surface inside the very confined cable chambers - 3.3 m wide on the Manhattan Bridge and 4.6 m wide on the Bronx-Whitestone. The eyebars are arrayed in very tight groups, with only 50 to 125 mm between eyebars across the width of the chamber and about 350 mm vertically. It is virtually impossible to reach inside to the center of the array, and special calipers had to be made to measure the remaining thickness of the eyebar shanks.

2. MANHATTAN BRIDGE

On the Manhattan Bridge, the average loss in section in one of the cable anchorages was over 27 percent, with losses in some eyebars up to 44 percent. An analysis of the remaining eyebar strength, showed that the cable could not safely carry the transit or highway loads on this side of the bridge supported by this cable (there are four subway tracks on the bridge). Fortunately, this side of the bridge was closed for rehabilitation work, and the closure was extended for 18 months so the cable anchorage could be reinforced.

Because of the lack of access to the eyebars, a new anchorage system for half the cable was designed (Figure 2). The anchorage has longitudinal vaults 2.5 to 6 m wide, separated by 1.5 m thick concrete walls. The 2.4 m deep steel transfer girders were installed above and below the eyebar array, extending into adjacent chambers through openings cut through the concrete walls. These girders are anchored into the massive gravity anchor block by means of seven high strength steel rods at each end, installed in 860 mm diameter inclined shafts drilled 20 m into the concrete. The lower end of the shafts was accessed by means of tunnels driven into the back face of the anchorage, through which steel anchor girders were installed to receive the bottom end of these anchor rods.

The cable had many corroded wires; 10 percent of the wires had ferrous rust and 34 wires were already broken because of section loss. Five hundred wires had badly rusted sections removed and new wires spliced in. It was necessary to cut and reconnect 18 of the 37 strands of the cable to the new transfer girders, just to provide enough space to access inner strands. Half of these were then cut and reanchored to the vacated eyebars.

The procedure of reanchoring the strands to the new girders consisted of cutting one strand, using clamps and jacks to gradually relieve the force in the strand. The wires in each half strand were cleaned, fluxed and fastened to sockets using molten zinc. These sockets were connected to the transfer girders by high strength rods and had a force equal to the calculated load on the bridge jacked into them by means of Biach jacks. The force in each strand due to dead load is 1,550 kN with live load the total strand force to 2,100 kN. During jacking, a force of 2,760 kN was applied to set and test the sockets, and to be certain that the strand was pulled back out of the cable, into which it had slipped by about 15 mm. The force was then lowered to 1,600 kN and the nuts turned into bearing.

In order to prevent stretching of the anchor rods or flexure of the girders as load was transferred to them, the anchor rods were prestressed to hold the transfer girders firmly against the concrete of the anchorage. Because of the great force involved, this prestress was applied in steps as the cable strands were attached to the girders. During all operations, forces in the strands were monitored by means of strain gages attached to wires, as well as to the rods between the sockets and the transfer girders.

After all strands were reanchored, the bridge was shut down to subway traffic for one night, and the strand forces were tuned. All strain gages were read at each step; a program was developed to provide the force in each strand at each step. Two rounds of tuning were required to bring the strand forces to within 3 percent of the average strand force. The anchor rod shafts were then filled with concrete, and masonry walls were built to seal off the cable chambers. To prevent further section losses in the eyebars and cable, the chamber will be dehumidified.

The cable socketing procedure, which required specially cast steel sockets capable of holding 128 bridge wires was thoroughly tested in advance by requiring that the Contractor attach one socket of each size to a test section of strand, test the socket in tension, and saw cut the socket into quarters to inspect the interior. A final procedure was developed, requiring preheating to 427°C. Because of the high temperatures required, testing of wire specimens removed from the cable as well as new wires were tested by immersing in molten zinc at 427°C and 538°C. These tests show that the tensile strength decreases by up to 28 percent at 538°C. The socket tests, however, show that the loss of strength is only 8 percent for the entire assembly, an acceptable loss because the anchorage is not the location at which the cable is subjected to its maximum tension.

3. BRONX-WHITESTONE BRIDGE

On this bridge, the losses in area of the eyebars ranged up to 15 percent. Because of the short length subjected to corrosion, and because yield had not been reached in the corroded section, the ultimate strength of the ASTM A7 steel in the eyebars was depended upon for reserve strength until the repair could be made. Several of the severely corroded eyebars have been strain gaged and monitored to be certain that they are not subjected to yield.

In this anchorage, the eyebars are arranged in horizontal alignment, as opposed to the Manhattan Bridge, where the center row of eyebars is offset one half space from the others. Thus, it is possible on the Bronx-Whitestone bridge to needle girders through the loops formed by the strands. The wider cable chamber also makes it possible to provide direct anchoring of girders into the concrete inside the chamber. Girders have been designed which will bear directly on the front face of the strand shoes. By jacking forces into the girders by means of high strength anchor rods (ASTM A354 Grade BC with a tensile strength of 965 MPa), the girders will prestress the eyebars, thus reducing the tensile stress to an acceptable level. The forces in the eyebars and anchor rods will be measured by means of load cells permanently installed in the structure. This arrangement avoids the need for cutting cable strands. (Figure 3).

4. TRIBOROUGH BRIDGE

The forerunner of these major rehabilitation projects was the reanchoring of one strand one cable on the Triborough Bridge in New York City. More than half of the wires in the strand had been broken, again because of drainage from the deck above, and when the Contractor started to work on the rehabilitation, the remainder of the wires rapidly failed as well. The resulting tangle of wires had to be realigned by means of steel combs which aided in holding the wires while clamps were installed to hold the strand in shape.

Because it was the first field socketing of a suspension bridge cable in place, it was decided to use four sockets on the strand, smaller than those later used on the Manhattan Bridge. The procedure developed for preheating, providing filler tubes for zinc in the side of the socket, aligning and insulating was valuable in later designing the Manhattan Bridge procedure.

The strand shoe to which the strand was affixed was saw cut away from the eyebar pin, a new bearing block installed behind the pin and high strength rods and jacks used to re-stress the strand. There was concern that the strand would slip into the cable, and plans had been made to temporarily reanchor each quarter strand. The full release of the strand made this unnecessary, and pairs of quarter strands were reanchored directly to the original eyebars as they were completed. There was no difficulty in prestressing the strand, and it was pulled smoothly back out of the cable.

6. GEORGE WASHINGTON BRIDGE

Again, leakage from the roadway above has caused substantial wire loss and breakage in three of the 61 strands in one of the cables. The Owner has decided to provide tiebacks to prevent further loss in strand force while socketing these strands. Working with the contractor, a system of clamps, bars, tension rods and a strut have been designed which can be adjusted to fit any of the three. Gusset plates of ASTM A514 steel will be used to connect the clamp to a strut which provides the resultant force to turn the line of action of the strand tension downwards towards an anchorage prestressed against the concrete.

Because of limited space between strands and limited clearance to the steel framing supporting the roadway, flat eyebars are connected to the gusset plates to provide the needed tension. These eyebars anchor to a jacking beam which is, in turn, anchored by two high strength rods to the temporary anchorage.

Upon completion of the sockets, the strands will be reanchored to the eyebars in a manner similar to the Manhattan Bridge, and the tension in the temporary anchorage relieved gradually as the force is transferred to the eyebars.

7. CONCLUSION

These four projects demonstrate the feasibility of repairing deteriorated suspension bridge cable wires or eyebar anchorages. Thus far, no two cases have been alike, but with the application of innovative design, the repair of these important elements is possible and the life of these major structures extended.

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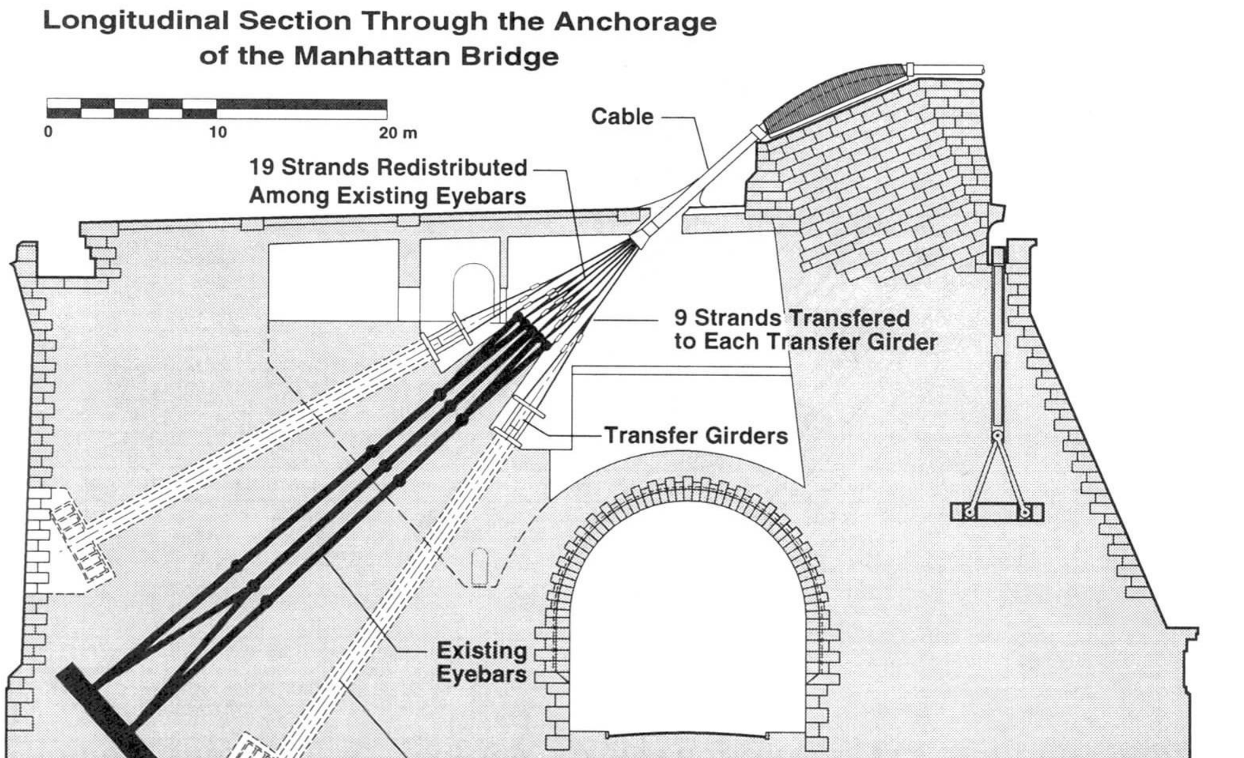
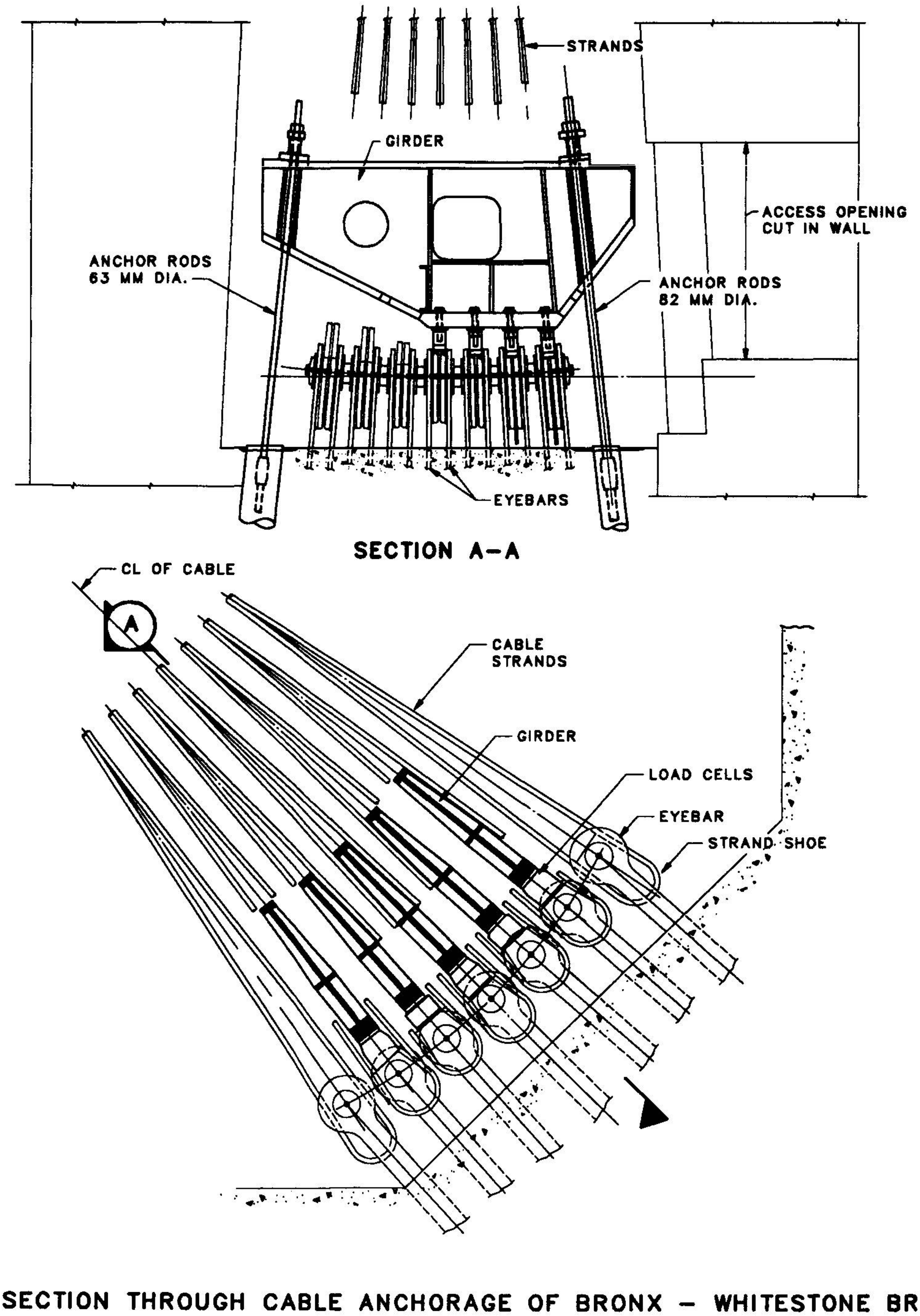


Figure 2. Reanchoring Scheme for Main Cable on Manhattan Bridge



SECTION THROUGH CABLE ANCHORAGE OF BRONX – WHITESTONE BRIDGE

Figure 3. Girders to relieve stress in eyebars of Bronx – Whitestone Bridge

A New Life for the Main Cables of Williamsburg Bridge

Nouvelle vie pour les câbles du pont de Williamsburg

Neues Leben für die Hauptkabel der Williamsburg-Brücke

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SUMMARY

This paper describes the cable preservation system designed for the main suspension cables of the Williamsburg Bridge in New York City. A review of the studies and field tests performed prior to the preparation of the Contract Documents for the cable rehabilitation precedes the work-in-progress report. The cable preservation system consists of the application of a corrosion inhibitor inside the cable, and red lead paste in the exposed surface of the main cable, wire wrapping and a neoprene wrapping system.

RÉSUMÉ

Les auteurs décrivent le système de protection des câbles porteurs principaux du pont suspendu de Williamsburg, à New York. Ils passent en revue les études en laboratoire et les essais sur le site qui ont précédé la préparation des documents servant à l'appel d'offres pour la réhabilitation de ces câbles; ils indiquent ensuite l'état actuel des travaux. Le système de protection consiste à appliquer un inhibiteur de corrosion à l'intérieur des câbles porteurs, une pâte au plomb de couleur rouge sur la surface extérieure exposée aux intempéries, un enroulement de fil métallique et une enveloppe externe en néoprène.

ZUSAMMENFASSUNG

Der Beitrag beschreibt das Schutzsystem für die Haupttragseile der Williamsburg-Brücke in New York. Zuerst werden die Laborstudien und Feldversuche geschildert, die der Aufstellung der Ausschreibungsunterlagen für die Tragseilerneuerung vorangingen, und anschliessend der gegenwärtige Stand der Arbeiten. Das Seilschutzsystem besteht aus einem Korrosionsverhinderer im Seilinnern, rote Bleipaste auf der Witterung ausgesetzten Seiloberfläche, Drahtwicklung und einer Neoprenebandagierung.

1. INTRODUCTION

The Williamsburg Bridge in New York City has a 488 m suspended span, two 182 m side spans which are independently supported on intermediate towers with no attachment to the main suspension cables (Figure 1), and steel bent and girder approaches. The bridge was opened to traffic in 1903, and currently carries eight traffic lanes, two heavy rail transit tracks and a pedestrian footwalk.

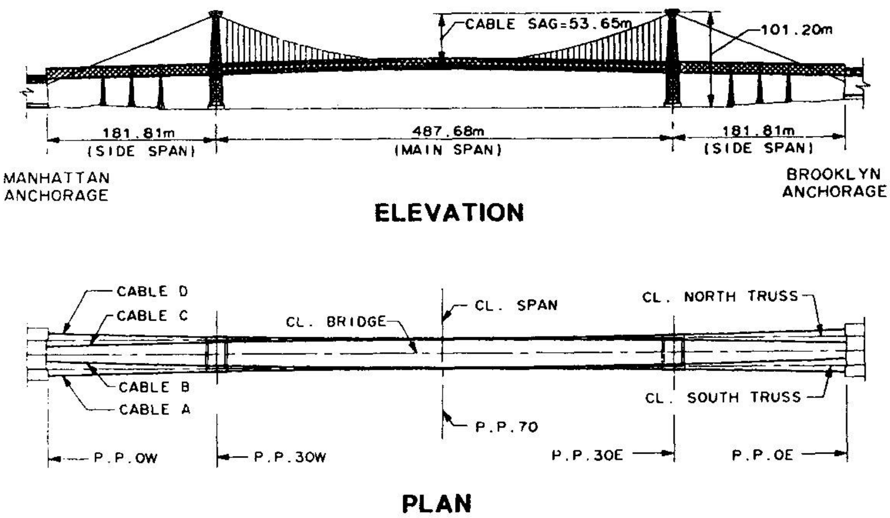

Figure 1
Plan and Elevation of Williamsburg Bridge

The main span is suspended from four cables of 476 mm diameter, each made of 7696 non-galvanized parallel steel wires 4.9 mm in diameter. The corrosion protection originally specified for the main cable wires included a shop coat of boiled linseed oil, and a shop coat and two field coats of slushing oil and graphite. The cables were compacted after spinning, and then wrapped with cotton duck impregnated with an asphaltic compound, and covered with a sheet metal sheath. By 1921 the sheath had badly corroded. The sheath was then removed, the cable was wedged and raw linseed oil was applied along its full length. After re-compaction the cable was wrapped with galvanized wire. In 1944 raw linseed oil was poured into the cables at their saddles on the tower tops, and in 1963 a similar operation was performed using fish oil and mineral spirits.

2. THE MAIN CABLE INVESTIGATION

A cable investigation conducted in the early 1980's projected that, by 1992, the cables would be unsafe to support the bridge and its traffic, and that cable rehabilitation would not be cost effective. The recommendation was to replace the cables. The Federal Highway Administration hesitated to participate in a cable replacement project on a substandard bridge with narrow roadway lanes and low clearances, and suggested a complete bridge replacement. In 1987 New York City and New York State Department of Transportation formed the Technical Advisory Committee (TAC) to evaluate the alternatives. It was recognized that the earlier studies were not supported by enough hard data, and Steinman was directed by the TAC to perform an in-depth cable investigation.

As the first steps in this investigation, a geometric survey of the cable sag and a photogrammetric survey of the deflection under controlled live load suggested that no significant loss of cable cross-section had occurred. Following this, the visual inspection, selection, sampling, metallurgical examination, testing and analysis of wire samples showed that significant corrosion had only occurred on a small number of wires, mainly those located in the bottom of the cable cross-section. Samples removed from the interior of the cables indicated that in some areas the oils used over the years to protect the cables had oxidized and dried [Ref. 1, 2, 3]. The worst condition was found at the Manhattan anchorage on Cable D. Hundreds of wires were broken between the splay casting and the strand shoes. The conclusion of the cable investigation was that the calculated factor of safety of the existing cables was at least 3.0, and that it could be increased to nearly the original 4.0 with minimal expenditure.

At the same time, the data gathered during the Biennial Inspection (1988) revealed that the most severe deterioration occurred at the approaches, while the trusses and towers of the main bridge were in relatively good condition.

In light of these conclusions, after extensive evaluation of replacement and rehabilitation alternatives, the Technical Advisory Committee decided that the rehabilitation of the suspension bridge and the complete replacement of the approaches was most viable from the economic point of view. The work was sequenced into several contracts, with the first one comprising the rehabilitation of the suspension system, which includes the cable preservation work described in this paper, plus the replacement of all suspenders, suspender connections to the truss, and cable bands. The reconstruction of the cable enclosures at the tower tops and anchorages, including replacement of the anchorage roofs, are essential part of this contract. The replacement of the approach structures and rehabilitation of the main bridge was staged into three contracts, with the first starting in 1995.

3. PRESERVATION SYSTEM FOR THE WRAPPED PORTIONS OF THE CABLE

After an evaluation of different cable preservation systems currently in use, which took into account the specific type of cable and its condition, the decision was made to use a penetrating liquid corrosion inhibitor, a red lead paste coating, non-galvanized wire wrapping, and a neoprene outer wrapping system.

3.1 The corrosion inhibitor selection

Several commercially available corrosion inhibitors were evaluated, and seven were selected for laboratory testing. Although some of these materials were found very good in resisting corrosion, most of them had very poor penetrating ability. Two non-proprietary materials were further selected for testing on the actual bridge cable. Pure raw linseed oil was finally selected, since it penetrated very well between the wires, including laterally and throughout the cable cross-section. This oil has been used successfully to treat suspension bridge cables for over one hundred years.

3.2 Cable oiling

A procedure for oiling and compacting the main cable was developed and tested in the field before being included in the Plans and Specifications for the Cable Rehabilitation.

Starting at the lowest point on the cable, a suspender and its cable band and the existing wire wrapping were removed, then 6 m long grooves were opened in the cable, using hardwood and plastic wedges (Figure 2). All the exposed wires were then inspected, and broken wires were spliced using a specially developed technique. Both ends of the broken wire were cut to allow for the splicing of a new piece of wire at each end, using press-on ferrules. A threaded ferrule connects the two ends of new wire, allowing it to be tensioned.

The uphill side of the wedges was then covered with petroleum jelly, to retard the flow of oil downhill in the groove and to obtain a better transverse distribution of the oil, and oil was applied inside the grooves (Figure 3). Based on the field test results, an amount equivalent to 25 liters of oil per linear meter of cable was specified in the Contract Plans; however, this application rate was somewhat modified during construction in response to varying conditions at different locations along the cable. After completion of oiling in one panel, the wedges were removed and the cable re-compacted. The procedure was repeated in each adjacent uphill segment of cable until reaching the cable saddles at the tower tops.

3.3 Cable Compaction

During the wedging and oiling operations the position of the main cable wires is disturbed, resulting in an increased diameter of the cable after wedges are removed. Cable compaction is then necessary to restore the original dimension and shape of the cable, essential for the correct installation of the cable bands and the proper functioning of the wire wrapping machine. Hydraulic compactors equipped with four 1000 kN capacity jacks were used for the cable compaction.

The main objective of the cable compaction is to minimize the voids between main cable wires, thereby reducing the opportunity for water and oxygen to enter and remain inside the cable, and for moisture condensation to occur. In addition, the clamping effect of the compacted cable allows for recovery of the full tension in an individual broken wire over a shorter length.

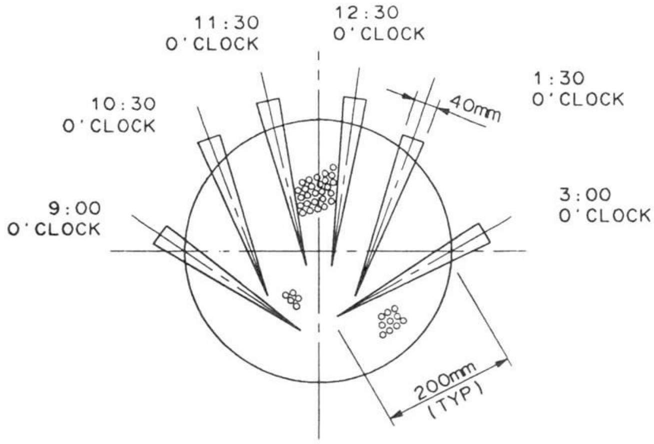

Figure 2
Cross-Section of Wedged Cable

Figure 3
Applying Oil into the Grooves

3.4 Cable Wrapping

The new wrapping system consists of a coat of red lead paste applied directly on the main cable wires, wire wrapping, and a neoprene outer wrapping system.

Red lead paste is used to coat the exposed surface of the main cable wires and to fill the voids between wires, in order to completely seal the main cable from moisture. Red lead paste was selected because it is the most effective corrosion inhibitor for the main cable wire surfaces.

The wire wrapping provides an armored protection for the main cable wires, and holds the compaction of the main cable wires between the cable bands. Non-galvanized wrapping wire is used on these cables (Figure 4) to avoid galvanic action which could lead to hydrogen embrittlement of the non-galvanized main cable wires.

The neoprene wrapping system provides a watertight protection against atmospheric attacks. The system consists of a liquid air-curing neoprene applied to the surface of the wire wrapped cable. Then 152 mm wide uncured neoprene sheetstock is spirally wrapped around the main cable starting at the "downhill" end of the panel. Thinner, which welds neoprene sheets on contact, is applied on the overlapping portion of neoprene sheets. Successive turns are installed with 50% overlap to create a "shingle" effect that prevents the entry of water through the joint between them. The cable band edges are caulked using a polyurethane sealant. An air curing coating is then applied on the neoprene wrapped surface. Ground walnut shells are sprinkled over the top surface of the cable, prior to the final coating, to provide an antislip surface (Figure 5).

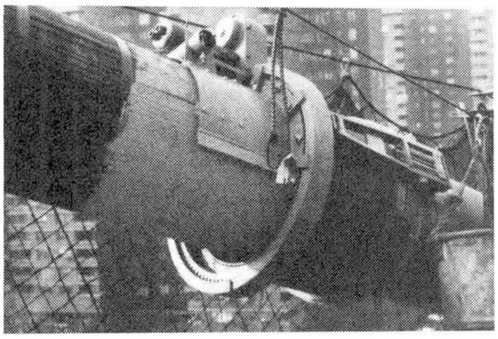

Figure 4
Wire Wrapping Machine in Place on the Main Cable

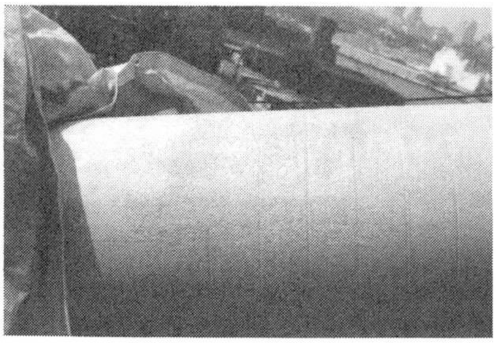

Figure 5
Finished Cable with the New Neoprene Wrapping

4. THE CABLE PRESERVATION SYSTEM AT THE ANCHORAGES

As part of the cable investigation, and after decision had been made to rehabilitate the main cables, Steinman replaced two strands and spliced many wires in the Manhattan anchorage of cable D [Ref. 4]. These repairs increased the calculated factor of safety from 3.0 to approximately 3.5.

The cable rehabilitation work at the anchorages includes oiling of all non-wrapped portions of the cable using a proprietary corrosion inhibitor and procedure developed by Steinman for the Brooklyn Bridge rehabilitation. Before oiling, any loose, broken, heavily corroded or galvanized wires are replaced using special splicing techniques. The corrosion inhibitor is reapplied every year as part of the routine maintenance. In addition, new watertight cable enclosures at the anchorages, a new waterproof anchorage roof and a passive ventilation system [Ref. 5], will help protect these non-wrapped portions of the cables from further corrosion.

5. THE CABLE REHABILITATION CONTRACT

The cable rehabilitation contract, bid at 73 million dollars, is now being completed by the joint venture NAB/KOCH under the supervision of the New York City Department of Transportation. Construction inspection is performed by Greenman Pedersen Inc. and construction support services are provided by Steinman.

Work started in September 1992 and is scheduled to be completed at the end of September 1995. The first stage was the construction of 2.4 m wide footwalks under each cable (Figure 6), except at the center of the main span, where the proximity of the cable to the roadway allowed the Contractor to work from scaffolding installed on the roadway. The work proceeded according to the Contract Plans, and as described here, except for some minor modifications. Work remaining to be done consists primarily of the cable enclosures and the remainder of the new roof at the anchorages. In addition to the work included in the Rehabilitation Contract Plans, the cable condition encountered after removal of the splay casting of Cable A at the Manhattan anchorage forced the decision to replace seven strands [Ref. 6].

Figure 6
Partial View of Williamsburg Bridge during the Cable Rehabilitation Work

6. CONCLUSION

The objective of the work described here was to rehabilitate the main suspension cables of the Williamsburg bridge to return them to nearly the original strength, and to provide for an additional life of at least 100 years. The protection that is being provided to the cables, together with regularly scheduled inspection, and maintenance when required, will provide for a potentially infinite life.

7. ACKNOWLEDGEMENTS

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Durability and Maintenance of the Shantou Bay Suspension Bridge, China

Durabilité et maintenance du pont suspendu de la baie de Shantou, Chine

Dauerhaftigkeit und Unterhalt der Hängebrücke über die Shantou Bay, China

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Guomin Yan, born in 1925, received his civil engineering degree in 1949 at Tsinghua University. bridge designer for about 35 years, he then turned to bridge research. He retired in 1990 and now is a consultant with the Major Bridges Reconnaissance and Design Institute, MBRDI.

SUMMARY
The Shantou Bay Bridge is under construction in China with a prestressed concrete box girder as its stiffened deck. The reason for using a prestressed concrete girder instead of the traditional steel type is mainly to improve durability of bridge deck against corrosion in the marine environment and to ease maintenance by eliminating the need for painting. But the self-weight of the whole bridge increases greatly, as does the cross-sectional area of two main cables. The stability of the bridge under wind pressure will be improved by the increased self-weight.

RÉSUMÉ
Le pont suspendu de la baie de Shantou est en construction en Chine. Le maître de l'ouvrage a donné la préférence à un tablier en poutre-caisson en béton précontraint, pour garantir une plus longue durabilité grâce à une meilleure résistance à la corrosion dans un environnement marin et à une maintenance plus facile en éliminant les renouvellements de peintures protectrices. Mais il a fallu accepter un poids propre nettement supérieur pour l'ensemble du pont et, en conséquence, de plus grosses sections des deux câbles porteurs principaux. Toutefois, cet important poids mort du pont agit favorablement sur la stabilité au vent de l'ouvrage.

ZUSAMMENFASSUNG
Die Shantou-Bay-Brücke ist zur Zeit in China im Bau. Für den Fahrbahnträger wurde Spannbeton dem traditionellen Stahl als Werkstoff vorgezogen, um eine höhere Dauerhaftigkeit gegenüber Korrosion durch die Seeluft zu erreichen und unterhaltsaufwendige Anstriche einzusparen. Aber das Eigengewicht der gesamten Brücke und damit der Querschnitt der beiden Hauptseile werden deutliche grösser. Das höhere Eigengewicht wirkt sich günstig auf die Windstabilität der Brücke aus.

1. GENERAL DESCRIPTION OF THE BRIDGE

Shantou Bay Bridge is located at the coast city Shantou of Guangdong Province in south China. Its main part was designed as a 3-span suspension bridge with span-length of 154 + 452 + 154 metres (Fig. 1).

The distance between two parallel main cables is 25.2 metres apart, it will give a bridge deck width of 24.2 metres and carry six lanes of motorway with two sidewalks. Each main cable will be made of 110 strands of 91 $\phi 5$ mm steel wires with cable diametre about 550 mm to 560 mm, and erected by PPWS method. The span/sag ratio of main cable in main span is 10, and in side spans is 29.6.

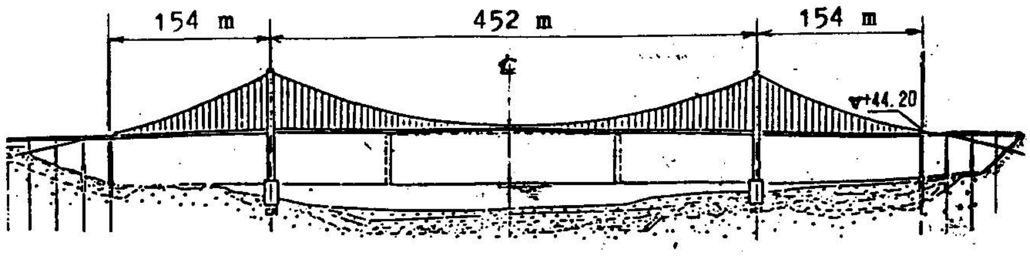

Fig.1 Profile of Shantou Bay Bridge

The concrete towers are of portal type with two intermediate horizontal beams (Fig.2). The total height of tower columns H = 95.10 m, and the top of tower is about 50 metres higher than the bridge deck level. The cross-section of hollow concrete tower columns are cast in "D" type, of which the straight wall is placed inside of the column, and the curved outside, thus to improve wind load condition. Besides this, the centre line of straight wall of tower column was designed to be coincided with the centre line of main cable. The outskirt dimensions of "D" type column are 6 m longitudinally and 3.5 m transversely, all in constant values (unvariable).

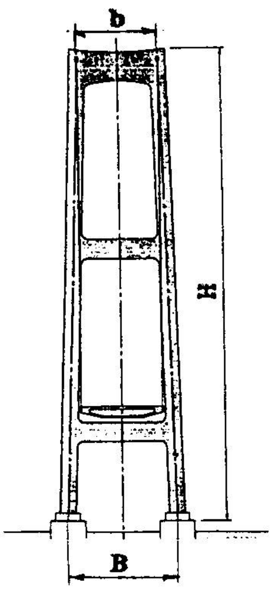

Fig.2 Portal Tower

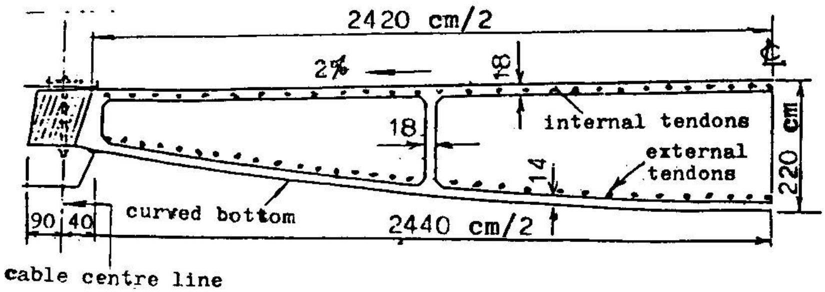

Fig.3 Half Cross-section of PC Box Girder

The PC stiffenef box girder is used for the main suspension bridge. Fig.3 is the half cross-section of this PC stiffened box girder. It is of streamlined shape with R = 7688 mm curved bottom plate. It is just something like a cross-section of the wing of an airplane if we put it upside-down. The 3-cell concrete box girder has two intermediate vertical webs with 18 cm in thickness. The thickness of upper and bottom plate is 18 cm and 14 cm respectively. The girder depth is 2.20 metres at the centre line of the bridge. The PC box girder is supported by the vertical hangers made of steel ropes with longitudinal spacings in 6 metres apart.

2. WHY PC BOX GIRDER IS USED AS STIFFENED BRIDGE DECK

Generally speaking, in order to reduce the self-weight of suspended construction of a suspension bridge, as well as its main cables and all the bridge parts such as cable saddles, cable bands, cable anchors, vertical hangers,etc, traditionally the steel girder (box or truss) will be used, especially for long span-length bridges. But for Shantou Bay Bridge, considering the durability and maintenance problems, PC box girder was finally selected to substitute traditional steel construction.

Shantou Bay Bridge is now under construction to link the mainland and Ma Yu Island in Shantou Bay. The suspension bridge is just over the sea. The marine condition is very unfavourable for steel girder, because it will be easy to make corrosion by heavy salt particles in the air surrounding the bridge superstructure. Therefore, using PC box girder can lengthen the lifespan of stiffened bridge deck by higher capacity of resisting corrosion than steel girder. Simultaneously, it can avoid periodical paint renovating which will be necessary for steel girder, and thus to ease the maintenance work of the bridge deck which must be done in high sky and inside the steel box girder with narrow space and high temperature & humidity.

3.DISADVANTAGES CARRIED BY PC BOX GIRDER

The very special feature of Shantou Bay Suspension Bridge to use PC box girder as stiffened deck for the sake of extending life span and easing maintenance will carry some significant disadvantages.

The major disadvantage is to increase the self-weight of the whole bridge.The unit weight of PC box girder is about 320 kN/m instead of 130 kN/m if steel box girder would be used. Therefore, the cross-sectional area and selfweight of two main cables also increase much more. Comparison with other suspension bridge of similar span-length and bridge width shows the unit weight of main cable will be increased about 80%. Other bridge parts such as vertical hangers (its longitudinal spacings of 6 metres is about only half of those used in suspension bridges with steel girder, so the amount of it is about double), cable bands (amount and size), cable saddles and anchorage materials etc. are also correspondingly increased.

The difficult of erecting PC box girder is the second disadvantage. Each girder section 5.7 metres in length and 170 tons of weight will be erected one by one to hang it to connect with the lower ends of a pair of vertical hangers. After a series of girder sections has been hung up, the 30 cm wide wet joints between the girder sections will be then cast-in-situ with some necessary fixing apparatus. There are altogether 121 girder sections, so the erection work will be heavier than to erect steel girder sections with about 12 metres in length and 150-160 tons of weight.

The third disadvantage is higher cost and longer construction period of bridge, because of that more cable work and girder erection must be done.

The fourth disadvantage is that, the self-weight of whole bridge increased by using PC box girder will be unfavourable to resist seismic response during earthquake. So, the substructure must be strengthened to meet the requirement of earthquake design.

4. ADVANTAGES COME WITH DISADVANTAGES

Although some disadvantages mentioned above will be carried by using PC box girder to substitute traditional steel girder, but some advantages reversely will come along with disadvantages.

First, the wind resisting capacity of the suspension bridge will be much more increased by the contribution of heavier self-weight of the whole bridge. Any wind induced vibrations will be much more moderated. The vibration amplitude will be smaller and frequency will be higher.

Another advantage perhaps would be shown after long time service of the bridge. Generally speaking, the main cables and their anchorages of a suspension bridge are not easy to renovate or strengthen, but the relatively minor members such as hangers and stiffened girder are easy to change or renovate if it would be necessary to be done. Using PC box girder is firstly a key factor to extending the life span of the suspended bridge deck. If the PC box would be damaged after its designed service time, or would be renovated from PC box girder to steel one by some unexpected accidents or widened by increased heavier traffic condition within its service period, it would be then possible to utilize the reserved excessive strength of main cables and their anchorage capacity for a new steel stiffened box girder which is much more lighter in weight than PC box girder.

5. CONCLUSION

Because of the temperature at bridge site is always higher than 0°C within a whole year, no snow or ice will appear, thus no salt-spreading damage problem on concrete deck will occur to PC box girder. Therefore, using PC box girder for a suspension bridge to extending its life span and ease deck maintenance will be available. Of course, the cost of the bridge will be more expensive by using PC box girder as its stiffened deck.

Railway Installation on the Tagus Suspension Bridge in Lisbon, Portugal

Trafic ferroviaire sur le pont sur le Tage à Lisbonne, Portugal

Eisenbahnverkehr über die Tagus-Brücke in Lissabon, Portugal

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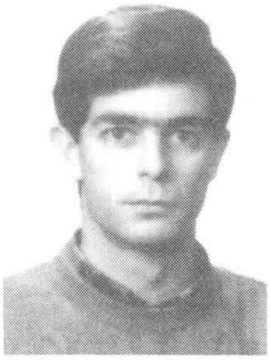

José J. Oliveira Pedro, born in 1968, received his civil eng. degree and MSc. at the Techn. Univ. of Lisbon. Since his graduation he has been involved in research and design in the field of bridge engineering at the University and also at GRID, Lisbon.

SUMMARY

Design solutions are presented for the railway installation on the Tagus suspension bridge in Lisbon, in particular the strengthening of its northern access viaduct. The roadway deck, presently with mid-span hinges requiring frequent maintenance, is transformed into a continuous deck by the use of external prestressing. The design solutions to install the railway deck at the viaduct are discussed. Typical results of the structural models developed and in-situ measurements are referred to. The construction works on the suspension bridge and the northern access viaduct are expected to start in 1995 and are outlined in this paper.

RÉSUMÉ

Cet article décrit le projet pour le passage du chemin de fer sur le pont suspendu sur le Tage à Lisbonne. Le renforcement du viaduc nord est présenté. Le tablier est actuellement une suite de poutres simples, qui nécessite un entretien fréquent. Il sera transformé en travée continue par application de précontrainte extérieure. Les solutions pour l'installation du tablier ferroviaire dans le viaduc sont discutés. Les résultats des modèles structuraux développés, de même que les mesures "in-situ" sont présentés. Les travaux du pont suspendu et du viaduc nord commenceront en 1995 et sont ici résumés.

ZUSAMMENFASSUNG

Dieser Artikel beschreibt das Projekt, den Eisenbahnverkehr über die Tagus-Hängebrücke in Lissabon zu führen, insbesondere die Verstärkung des nördlichen Viaduktes. Die Fahrbahnplatte ist gegenwärtig mit Zwischengelenken ausgebildet, die einen ständigen Unterhalt benötigen. Das System soll mit Hilfe der Vorspannung in eine durchlaufende Platte umgewandelt werden. Einige entwickelte konstruktive Modelle und Resultate der Messungen werden beschrieben. Die Bauarbeiten an der Hängebrücke und am nördlichen Viadukt dürften 1995 beginnen und werden in diesem Artikel erwähnt.

1. INTRODUCTION

Tagus Suspension Bridge (Fig.1) and its Northern Access Viaduct was built almost 30 years ago and originally designed for highway and railway traffic. The bridge was built with a roadway deck only, leaving the railway installation as a 2nd phase project. Three decades later the Government decided to install the railway deck, however for train loads much higher (about 2.5 times more) than originally envisaged. The original design for the railway installation was based on strengthening of the suspended spans through a set of cable stays anchored on the existing towers.

On the northern side, a long access viaduct (about 1 km) required also the installation of the railway deck as also the strengthening of the roadway deck. A brief description of the design studies and the solutions achieved for this project is presented in this paper with particular relevance for the design aspects where the authors were involved, namely the Basic Studies for the suspension bridge and the Rehabilitation and Railway Installation on the Northern Access Viaduct.

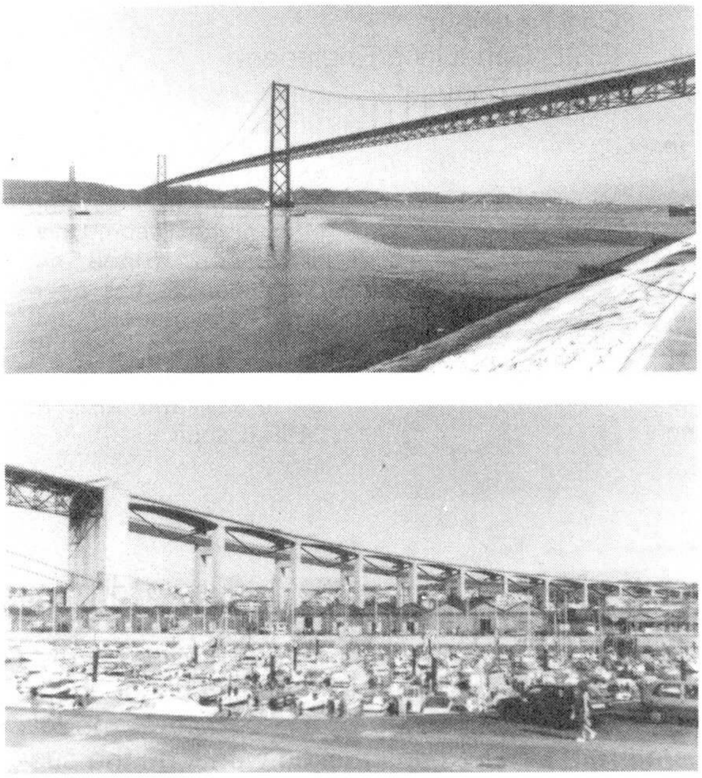

Fig.1 Tagus Suspension Bridge and Northern Access Viaduct with a perspective of the railway deck.

2. PROJECT DESCRIPTION

The railway installation on the Tagus Suspension Bridge, with a main span of about 1000m, requires the installation of a secondary cable on the bridge, the strengthening of the main truss and the execution of new structural system under the roadway deck to support the new railway deck (Fig.2).

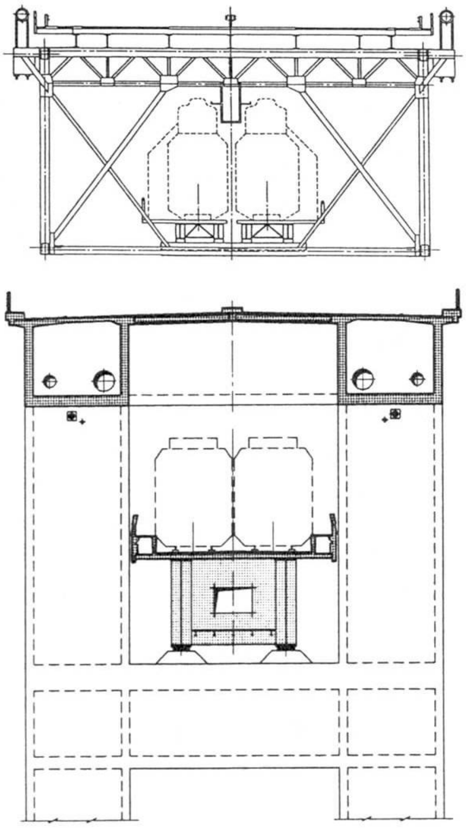

Fig.2 Location of the new railway deck on the bridge and access viaduct

At the Northern Access Viaduct (Fig.3) the existing structure is a prestressed concrete bridge 937m long with typical spans of 76,0m and 74,2m. The roadway superstructure was built also in the sixties by a balanced cantilever scheme with expansion joints at every mid span. These joints allow relative longitudinal movements and rotations between the two extreme sections of the cantilevers but restrict vertical and transversal horizontal displacements. The existing hinge devices are made of high strength steel and are shown in Fig.4. Due to a deficient performance of these hinges, relative vertical displacements at the end sections of the cantilever produce deterioration of the expansion joints requiring frequent maintenance. Presently 95% of the original expansion joints were

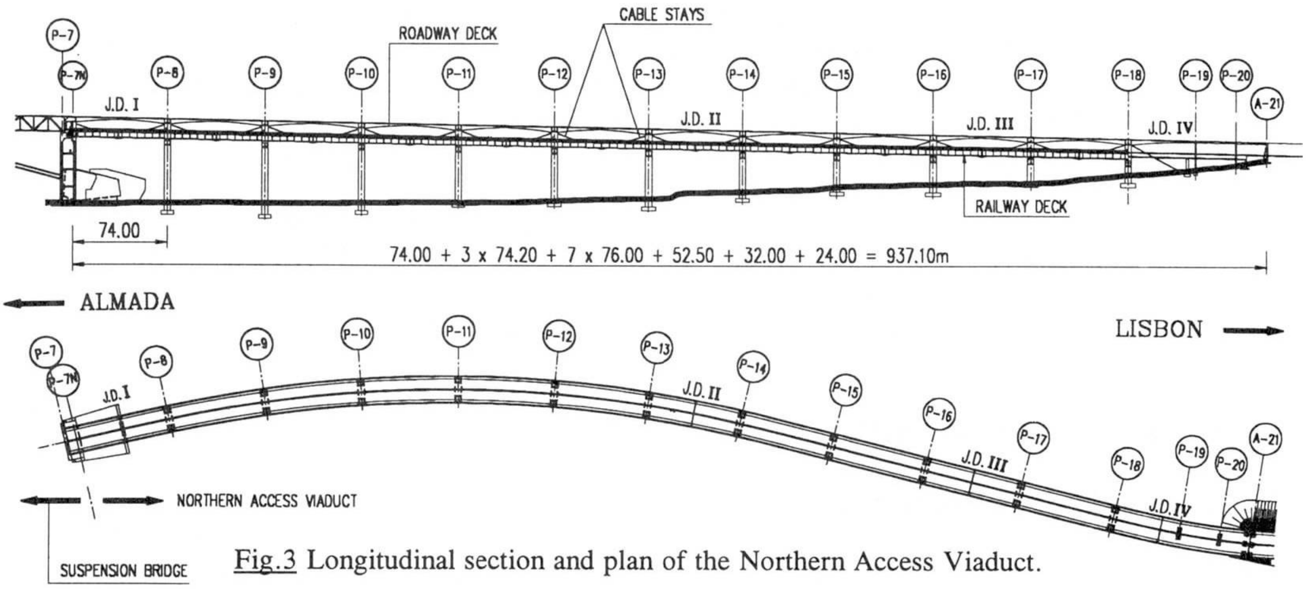

Fig.3 Longitudinal section and plan of the Northern Access Viaduct.

replaced by neoprene Transflex type joints requiring also considerable maintenance (including replacement by new ones) due to the dynamic effects of the relative displacements refered to above.

A design to eliminate the expansion joints was then envisaged since creep and shrinkage deformations are stabilized. Besides, due to the dynamic load effects at the joints, extensive cracking of the cantilever segments, adjacent to the joints, were detected by the inside of the box girders. So, a rehabilitation design of the roadway deck had also to be develloped.

The railway deck shall be installed between the two legs of the existing piers. A continuous composite steel-concrete bridge superstructure described in section 5 was designed. This structure is supported at the cross beams between the two pier legs (Fig.2).

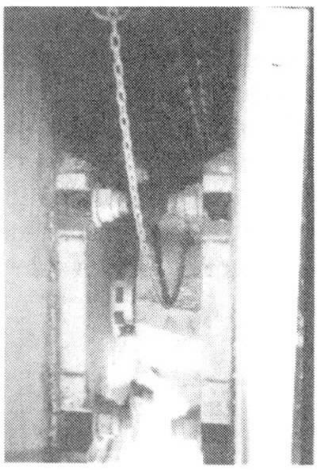

Fig.4 Existing hinge devices.

3. BASIC DESIGN CRITERIA

Prior to the preliminary design phase, a set of basic studies were develloped to establish the design criteria for the Suspension Bridge. That was decided due to the singularity of the project namely by the lack of national or european codes to be directly applied to a project like the one under discussion. The bridge was originally designed for light trains and the standard UIC ("Union Internationale de Chemins de Fer") railway loads, adopted in the portuguese code, would cause a too severe design criterion taking into consideration the type of trains expected to run on the bridge. The avaiable load capacity of the piers, towers and foundations as also the limits for deformability required for the suspended spans, were taken into consideration. On the basis of several studies, it was decided to design, the strengthening of the bridge for a set of train loads the most severe ones are freight trains with maximum load of 13800kN, passenger trains with a maximum load of 7232kN for the case of 4UTE type and double deck trains with 7400kN.

It was decided by economical reasons to establish the following traffic constraints:

- only one freight train on the suspension bridge;
- maximum of two passenger trains simultaneously on the suspension bridge;
- maximum speed of the train - 60 Km/h;
- maximum expected number of trains on the bridge - 250 per day in each direction.

The structural design was based on a set of limit state design conditions for rare, frequent and "quasi-permanent" load combinations. At the ultimate limit state (ULS) the train loads, the highway loading and other variable loads were combined according to standard ULS format adopted in the new Eurocodes 1 and 3 as also in the Portuguese Code for Actions.

The design of steel components were based on British Standard for Steel Bridges BS5400 Part 3 and UIC recommendations for railway bridges. The maximum slopes allowed under railway loading were limited to 2.5%. In the Viaduct, the longitudinal profile has already 2.0% and so the maximum allowable slopes due to structural deformations are 0.5%.

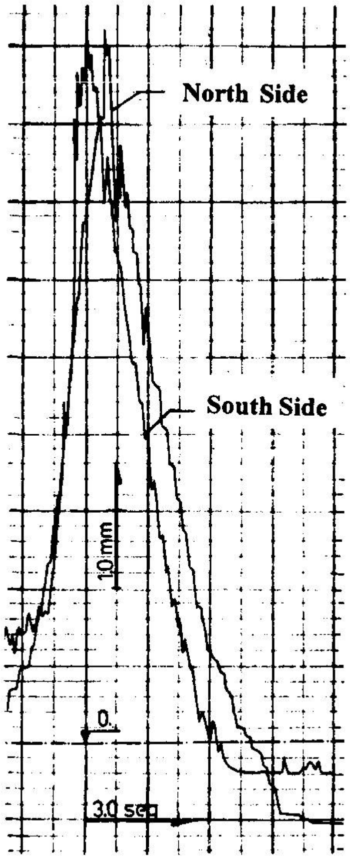

Fig.5 Displacements of the two adjacent sections to hinge 17-18 duo to load test vehicle, CMEST Report 3/94.

4. REHABILITATION OF THE ROADWAY DECK

4.1 Presente Structural Performance

Presently two types of anomalies are observed in the roadway deck - relative vertical displacements between the cantilever end sections at some of the expansion joints and extensive cracking at the box girder segments adjacent to the expansion joints. Typical field observations are shown in Fig.5. The refered relative vertical displacements are due to anormal deterioration of the steel hinge (Fig.4) and are responsable for the high maintenance that had been required at the expansion joints. In what concerns cracking of the concrete superstructure it was concluded, from field observations and from the analysis of the original design, it is due to the impact load effects induced by the relative vertical displacement at the expansion joints combined with a very low level of ordinary steel reinforcement and complete lack of prestressing (Fig.6) at the end segment of each cantilever.

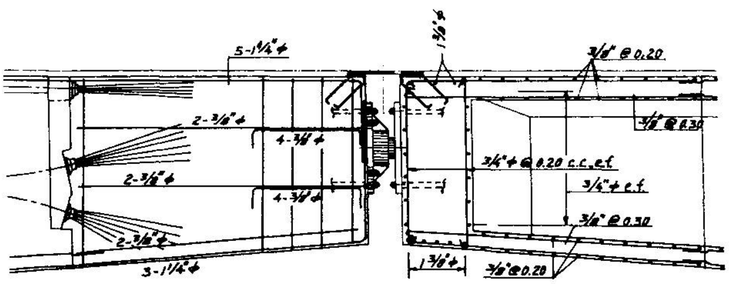

Fig.6 Typical cantilever joint - prestressing and reinforcement.

4.2 Improving Structural Continuity

As previously refered to, it was decided to eliminate as much as possible, the expansion joints at the roadway deck. The main structural effects are increased bending moments in the piers and foundations due to thermal actions in the deck and induced positive bending moments at the span segments due to traffic loads and positive thermal gradients at the deck sections. The former effect, has a direct implication on the number of expansion joints that can be eliminated. The last effect requires the design of a strengthening system for the deck.

From a detailed structural study it was concluded about the possibility of eliminating all the expansion joints except at the deck end sections (EJ I and EJ IV in Fig.3) and at the two intermediate sections (EJ II and EJ III). This solution was adopted through an optimization process where the number of expansion joints was maximized under the constraint of the design resistent bending moments of the piers. The existent foundations were checked for the structure obtained after the continuity introduced at the expansion joint deck sections. The resulting structure after elimination of 67% of the present expansion joints has a continuous deck along 452m, 225m and 152m as may be seen in Fig.3.

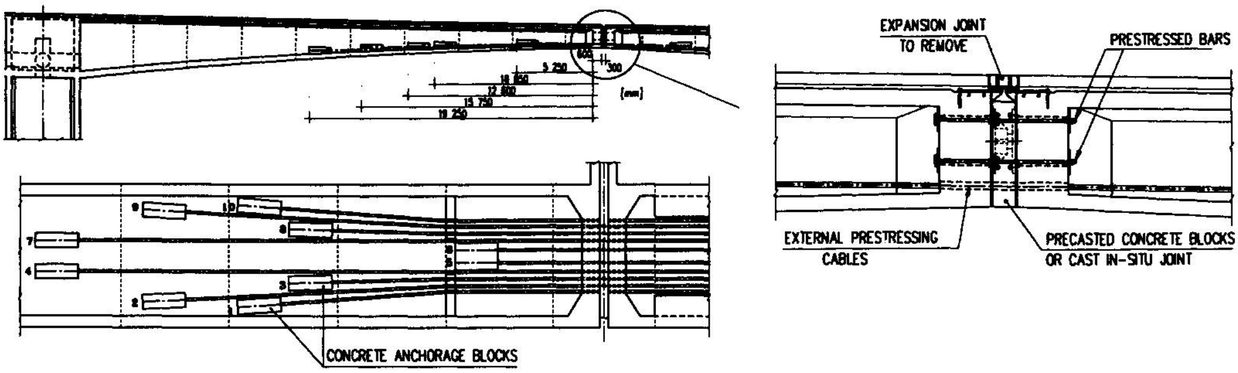

Fig.7 Design solution for a continuos joint: external cables layout and prestressing bars.

The design solution to obtain continuity requires:

- to insert precasted concrete blocks or alternatively to cast in-situ a transversal segment;
- to introduce a continuity prestressing at the joint and at the span sections through prestressing bars and external prestressing cables (Fig.7).

One typical layout of the cables arrangement is shown in Fig.7 which were introduced to rehabilitate also the box girder segments adjacent to the original expansion joint. Since no complete interruption of traffic is allowed during the execution of the works, a temporary prestressing is introduced through two of the cables in order to have minimum compressive stresses of about 1 MPa at the bottom flange box girder sections before permanent continuity is obtained. So, the temporary prestressing is stressed before continuity is introduced at the expansion joints, being the temporary cables replaced by permanent ones at the final stage of the closing operations.

4.3 Structural Behaviour

The bridge structure as modified by the continuity introduced at the span joints referred to in the previous section, was analysed with a 3D frame model. Longitudinal 3D beam elements at each segment of the two box girders, rigidly connected to transverse beam elements modelling the bridge deck, were considered. A linear structural analysis was carried out, since creep and shrinkage effects are no longer present. The ageing effects on the concrete modulus of elasticity and on its compressive strength was taken into consideration.

The structural design for the strengthening of the bridge deck is governed by load combinations of "permanent actions + highway loads + differential thermal actions" where internal and external prestressing load effects are included in the permanent actions. One of the most interesting challenges was to design the external prestressing to garantee no tensile stresses at the joints where continuity was introduced (i.e. the decompression limit state) for the "rare combination". This load combination includes the characteristic value of the highway loads (4 kN/m^2 uniform live load + 50 kN/m uniform transverse load acting at any cross section to produce the most severe load effects) together with the "frequent value" of the thermal gradient along the depth of the bridge deck (7.5°C between the top and bottom fibers). Local load effects under the "truck" live load (600 kN in 3 axes of 200 kN each, 1.5m distance between axles) were investigated on the basis of a finite element shell model of a typical viaduct span. However, for the normal stresses (bending+axial forces) at the joints where continuity was introduced, the "rare combination" referred to above is the governing design condition. Fig.8 shows the variation of the normal stresses along the extreme fibers of the bridge deck, obtained after the external prestressing. At each mid span sections the maximum compressive stress is about 6 MPa while the stress at the bottom fiber is between -1 MPa and zero. One shall note a permanent compressive stress at the two joint adjacent sections (-1 MPa at the top and bottom fibers) due to the almost axial external prestressing introduced at the span where the expansion joints were kept.

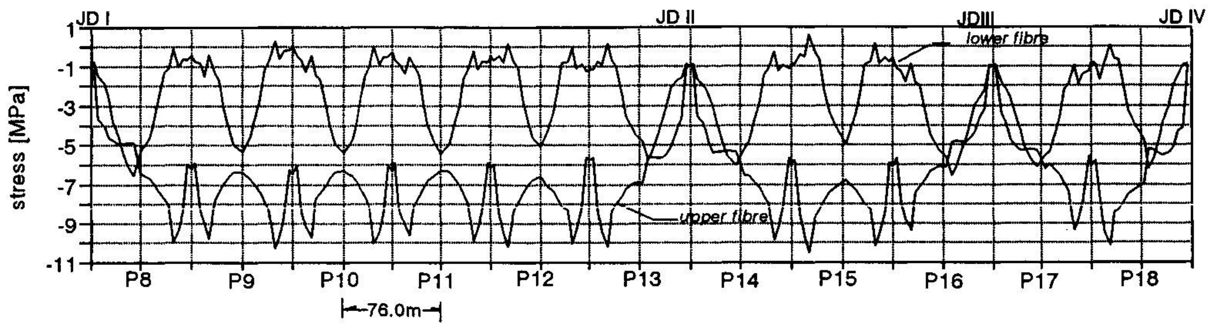

Fig.8 Normal stresses [MPa] for the rare combination at the extreme fibers of the deck.

Finally one notes the local effects introduced at the box girder flanges by the anchorages blocks (Fig.8) of the external prestressing. A detailled finite element analysis was required to predict increased craking effects under transverse tensile stresses introduced by the anchorages.

5. INSTALLATION OF THE RAILWAY DECK

Taking as a constraint, the avaiable load carrying capacity of the piers and foundations, structural solutions for the railway deck were investigated. The aim was to design a light railway deck, because earthquake forces tend to be the governing design actions for the piers. Then a steel structure was prefered to a prestressed concrete one. However, if a full steel deck was selected, increased difficulties had to be faced to reduce noise impact, in particular at the northern spans where the railway is only a few meters above residential and office buildings. So, a composite concrete and steel superstructure, with side noise barriers (Fig.9), was the most convenient. Several structural composite solutions, were investigated namely: a superstructure with two plate girders, a single cell box girder solution,a superstructure with 3 truss girders and a cable stayed plate girder solution. This last solution was selected for the final design. It consists on two plate girders, 4200mm deep, with transverse diaphragms. A cable stayed scheme is introduced at every span, with a slidding deviation cell under the mid span section of the deck (Fig.9), with a spacial configuration due to several geometrical constraints. The stays, anchored at the piers, are independent at every span and induce upward(prestressing) forces at mid span in order to reduce the permanent bending moments in the girder. A meaningful reduction in the plate thicknesses of the flanges is achieved by this scheme.

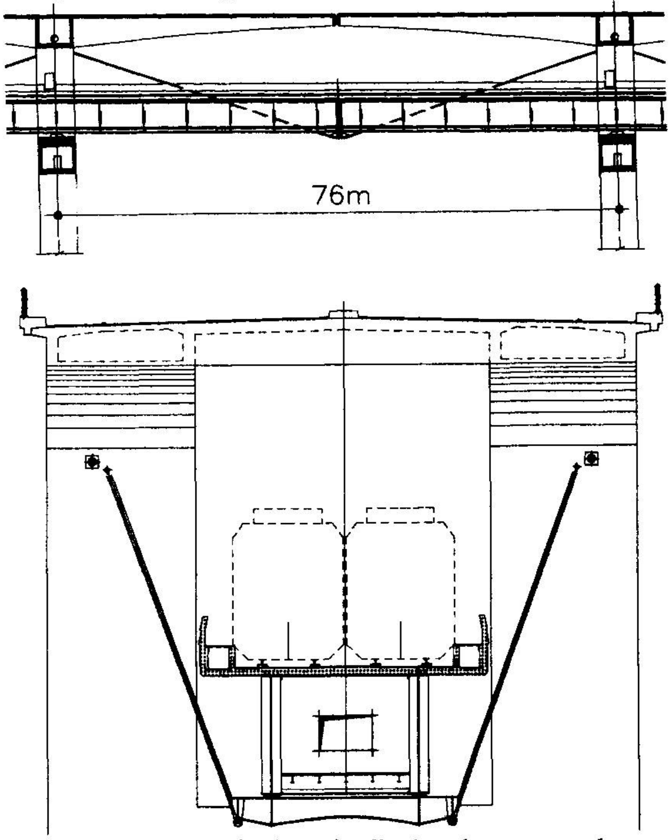

Fig.9 Railway deck. Longitudinal and transversal section.

AKNOWLEDGMENT

The authors are grateful for the cooperation and permission of GECAF ("Gabinete de Gestão das Obras de Instalação do Caminho de Ferro na Ponte sobre o Tejo em Lisboa") to publish the results of the study.

Repair of the Stay Cables of the Polcevera Viaduct in Genova, Italy
Réparation des câbles de haubans du Viaduc de Polcevera, Gênes, Italie
Reparatur der Hängeseile des Polcevera Viaduktes in Genua, Italien

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SUMMARY

The Polcevera Viaduct, designed by Prof. R. Morandi, was built in the 1960s. It is one of Italy's most important bridges with prestressed concrete stays. The article describes the restoring project of the concrete stays and the works performed without stopping the traffic. The project required the installation of new tendons for each couple of concrete stays.

RÉSUMÉ

Le viaduc de Polcevera, conçu par le Professeur R. Morandi, a été construit dans les années 1960. C'est l'un des plus importants ponts en béton précontraint. L'article décrit le projet de réparation des haubans en béton et les travaux exécutés sans bloquer la circu-lation. Le projet a nécessité l'installation de nouveaux câbles pour chaque couple de haubans en béton.

ZUSAMMENFASSUNG

Das Polcevera-Viadukt, das von Prof. R. Morandi entworfen wurde, wurde während den 60er Jahren gebaut. Sie ist eine der wichtigsten vorgespannten Brücken. Der Abschnitt beschreibt den Restaurierungsentwurf der Betonschrägseile und die Arbeiten, die ausgeführt wurden, ohne den Verkehr anzuhalten. Der Entwurf erforderte die Einrichtung von Schrägkabeln für jedes Paar von Betonschrägseilen.

1. INTRODUCTION

The Polcevera Viaduct, designed by Prof. R. Morandi, was built in the '63 - '66 years to connect the motorways Milano-Genova and Genova-Savona.
The concrete bridge, with 11 spans 24 m wide ranging from 43 m to 208 m for a total length of 1121 m, passes over the Polcevera river, the marshalling yard and a town quarter.
The main important and striking part of the viaduct consists of three up to 172 m stayed cantilever box girders, that, with the 36 m central closing girders, arrives to cover 208 m.
During the periodical inspections and the restoring works made in the course of last years to the structure, furthermore the general degradation of the concrete stays, some local higher fault was found at the pilon 11.
At the bottom of the connection of the concrete stays with the transverse beam on the pilon top, a big void was pointed out, where several 0,5" strands were completely corroded.
A severe series of investigations were immediately started in order to make a complete check-up of the structure.
The investigations have the double aim to evaluate the phisical-technical characteristics of the concrete and of the reinforcements and to analize the conditions of the prestressing tendons, defining their tensioning level with particular care for the concrete stays.
About the first aim the following tests were made:

- on the concrete : ultrasounds; pull-out; Windsor; crushing of carrots; strength investigations; determination of the thickness of the concrete affected by carbonation

- on the steel : laboratory tests made on samples of bars and strands

About the second aim the following tests were made:

- endoscopic inspections on prestressing tendons
- releasing of tension on the concrete carrots
- reflectometric inspections
- dynamic investigations

The analyses and the valuations of the results have remarked the necessity of a general restoring of the bridge with some localized particular interventions and the necessity of a structural reinforcement of the concrete stays of pilon 11.
Many reasons, as the conservation of the bridge architecture, the minimisation of structural risks due to the eventual demolition of the prestressed concrete of the stays, the impossibility of closing the bridge to the traffic, the opportunity of maintaining the rigidity of the existing concrete stays, have addressed to the solution of reinforcing the concrete stays with additional tendons.

<u>The technical concept</u>

In order to better understand the originality of the designed solution, it is necessary to know the statical behaviour of the prestressed concrete stays.

The stays are constituted by two orders of 0,5" strand tendons.
The main tendons (24 x 12T13 strands), were tensioned at the extremity of the bridge and principally they support almost all the dead load of the cantilever box girders.
Successively the concrete beams of the stays were casted in situ along the main tendons, in segmental way, till 3 m from the deck and the concrete beams were post-tensioned by means of the secondary tendons (28 x 4T13 strands).
The secondary tendons were then extended till the anchoring points (the abutment or the transverse beams), the gap was concreted and finally the extensions of the tendons were tensioned and all the tendons grouted.
So, the post-tensioned stay-beams support almost all the live loads.
In this way the Designer reached the aim to minimize the variations of tensions (fatigue loading) in the strands, because all the prestressed concrete of the stay-beams is working under the effect of the traffic.
The repair solution has taken care of this situation as well as the necessity to control the compressive stresses in the concrete stays.
Because of the corrosion at present some strands are broken and other ones could be broken in the future, causing an additional loading on the concrete up and down of the breaking sections; it is therefore necessary to regulate the stress in the concrete to avoid its indesired bursting.
The reinforcement solution, designed by F. Pisani, provides two orders of cables for each concrete-stay:

6 + 6 , 22T15 Super strands, "long" cables
3 + 3 , 31T15 Super strands, "short" cables

The long cables, located along the two vertical faces of the concrete-stays, are linked to them by means of steel "ties", fixed on the top of the pilon to the steel "caps" and under the bottom of the transverse beam or abutment to the steel plates, "breeches", that on the transverse beam is a bi-direction ribbed plate.
The short cables, located on the two sub-horizontal faces of the concrete stays, are fixed to the lower extremities of the concrete stays through the steel "checks" and to the same "breeches".
The long cables can be compared with the main tendons of the concrete-stays and the short one with the secondary tendons.
The short cables have also the temporary function to bear all the load of the bridge during the cutting operation of all the old strands in the lower extremity of the concrete stays.
They have also the permanent function to allow the regulation of the stress level in the concrete stays.
The new stay cables have been manufactured with unbonded strand encased in a HDPE duct finally grouted. They can be checked, tension regulated and substituted entirely or strand by strand.
The structural analysis of the intervention has been made by means of the F.E. code SAP 90.
The effects of any operation on the structure were checked through:

- topographic survey of the top of the pilon, of the extremities of the cantilever box girder, of the sag of the concrete stays.
- monitoring, with the deformameters, 6 characteristic sections along the concrete stays and 3 ones in the cantilever box girder.

The sequence of the restoring operations

All the restoring operations were made in symmetrical way, first on the couple of concrete stays to mountain side and after on the sea side one.

Phase 1 - Installation of the monitoring instruments and totographic marks

2 - Installation of the "caps", "ties" and "breeches"

3 - Installation of 4+4 long tendons

4 - Tensioning of the oversaid tendons. This operation is made in order to compensate the effects of the additional loads applied to the structure (ties, breeches, strands)

5 - Installation of 8 + 8 long tendons

6 - Tensioning of only 4 + 4 tendons of the oversaid ones

7 - Demolition of the ending 50 cm of the concrete stays. During this operation 8 + 8, 1000 kN capacity jacks, were used to prevent the bursting of the prestressed concrete of the stays.

8 - Demolition of other 6,50 m of the ending concrete of the stays. During the phases from 1 to 9, the shoulders have been reinforced by means of injected Titan bars and of a concrete structure that made up also the necessary room for the post-tensioning operations.

9 - Re-building of the 6,50 m tract, using high resistance reinforced concrete.
The efficency of the adherence of the concrete to the naked old strands of the concrete stays, were checked with several laboratory tests. The tensioning force of the short tendons will be entrusted to that adherence.

10 - Installation of the "checks" and their fixing to the concrete extremities with 80 + 80 steel bars.

11 - Installation and tensioning of the 6 + 6 short tendons, step by step, through 8 stages.
At this point all the load of the bridge previously supported by the concrete stays, is transferred to the short tendons.

12 - Cutting of all the old strands.
Now the old concrete stays are completely separated from the deck.

13 to 18 - Reduction of stress in the short tendons and increasing of tension in the long ones, step by step.
At the end of these phases the tension in the two orders of tendons assumes the final value as well as the geometry of the bridge.

19 - Tendons injection

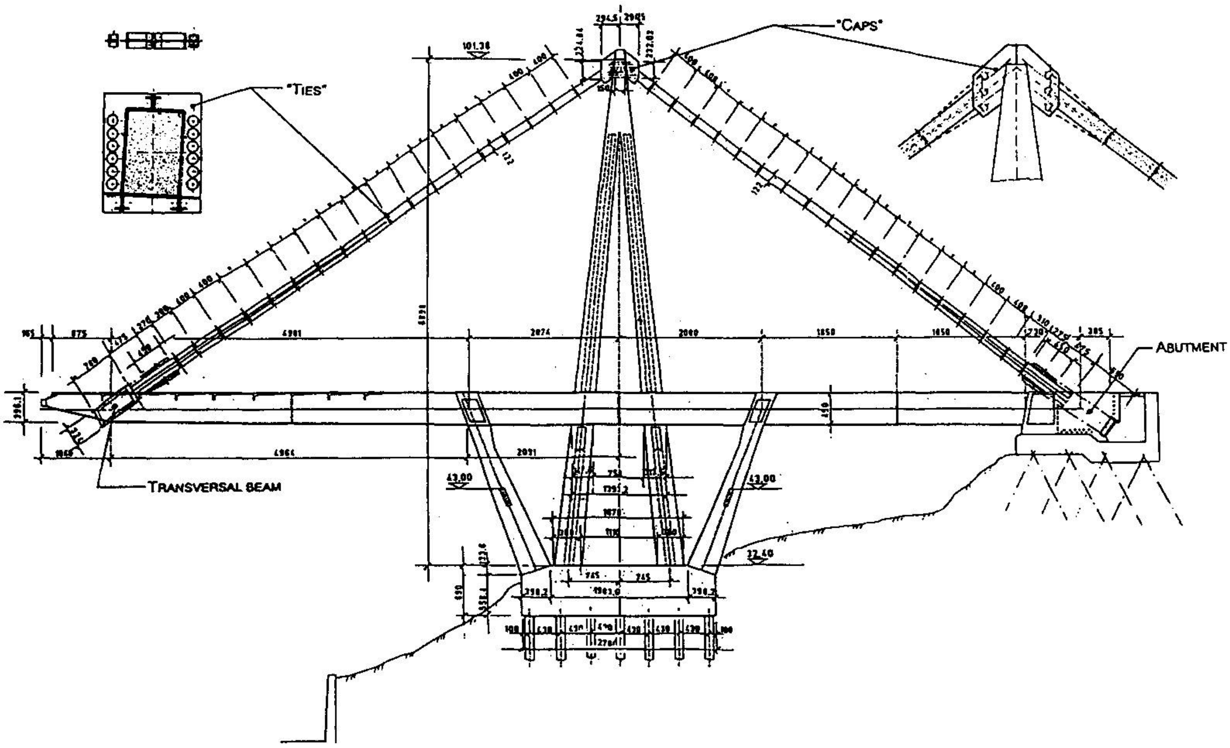
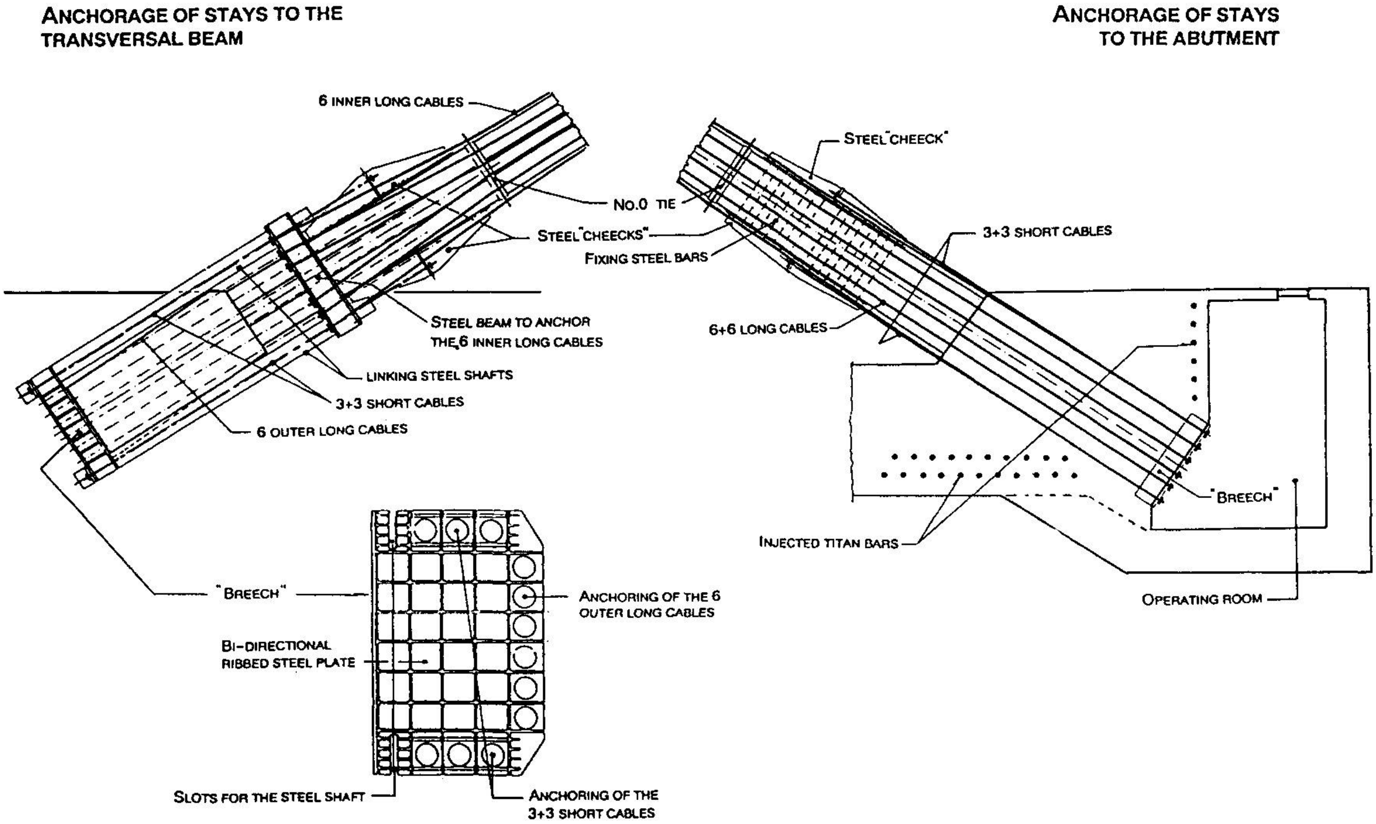


Fig. 1 General view of the repair works

Fig. 2 Details of the lower anchorages of the stays



20 - Final controll of the tensions

21 - Installation of the bi-directional restraint and concreting of the final extremity of the concrete-stays. A gap of 10 cm will remain. The bi-directional restraint has the scope to avoid the tail-vagging of the concrete stay extremities and to allow only the sliding of concrete stays along their longitudinal axis.

All the complex restoring works were made with the presence of the traffic on the bridge. Only during the installation of the steel "caps" and "breeches" and the delicate phases from 7 to 18 some limitations or reductions in traffic were applied.

The works were performed during 1992 - 1994.

Mr. F. Pisani of Roma designed the repair solution

Prof. F. Martinez y Cabrera of Milano verified the project and tested his execution
SPEA - Ingegneria Europea - of Genova was the Consulting by means of Mr. S. Bodrato and Mr. R. Rigacci
ALGA-PRECO Company of Milano realized the project with the technical management of Mr. A. Lodigiani
ISA - Costruzioni Generali SPA - of Genova made the building and complementary interventions for the installation of the stays.

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Full Scale Fatigue Tests for Stay Cable Systems

Essais d'endurance en grandeur réelle sur des systèmes à haubans

Ermüdungstests in vollem Massstab für Schrägseilsysteme

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SUMMARY

Full-scale axial and flexural fatigue tests were implemented for a large capacity site assembly stay cable systems. It was confirmed that the system fulfils every requirement of the PTI recommendation. Through the test, parametric data was acquired to evaluate the system's behaviour during service. In the flexural fatigue test, the bending angle, which represents the worst conditions induced by the wind, was determined; the test confirmed that this system could cater for all possible vibrations arising on the actual cables with the combined use of external passive damping devices.

RÉSUMÉ

Des essais d'endurance en flexion et à l'effort normal ont été exécutés sur des systèmes à haubans utilisés lors de montage sur de grands chantiers. Les essais ont confirmé que le système répondait à toutes les conditions requises dans les recommandations PTI. Pendant l'essai, différentes données paramétriques ont été recueillies afin d'évaluer le comportement des systèmes en service. Dans l'essai d'endurance en flexions répétées, l'angle de flexion a été déterminé; l'essai a confirmé la capacité du système à résister à toutes les flexions alternées que peuvent subir les câbles réels grâce à l'utilisation combinée d'amortisseurs passifs externes.

ZUSAMMENFASSUNG

Axiale und Biegungs-Ermüdungstests wurden in vollem Massstab für ein Schrägseilsystem an einer grossen Anlage durchgeführt. Es wurde bestätigt, dass das System alle Anforderungen der PTI-Empfehlungen erfüllt. Während des gesamten Tests wurden verschiedene parametrische Daten gesammelt, um das Systemverhalten bei der Arbeit zu beobachten. Beim Biege-Ermüdungstest wurde der Biegewinkel, der die schlechteste Bedingung darstellt, bestimmt: es wurde bestätigt, dass das System allen möglichen Biege-Vibrationen entsprach, die an den Kabeln mit der kombinierten Verwendung von externen passiven Dämpfungsvorrichtungen auftraten.

1. INTRODUCTION

The cables of cable stayed bridges are exposed to high intensity fluctuating loads, in axial and transverse directions, arising from the traffic loads and wind forces. A number of large scale cable stayed bridges have been constructed in recent years and full scale axial and felexual fatigue tests on the stay system are becoming important. Specification of axial fatigue test already exists such as PTI recommendations [1]. But the tester to investigate large scale cable systems conveniently did not exist so much.
A great deal of research on wind excited cable vaibration is reported in Japan. However, there exist no specifications for the flexural fatigue test at present.
The authors developed a large axial fatigue tester and implemented an axial fatigue test on a large scale cable system, Dywidag stay cable system. For the full scale flexural faitigue test, a test bending angle was assumed and loaded with continuous alternate bending on the system.

2. TEST SPECIMEN

The test specimen was the site assembly type system which was locally manufactured using JIS standard materials throughout. 15.2mm diameter strands were anchored at the wedge plate individually at both ends. The system provides a series of cable capacities up to 28,173KN nominal strength (Pu). Cable tension is transfered to the bearing plate through the ring nut and the shims (Fig.1). To obtain advanced fatigue resistance at the wedge, high strength grout is injected to minimize the fluctuating stresses reaching the wedge portion. Also, to avoid the sencondary bending stresses caused by the repetitive cable bending motions, an elastic damper is provided at the exit of the system. From the series of cable capacity, C61(Pu=15,912KN) cable system for the axial fatigue test and C37(Pu=9,652KN) for the flexural fatigue test were adopted respectively in consideration of the capacity of the cable and its frequency of actual application.

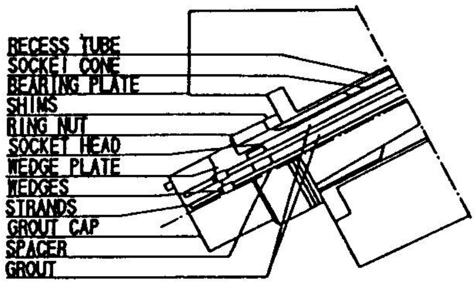

Fig.1 Cable anchorage system

3. AXIAL FATIGUE TEST

3.1 Tester

KASC (Kajima Stay Cable) Tester has the world's largest capacity at present, with a possible static load of 29.4MN and dynamic load of 17.6MN (Fig.2).

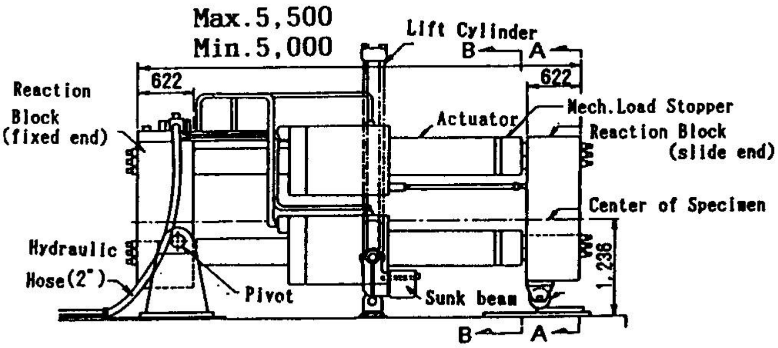

Fig.2 KASC Tester

This tester incorporates a pair of reaction blocks and three electrohydraulic actuators in between, and is designed to fulfill the requirements of the PTI recommendations. The tester can be inclined up to 60 degrees to enable to grout test specimen at the same inclination of actual cables (photo.1).

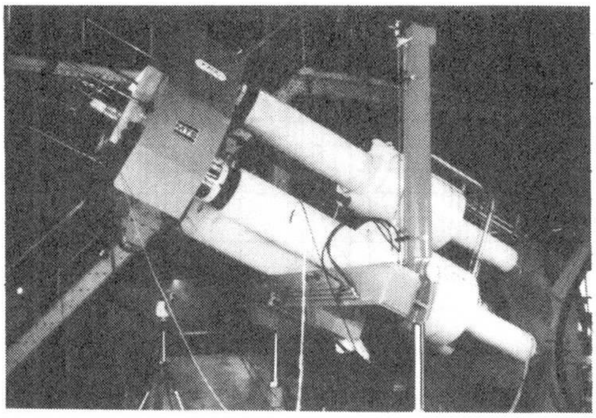

Photo.1 Inclined KASC Tester

Bridge Name	Max.Span	Fluctu.Stress
Shiraya Bridge	125 m	43 N/mm^2
Aomori Bridge	240 m	129 N/mm^2
Yobuko Bridge	250 m	119 N/mm^2
Usui Bridge	111 m	62 N/mm^2
Chichibu Bridge	196 m	104 N/mm^2
Tajiri Bridge	169 m	88 N/mm^2

Table 1 Typical fluctuating stress of P.C.Cable-Stayed bridge

3.2 IMPLEMENTATION AND EVALUATION

The axial fatigue test was implemented according to the PTI recommendations. The tension of the axial fatigue was set to 0.45Pu for the upper load and 160N/mm2 for the amplitude of fluctuating stress. After 2 million cyclic loadings, the load was statically increased to more than 0.95Pu as the PTI. In Japan the upper load is usually 0.40Pu which is 10% less than the PTI [2]. Fluctuating stress level for the test is also adequately safe even compared to stress caused by lane loading that is higher than actual (Table 1). According to the PTI, strand wire breakage after completing the test shall be less than 2% of total number and the system shall be safe at any portion during and after the 0.95Pu static loading.

3.3 PERFORMANCE AND RESULTS

Strand breakage was monitored by the highly sensitive accelerometers. The breakage, however, was not detected because of the meters monitored cracking of grout, too. After 2 million cyclic loadings, the test specimen was loaded statically up to Pu which is higher than PTI requirement of 0.95Pu (Fig.3). Precise inspection was made on every portion of the specimen and it was confirmed that there were no breakages except for metallic frettings and the system fulfills the PTI recommendations.

3.4 SIGNIFICANCE OF THE FULL SCALE TEST

In spite of the single strand has fatigue limit of 196N/mm2 (Fig.4), two serious metallic frettings arose under the fluctuating stress of 160N/mm2. The fretting location was the exit of the wedge plate where the polyethylene spacer is attached. After consideration of every dimension of the wedge plate and the polyethylene spacer, it was found that the cause of these portinal frettings was geometrical touch of strand with wedge plate, and fretting can be prevented with a small modification to the spacer hole diameter. To obtain advanced fatigue resistance high strength

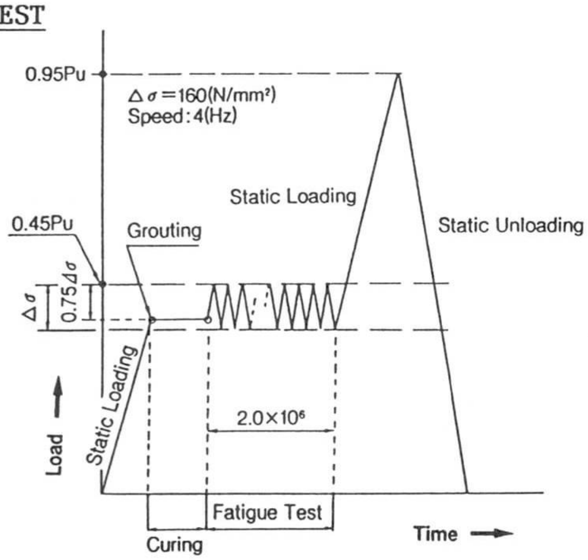

Fig.3 Loading pattern for axial fatigue test

grout is injected into the socket in this system. At the initial stage of cyclic loading, reduction ratio of the transfered fluctuating stress at the wedge was almost 50% and it remained so even after 2 million cycles. Although this bond effect might be affected by the state of the injected grout, for example the existence of air voids inside the anchorage system, it is supposed that the actual stay cables can get same bond effect because of the grouting work of test specimen was performed at nearly actual inclination of 28 degrees with KASC TESTER.

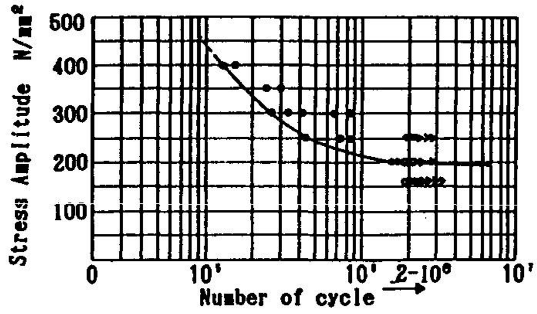

Fig.4 Voehler diagram for single system

4.FLEXURAL FATIGUE TEST

4.1 TEST METHOD

In general, many full scale flexural fatigue tests are performed to obtain an accurate flexural Voehler curve for that specific cable systems [3] and a huge budget is required to complete this. At present, if a certain amount of vibration is induced by wind on the stay cable, it is common practice to provide the cable with damping devices without numerical analysis of fatigue damage degree.
Therefore, in the test at this time it was aimed to investigate the soundness of system under the largest bending angle which can be reached after using general dampers against the possible maximum cable vibrations experienced in practice [4].
C37 cable system had been fixed on the reaction blocks on both sides and the alternate vertical deformation (up and down displacement) was given at the center of the cable (Fig.5). Considering the vibrations during construction period, the primary construction phase was implemented 2 million cycles with an average cable tension of 0.55Pu without the grout injection. After completing the primary construction phase, cable tension was reduced to 0.4Pu by adjusting the shims and subsequently the grout was injected. This completed phase was also continued with the alternating loads until 2 million cycles.

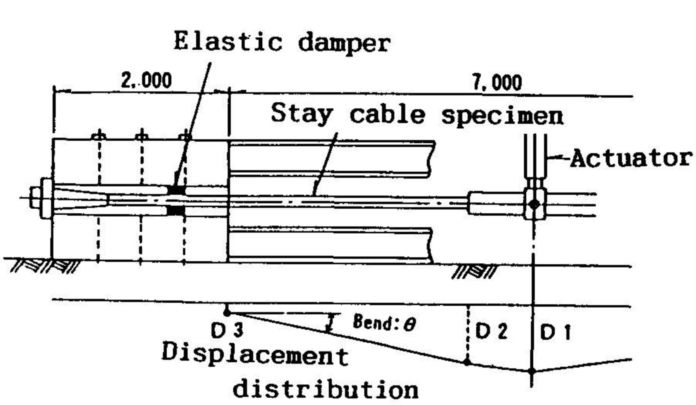

Fig.5 Half drawing of flexural fatigue test

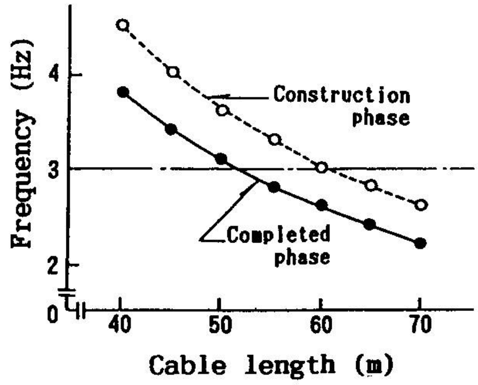

Fig.6 Cable length - frequency relationship

4.2 BENDING ANGLE

The major vibrations of stay cable are rain-vibration and wake-galloping. For large capacity cables, every individual stay is not bundled but is composed with a single stay. If multiple cables are aligned in parallel configuration with small spacing(within 2 ~ 5 times the cable diameter), wake-galloping would be the predominant cause of vibration [5]. Therefore, most of the non-bundled cable exibits rain-vibration, not wake-galloping. According to past site records, rain-vibration is said to be observed on cables whose frequency are less than 3.0 Hz. Preventive devices frequently applied in Japan for cable vibration are high viscous dampers, high energy dissipating rubbers, and diagonal connection ropes between cables. These devices are adopted selectively considering the mode, amplitude, type of vibration, appearance of the structure, etc, but it is reported that highly viscous dampers can reduce the rain-vibration to 0.3D or less in the case of a cable span 200m (where, D is the diameter of the stay cable) [4]. Therefore, for the C37 cable system, rain-vibration is thought to appear when the length exceeds 50 ~ 60m (Fig.6), and the bending angle at the entrance of the anchorage can be reduced to ±0.3D, namely less than ±0.1° when applied the damping device (Fig.7). In the test, considering the uncertainty of the preventive devices, test bending angle was determined to ±0.3° against the primary construction phase and ±0.27° against the subsequent completed phase.

4.3 TEST RESULTS

The specimen was removed from the tester and investigated after 4 million cycles of total loadings for the primary construction phase and the completed phase. Minute compression deformations were observed on the surface of strand wires at the elastic damper location. These compression deformations were apparently due to the mutual contact of the strands. There were no breakages or deformations in any other components. From the strain data, it was found that the strand behaviour was independent during the non-grouted condition and composite with mortar and other strands after the grout was injected (Fig.8). As predicted in the pre-testing analysis, maximum stress appered at the elastic damper location at the completed phase. It was also confirmed that only a little fluctuating stress reached to the end wedges at the completed phase (Fig.9). All strands positioned at the elastic damper, which were exposed to the most severe cyclic bending, were investigated those final static strength. Every strand had

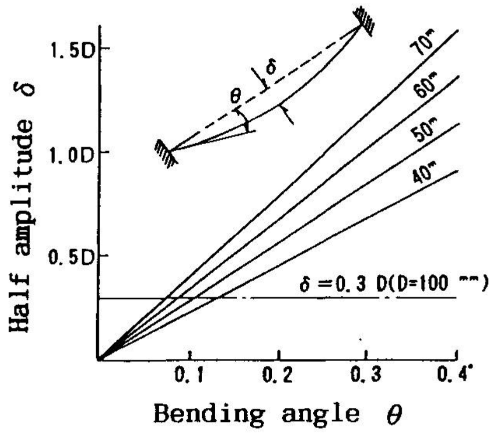

Fig.7 δ − θ relationship

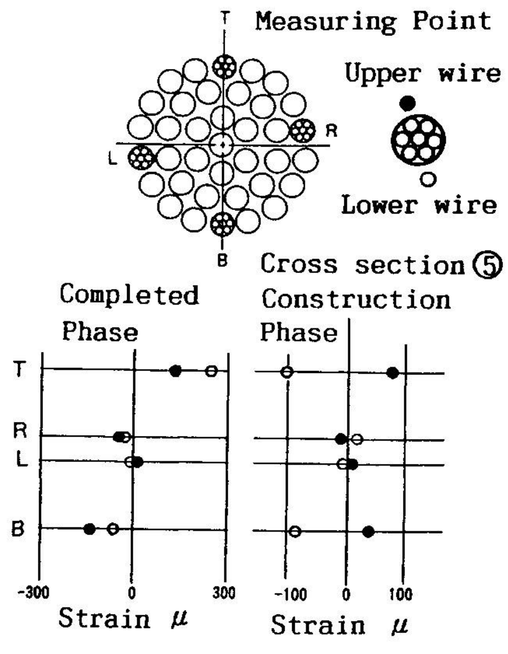

Fig.8 Strain distribution (N=10,000)

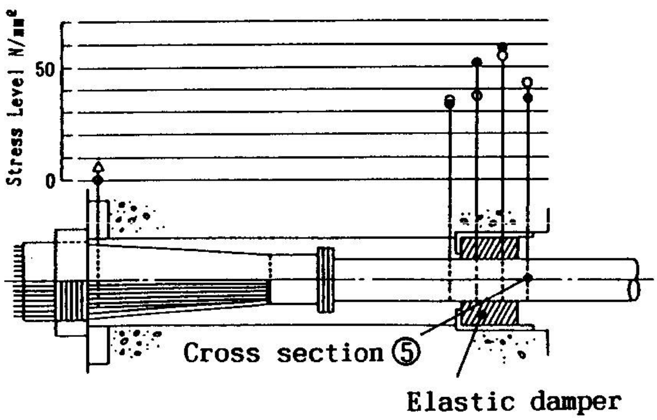

Fig.9 Stress distribution at elastic damper

Nominal Strength	2,550 kN
Max. Tensile Strength	2,610 kN
Min. Tensile Strength	2,560 kN
Average Strength	2,590 kN

Table 2 Tensile strength

retained strength in excess of the nominal strength (Table 2) and ductility was also far above that is required in the standard. Therefore, it could be concluded that this C37 stay cable system was safe for the test bending angle.

5.CONCLUSION

Large capacity stay cable sysytem of site assembly type was examined to find the soundness of the system with respect to axial fatigue and flexural fatigue. Using the world's largest capacity tester, the significance of the full scale test was recognized by the clarifying of the soundness of the individual components of the stay system including the bond effect, and improvement on undesirable factors in the system.
The flexural fatigue test was performed with the assumed bending angle which was introduced based upon the performance of presently existing damping device. Throughout the test, it was found that this stay system was sound and offered an adequate safety with respect to the bending angle.

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Investigation of Obsolete Structural Elements and Retrofit of Old Steel Structures

Recherche sur des éléments affaiblis et réparation d'anciennes constructions métalliques

Forschung an überholungsbedürftigen Bauteilen und Reparaturen an alten Stahlkonstruktionen

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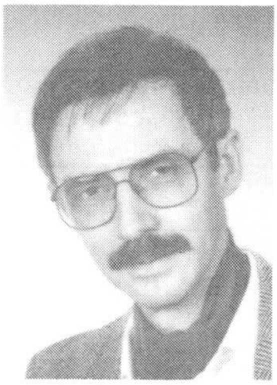


Ulrich Morf, born 1942, obtained his engineering and doctoral degree at ETH (Swiss Fed. Inst. of Technology) in Zurich. After being a design engineer in a steel construction firm, he started his testing and research activity with EMPA. Since 1971 he is the head of the Metals Technology and Joining Dept.

SUMMARY

Examples of the state-of-the-art of structural safety assessment of old steel elements and obsolete structures are described based on the experience in the research laboratories and field work. Current studies on fracture behaviour of old structures and steels, and repair work are presented to define design and strengthening strategies and quality assurance and testing. This will assist in retrofitting such structures.

RÉSUMÉ

Des exemples illustrent l'état de la technique d'analyse de la sécurité de vieux aciers et de structures dépassées, sur la base d'expériences de laboratoire et de chantier. Des études actuelles sur le comportement à la rupture des structures, des aciers et des réparations sont la base pour élaborer l'étude, les stratégies de renforcement, de l'assurance de la qualité et des essais concernant la réparation de ces structures.

ZUSAMMENFASSUNG

Beispiele für den Stand der Technik der Sicherheitsanalyse an alten, überholungsbedürftigen Stahltypen und Konstruktionen werden vorgestellt anhand von Erfahrungen aus Forschungslabor und Feldarbeit. Aktuelle Bruchuntersuchungen an alten Strukturen, Stählen und Reparaturstellen werden vorgestellt und Entwurf, Reparaturvorgehen und die begleitende Qualitätssicherung und Verfahrensprüfung für die Sanierung erläutert.

1. OBJECTIVES OF REPAIR AND STRENGTHENING OF STRUCTURES AND JOINTS

The scope of this paper is concentrated on the safety and fracture toughness of old steel structures with respect to the influence of static load, strain rate, temperature and influence of weldments and environment. In some cases the influence of fatigue and embrittlement is mentioned. The determinant structural element that initiates repair work are in most cases the connections between main girders or floor beams and specifically tensile elements or strings.
Therefore strategies of repair and retrofitting of structures have to consider multiple parameters influencing the technology of repair beside the criteria for structural safety.
The figures given in a first overview are examples which combine the parameters of mechanics (see fig. 1) with the ones of technology and practise how to repair all kinds of joints with rivets, bolts or high strength bolts (HS) and by welds or other joining techniques.
The last column shows recent examples, if the goal of retrofitting is a fail-safe design. This can be achieved by using additional structural elements as reinforcing plates or sheets (fibres or composite materials) or prestressing tendons as used extensively for concrete bridges.

Method of Retrofit Case,Technology ↓ *service condittions*	**splices „soft repair“**	**welded joints „cut“ and „paste“**	**strengthening, external prestressing („fail safe“element)**
Riveted joint to be reinforced *Static load*	replace by HS-bolts friction type	reinforcing by fillet weld or buttweld	external cables fixed with bolted adapters
Riveted Joint to be reinforced *at low temp.*	replace by HS-bolts fitted or friction type	welding not recommended	external cables fixed with bolted adapters
Riveted joint to be reinforce. *Fatigue load*	replace by HS-bolts fitted or friction type	welding not recommended	external cables evtl. composite laminates
Welded joints to be reinforced	repair (Hi-quality and weldability), NDT	replacement after NDT and new WPS	external cables or composite laminates

Fig.1. Goals and technology for repair of steel structures

The aims of repair and strengthening of old structures are to extend their lifespan, but especially for old structures the goal is to redesign the structure such that well-defined safety is achieved (see also fig. 2.4). If old iron material is present, a soft reinforcing technology of splices is recommended to avoid the effects of cutting and welding. Melting and metallurgical effects may cause cracks in the lamellar structure of the steel. To avoid load redistribution effects in a non isostatic structure e.g. in a continuos beam repaired by welding, it may be recommended not to change the static system and to do only minor welding work (less internal stresses) in a old structure. In this context old codes may be helpful to study the old design philosophy (e.g. [1]).
It is a must in any redesigned joint and reinforcement, to assess the quality of the old material before and after the repair by means of actual non destructive testing (NDT) and for any new additional structural element as well. One favourable property of old steel is (type Tomes steel or wrought iron), their low sensitivity to corrosion. However, poor maintenance and therefore damage by corrosion are still the main reason for the repair or demolition of most old steel structures. Note, that the cases reported here, are structures manufactured about 100 years ago.

2. PRELIMINARY INVESTIGATIONS, NON DESTRUCTIVE TESTING AND SMALL SCALE EXPERIMENTS

2.1 Preliminary investigation and strategy

An analysis and retrofit program may cover the following items and steps of operations:
- Investigation of damage and extrapolation of risk to the end of the extended service life: cause and degree of damage, remaining period and failure scenario, structural behaviour and consequence of repair for safety, remaining service life, timing, maintenance, inspection.
- Actions of retrofit program, design and technical specifications.
- Environmental studies for adequate corrosion protection of surfaces and strengthening elements.
- Choice of methods "soft repair" or welded joints or strengthening and reinforcing (as fig. 1): Technology of joints with „soft repair“ methods do not affect the ductility and plastic properties and embrittlement is avoided; another case is the effect of welding which often causes reduced toughness in the HAZ or cracks in zones of laminated concentrations of phosphorus (P) and

sulphur (S) (see fig. 2.1 d).
Strengthening by external prestressing or by bonded advanced composite sheets, where experienced methods are available now (see par. 4 and [7] for composites).

- Execution, quality management, non-destructive testing and documentation: Specification of repair, materials and testing, WPS and NDT criteria for welds.
- Consequences for extended service, NDT and inspections by owner and consultants

2.2 Preparation of structures and inspection of critical zones

Accessibility to the specific joints of all tensile members as well as adequate surface grinding before any inspection and non destructive testing are mandatory.
If ultrasonic testing (UT) is used there may be no reliable back-wall signal due to the lamellar structure of old steels. The NDT procedures recommended are visual testing (VT), magnetic particle- (MT) or penetration testing (PT). Experienced inspectors are able to detect small cracks by VT, but the use of a lens and PT may be necessary if steels near rivet heads and bolts are inspected as in fig. 2.1 and 2.3 (lit. [2]). These figs. show other typical damage, corrosion pittings and cracks due to on the site welding, which can be caused by improper repair work. Therefore quality assurance of any repair must include VT, PT or MT and UT inspections if possible.
Investigations and welding qualification programs (as WPS explained in [4]) with fine grain steels for power plants are good examples to pre-qualify any old steel for its fracture and failure behaviour.

2.3 Fracture mechanics analysis (see [3] [4])

An investigation of Swiss Railroad (fig. 2.2 in [2]) including results of EMPA, shows the temperature transition curves of CVN-toughness of wrought iron and mild Tomas-steel (period 1890 to 1940, covered in [1]). This latter steel contains impurities as well and has a typical upper and lower shelf of toughness, compared to the wrought iron with its typical laminated carbon inclusions. The "old iron" shows less temperature sensitivity but low CVN-energy as well. Both steels show low toughness, they are sensitive to strain rate and temperature- and ageing effects may appear, if cold forming is present. Nevertheless due to multiple crack branching the K_{IC} of old steels may be surprisingly high (see fig. 2.3 and [3]). Our results showed, that at increasing load, close

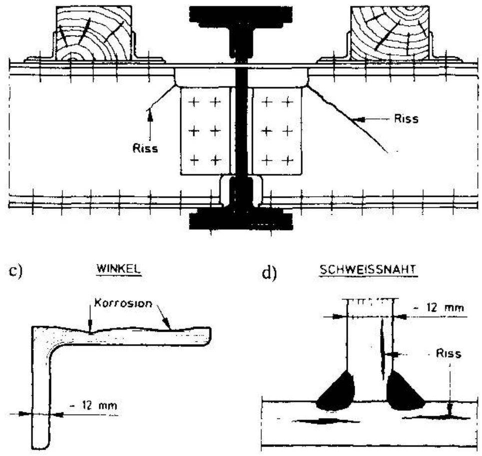


Fig.2.1 Examples of possible cracks and notches
above: cracks in web due to local deformation or service load
below left: corrosion pitting between corner and plate
below right: influence of welding in web and flange due to filet weld
(Stahlbau 58 (1989), Brühwiler, Hirt, Morf, Huwiler)

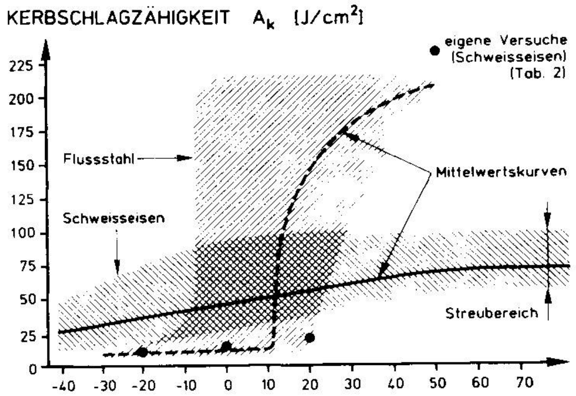


Fig.2.2 Charpy-V-notch fracture toughness Ak(J/cm2) vs.temperature in °C (transition) for wrought iron and old mild steels
Schweisseisen=wrought iron,
Flussstahl=mild steel (1890...1940)

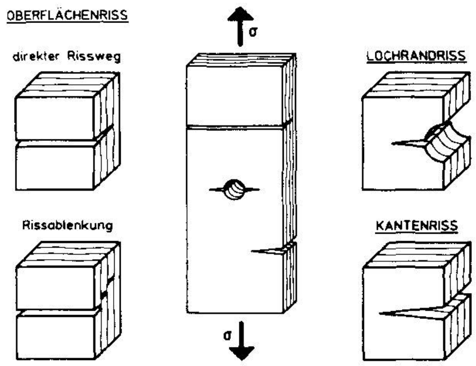
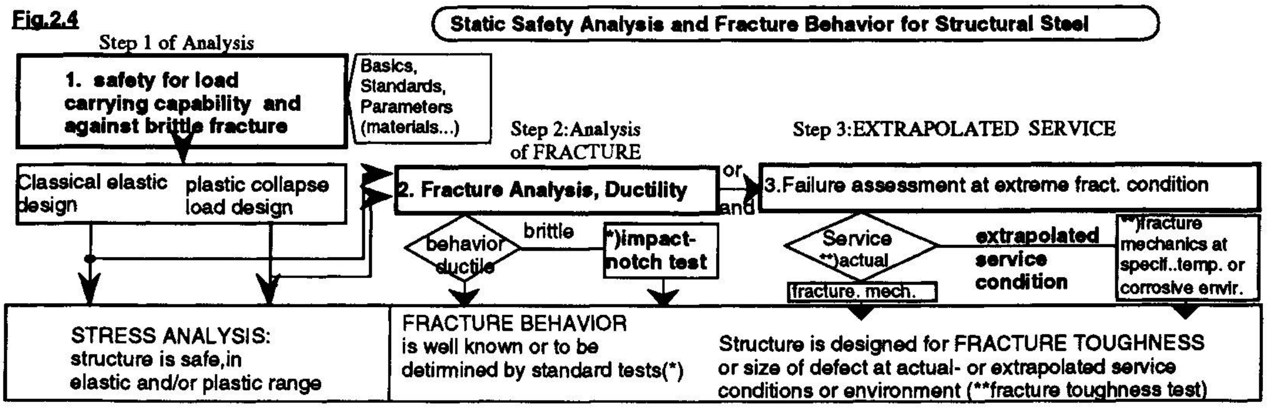


Fig.2.3 Orientation of cracks found in anisotropic steels as wrought iron or old mild steels
left side: surface cracks with multiple deviation
right side: edge crack or corner crack in hole

to the yield point (R_p), small cracks may not trigger spontaneous fracture. Wrought iron plates have been tested up to R_p with 10mm edge-cracks at holes. To redesign such structures a combined „Static safety and fracture behaviour analysis" is necessary according to fig. 2.4 (complete flow chart see [4] and design curve in [5]). For extrapolated behaviour at extreme service and environment fracture mechanics testing could be worthwhile, since inexpensive small– sample CVN-test methods (HR6-method in [5]) are available in practise.
To reach the optimal load capacity in a cracked plate—that means the status of plastic collapse starts before unstable crack growth—the rule described in [5] for the required fracture toughness is: (1) $\mathbf{K_{req}=1.38\ R_p\sqrt{t}}$ where
- R_p is the actual yield point of the representative element (thickest representative flange or wall) at extreme service conditions as local strain rate and lowest service temperature,
- t is the thickness of the same structural element.

2.4 Mechanical testing with small samples
In any case we suggest full size testing , if scale factor problems (t >>10mm) are present; in such tests extreme thickness and strain rate and lowest temperature may be simulated. In the case of a riveted structure it is realistic to use a choice of the thickest representative elements, corners and plates and design a testing program similar to the WPS for pressure vessels (SVTI 505, Swiss board for techn.Inspections or ASME in USA). In this test plate, sets of small specimens are used as for WPS. In our case precracked CVN samples are used to check whether the condition of fig. 2.4 for step 3 is fulfilled, means if for a given crack situation the stress intensity is below K_{IC}:
(2) $\mathbf{SIF < K_{IC}}$ This safety equation for the fracture toughness requirement (formulas (1) and (2)) SIF < K_{IC} < K_{req}) is traditionally replaced by the simple check of „ductile behaviour" (step 2 in fig. 2.4) by means of welded bend test specimens and CVN-tests at the lowest service temperature. For actual applications we recommend here to apply Eurocode 3 (1994) Annex C. In this provisional code for structural steel, the minimum temperature for service T_S(°C) is given for minimum fracture toughness (K_{IC}), test temperature and temperature shift due to shock:
(3) $\mathbf{T_S(°C) = 1.07\ T_{CVN\,27J} + \beta + \Delta T[de/dt]}$ where $\beta = 100(\ln(K_{IC}) - 8.06)$
- The term ΔT[de/dt] is negligible since it depends on the global strain rate [de/dt in s^{-1}] which is usually close to zero. For medium shock loads use $\Delta T[de/dt]=2100(de/dt)$.
- $T_{CVN\,27J}$ is the test temperature at which the CVN-energy is 27J. In the case of some old steels the result for T_S (°C) may be such, that retrofit by welding is not possible!

Fig.2.4



3. ENGINEERING OF IMPROVED JOINTS AND REPAIR TECHNOLOGY

3.1 Methodology for specific repair of old steels
The analysis of old steel structures and possible repair strategies combines the methods of inspecting damaged structures and procedures of production as WPS for bridges and pipelines. For concrete structures a RILEM draft [6] covers special questions about corrosion in this context.
In cases of damage to steel structures, a checklist for some types of connections is shown in fig.1.

3.2 Case studies of repair technologies for buildings and bridges
•The first example of an old low carbon iron **arch string bar** (35mm*52mm), manufactured in the 17th in Engadin/Switzerland, was to be repaired in a church. The objective was to design a fail safe

splice, in addition to the repair by butt welding. The latter was successfully accomplished. Details, material data and welding procedure specification are given below:

description of steel: low carbon iron with high contents of manganese and phosphorus (P:0.1%) tensile strength: 230 to 330 N/mm^2, CVN-energy (in HAZ): 50 and 220J

welded specimen : high variation of hardness HV 100...260 (in the HAZ: HV100...180)

WPS (see fig.3.1): electric arc welding with basic electrodes 2.5/3.25, 13 passes. Some cracks in the HAZ had to be repaired in zones of high contents of P and S.

•**Riveted joints transformed in slip resistant joints** with high strength bolts are good examples of retrofitting methods and typical for railroad bridges. Two cases reported from Swiss railroad are old truss systems made of wrought iron in 1875 with additional elements of mild steel. In most cases gusset plates spliced with diagonals or beams had to be reinforced. The aim was to replace rivets by slip resistant high strength bolts. Based on German tests, the improvement of fatigue strength and of the slip behaviour is evident (Fisher [8] p.238).
Below the joint or splice is described first and then the method of strengthening and inspection with points to observe (slides by E.Brühwiler Swiss Rail/ETHL):

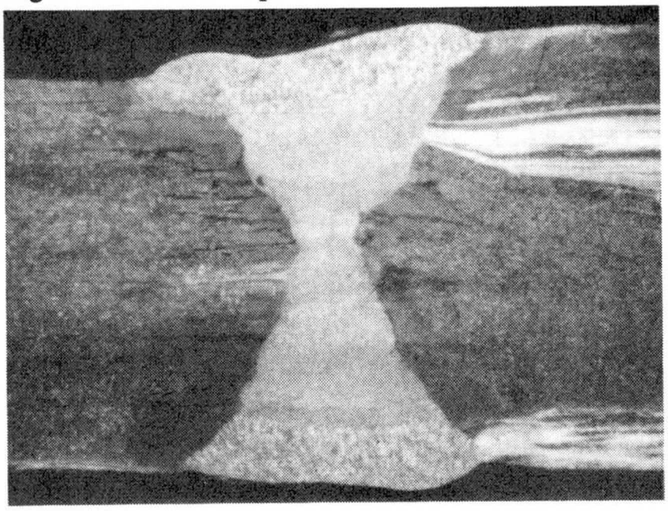
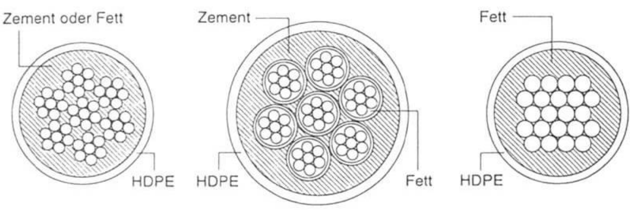


Fig.3.1 Repair of a Arch String in a Church 17th
Material: low carbon iron w.inpurities (high contents P, Mn)
WP Spec.(accord. EN287): electric arc weld, manual (111)
type and pos.: butt weld, X-shape 35mm (BW)/ vertical (PF)
electrode /diam.: Oerlikon Spec. /2.5...3.25mm at 70...90A



bundle of strands protected by cement or grease | bundle of individually greased and plastic-sheathed strands | bundle of wires protected by cement or grease

Fig.4.1
Cross-sections of three types of exterior cables
(SI+A 21/94, T.Vogel)

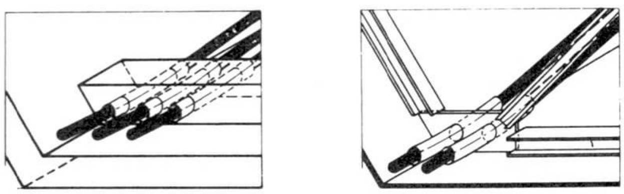


Fig.4.2
Saddles and deviations in concrete and steel girders
(VSL news letter II/89)

<u>Non symmetric bow-truss bridge "Linth":</u>

steel / splices: bow with wrought iron flange / reinforced by lateral filet welding in 1930 and attachments of new wind bracings.
Some secondary elements fixed by filet welding on surface of riveted bow.

inspection, observation: In the case of wrought iron, welding between edges of plates is possible, but welds on surfaces are likely to produce lamellar cracks and tearing.. Ribs on plates (as rain drains) should be checked for cracks in HAZ next to the fillet welds. Welds between edges of plate material loaded by shear seemed to behave well.

<u>Continuous truss beam "Rhine":</u>

steel / splices: wrought iron / extension of gusset plate with combined action of rivets and new high strength bolts. Extensions of diagonals (attached to gusset plate) by lateral but welding.

inspection, observation: Butt welding was avoided if possible. Load transfer between elements and fillet welds was designed such that shear along edges of plates was predominant. The main points to check or retrofit were obsolete and locally corroded splices and gusset plates. Experience of old road bridges in Basle showed, that the safety and stability of corroded plate girders (wave shaped deformation) is reduced due to expanding corrosion products.

4. RETROFIT OF STEEL STRUCTURES BY EXTERNAL PRESTRESSING

External prestressing was used since the beginning of prestressed concrete in 1936, later also as a tool for strengthening of structures. For special cases of retrofitting old steel bridges any type of ropes and locked coil cables have been used as the Swiss example of Aarwangen shows. The scope of applications is wide, especially if today's possibilities of external prestressing technologies and the specific cable sheathings (see fig. 4.1) are applied (draft doc. FIP [9]). The external tendons are advantageous for strengthening steel structures, beams and trusses, below beams or inside of structures as in the recent example of the "Bois de Rosset" bridge (see fig. 4.2). Particular solutions are possible if the external tendons are prestressed such that stresses in tension in the main structure are minimised (against brittle fracture) and the external tendons are counteracting the live loads.

Requirements for structure and materials of external tendons:
Tendons exposed to live loads and environment are to be protected against corrosion and damage. In order to gain full advantage from exposure and easier accessibility the following is important:
- Design of structure, cables and attachment of fittings shall allow inspection and monitoring
- In special cases tendons and anchorages must be restressable, detensionable and replaceable.
- To obtain reliable quality of the protection, three examples of well known permanent corrosion protection methods are shown in fig.4.1 which are state of the art of the respective producers.
- The steel (according to prEN10138) may be supplied with a permanent corrosion protection in the factory, depending on the product as galvanised wire or strand, greased waxed or otherwise soft protected and plastic-coated monostrand or Epoxy-coated wire, strand or bar.
- In the deviators high lateral pressure occurs. Test with strands have shown, that linear pressure of 600kN/m can be reached without noticeable reduction of tensile strength of the prestressing steel. However, the influence of fretting corrosion between steel and pipes (if used as sheathing) should be investigated in a specific fatigue test. In this respect locked coil bridge cables, investigated by EMPA, also showed this sensitivity between the outer and the second layer of wires.
- In case of riveted structures it is recommended to design special adapters to be bolted to the main structure or use symmetrically welded anchorage devices, if weldable gusset plates are present.

DEFINITIONS

CVN Charpy-V-notch impact bend specimens (if precracked basis for evaluation of K_{IC})
HAZ heat affected zone in welds (problems arise if impurities are melted by weldment)
HR6 assessment according to Cent. Electricity Generating Board, CEGB 1986 doc. R, HR6
K_{IC} fracture toughness (crit.factor of stress intensity)
K_{req} required fracture toughness
NDT non destructive testing (for definitions of MT, PT and UT see chapter 2.2)
WPS welding procedure specification (e.g. according natl. codes or standards)
SIF stress intensity factor

The author is thankful to his co-workers H.J. Schindler, M.Harzenmoser, T.Meier and R.Primas.

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New Method for Connecting Concrete Slabs to Steel Bridge Girders

Nouvelle méthode de liaison de dalles avec des poutres métalliques de pont

Neues Verbindungsverfahren bei Verwendung von Betonplatten in Stahlbrücken

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SUMMARY

A new process developed for connecting concrete slabs and steel girders is useful in replacing traffic-damaged concrete slabs with precast concrete ones. According to this process, the joint called the box dowel permits level adjustment by screws, and ensures connection with the steel girders by welding. This paper deals with experimental results of the performance and rigidity effects of the box dowel, and introduces an application of the process to an actual bridge.

RÉSUMÉ

Un nouveau procédé de liaison de dalles et de poutres en acier s'avère utile pour remplacer les dalles détériorées par des dalles en béton préfabriqué. Ce procédé de liaison par goujons, permet de régler le niveau au moyen de vis et d'assurer la connexion avec les poutres en acier par soudage. Cette étude décrit les résultats expérimentaux en matière de performance et de rigidité des goujons . Une expérience pratique est mentionnée pour un pont existant.

ZUSAMMENFASSUNG

Ein neues Verfahren, das für die Verbindung von Platten mit Stahlträgern entwickelt wurde, ist geeignet, um Betonplatten, die durch starke Verkehrsbelastung beschädigt sind, mit vorgefertigten Betonplatten zu ersetzen. Dieses Verfahren, ermöglicht die als Boxdübel bezeichnete Verbindung Höhenlageneinstellungen mit Schrauben vorzunehmen und gewährleistet durch Schweissen die Verbindung mit den Stahlträgern. Die vorliegende Abhandlung beschreibt Versuchsergebnisse über die Leistung und Steife des Boxdübels und führt ein Anwendungsbeispiel bei einer tatsächlichen Brücke auf.

1. INTRODUCTION

Recently increases in traffic volume, in the sizes of vehicles and in the loads carried by them have been bearing harder and harder on the existing road steel girder bridges, and the number of reported cases of damage on their concrete slabs are increasing. Whole replacement of concrete slab must be carried out quickly and safely in a manner to minimize the period closed to traffic flow, and one of solutions to this has been the use of precast concrete slabs. The authors have developed a new jointing process, the connection of slabs and steel girders. This process called box dowel which permits level adjustment by screws and ensures connection with the steel girders by welding. This paper deals with experimental results of the performance and rigidity effect of the box dowel according to this newly developed jointing process, and also introduces an application of the process to an actual bridge.

2. BOX DOWEL

Figure 1 illustrates a newly developed box dowel and its connection to the steel girder. As shown, the box dowel consists of an outer cylinder embedded into a precast concrete slab and an inner cylinder that is threaded into the outer cylinder at site. After level adjustment of the precast concrete salb, the bottom end of the inner cylinder is integrated with the steel girder by welding. In practice, the clearance between the concrete slab and steel girder is filled up with non-shrink mortar. From the design point of view, however, the bond stress of mortar is disregarded, and all the shear force acting between the concrete slab and steel girder is assumed to be taken up by the box dowel. The mechanism of transmision of shear force from concrete slab to steel girder by the box dowel is illustrated in Figure 2.

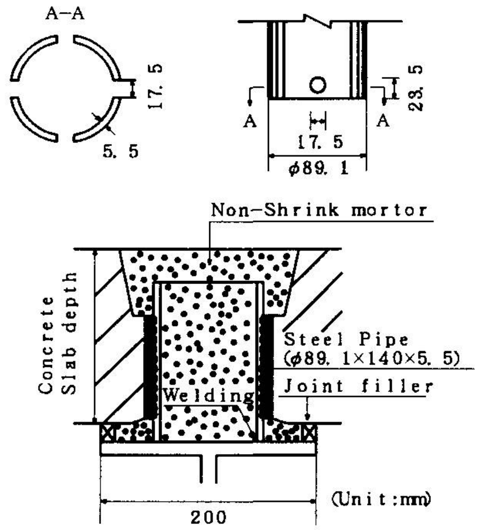

FIG. 1. SHAPE OF BOX DOWEL AND JOINTING METHOD

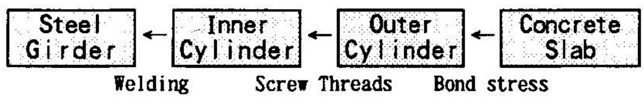

FIG. 2. MECHANISM OF TRANSMISION OF SHEAR FORCE BY BOX DOWEL

3. SPECIMENS AND TEST METHODS

According to a road bridge engineering manual available in Japan, the shearing strength per dowel is determined by the shearing capacity of a dowel itself whitch is a function of the type and dimensions of the dowel, or by cleaving strength or bearing strength of concrete. For the purpose of direct measurement of the shearing capacity per box dowel, the authors conducted a punching shearing test according to the loading arrangements illustrated in Figure 3. Specimens consisting of H-beam and concrete with foam styrol included in between were prepared so that all the shearing force acting on the specimen would be borne by the box dowel, and static loading test and fatigue test were carried out on five specimens, respectively. High-strength concrete with a standard design strength of 500 kgf/cm² was used for specimens. In the loading tests, the box dowel was given a shearing force by applying a uniform axial load on the center of the H-beam as illustrated in figure 3.In the static loading test, the loading hysteresis cycle was 0→20 tf→0→40 tf→0→60 tf→0→Max. load. Static rupture load, Pu(tf), and the load vs.Penetration curve were determined, and the dowel slip displacement caluculated.In the fatigue test,load was changed sinusoidally at a cycle of 150 times per minute from minimum value (10% of static rupture load) to maximum value (36, 42, 45, 48% of static rupture load)to determine the fatigue life (i.e., the number of cycles till the dowel fails). The changes in slip displacement of dowel were also measured using a dial gauge under minimum and maximum load conditions.

4. TEST RESULTS

4-1. Static loading test

In the static loading test, all failures of specimens took place in the form of shear fracture with slip at the welds of the box dowel, and were accompanied by the deformation of inner cylinder. The load vs. displacement curves measured in the static loading test showed almost the same tendency. An example of measurements taken with specimen S-4 is given in Figure 4.

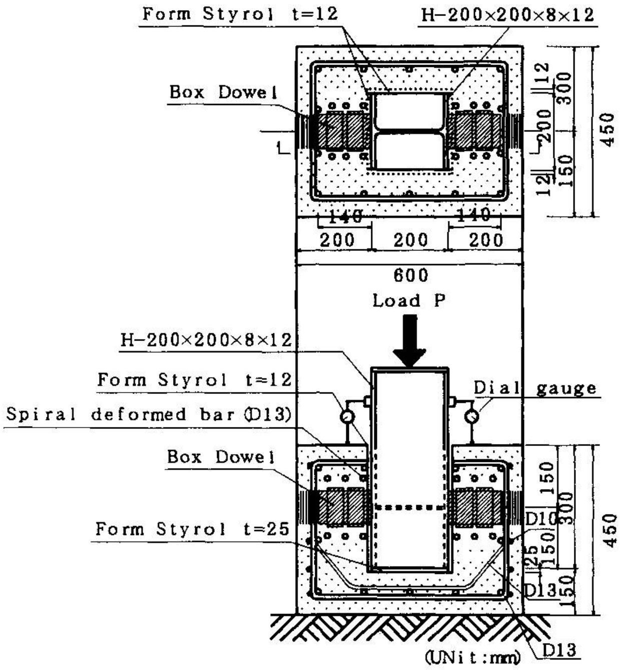

FIG. 3. SHAPE AND SIZE OF SPECIMEN AND LOADING METHOD

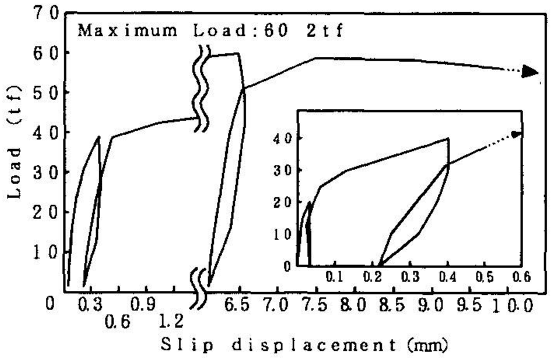

FIG. 4. LOAD VS. SLIP DISPLACEMENT CURVES (S-4 SPECIMEN)

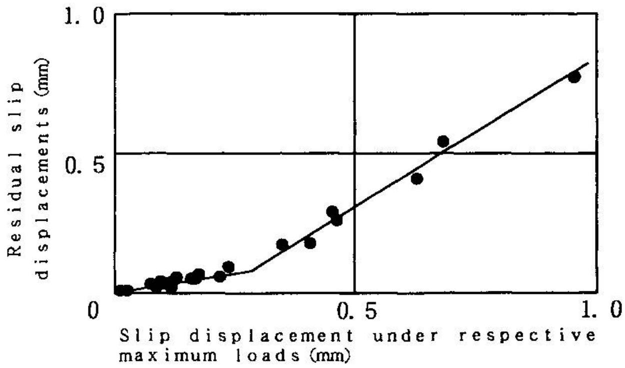

FIG. 5. RELATIONSHIP BETWEEN SLIP DISPLACEMENT UNDER LOAD VS. RESIDUAL SLIP DISPLACEMENT

It is evident from the measurements shown that the slip displacement of box dowel follows elastic behavior up to nearly P = 20 tf (τs = 700 kgf/cm²) where the slip displacement is about 0.05 mm. The rates of increase in slip displacement increase with load if P is in excess of 20 tf, and the maximum load (static fracture load) is observed at slip displacements ranging from 5 to 10 mm. Figure 5 shows the relationship between slip displacements under loads (P = 20 tf, 40 tf, 60 tf) and the residual slip displacements after removal of loads. The relationship between slip displacement under load and residual slip displacement after removal of load shows that the residual slip displacement increases sharply when the loading slip displacement exceeds 0.3 mm, suggesting that the load that causes a slip displacement of 0.3 mm will be a limit load for the functional performance of a composite system under static load conditions. Namely, the loads that develop a slip displacement of 0.05mm and 0.3 mm, respectively, are considered the limit loads within whitch the box dowel is structurally allowed to change its rigidity behavior. These limit loads were determined from the load vs. slip displacement curves of test specimens, and are given in Table 1 together with the shearing fracture strengths.

Specimen No.	Shearing strength per box dowel (tf)		
	Slip displacement 0.05mm	Slip displacement 0.3mm	Rupture load
S - 1	12.9	20.3	30.9
S - 2	9.4	17.8	33.6
S - 3	8.3	21.0	27.5
S - 4	11.0	18.3	30.1
S - 5	13.0	20.2	32.9
Average	10.9(750)	19.5(1350)	31.0(2150)

The values in parentheses refer to average shear stress intensity (kgf/cm²).

TABLE 1. SUMMARY OF STATIC LOADING TEST RESULTS

Specimen No.	Max.load ratio(%)	Number of cycles(x10⁴)		
		$N_{0.3}$	$N_{1.0}$	Nu
D - 1	36	91.8	125.9	140.0
D - 2	42	13.3	21.6	25.45
D - 3	45	0.62	3.95	10.54
D - 4	48	5.08	10.24	10.45
D - 5	48	2.82	5.65	7.21

TABLE 2. SUMMARY OF FATIGUE TEST RESULTS

4-2. Fatigue test

The sizes of repetitive load applied to test specimens D-1 through D-5 are shown in Table 2. As an example, the number of loading cycles vs. slip displacement curves are shown in Figure 6. As the changes in slip displacement due to repetitive loading are ascribable to changes in residual slip displacement, the relationship between the number of loading cycles and the average loading slip displacement (i.e., the average value of the slip displacements under minimum and maximum loads) is shown in Figure 7.

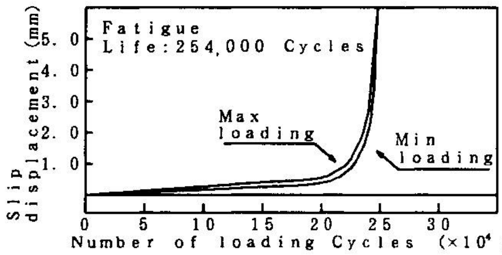

FIG. 6. NUMBER OF LOADING CYCLES VS. SLIP DISPLACEMENT
(DURING MIN. AND MAX. LOADING)

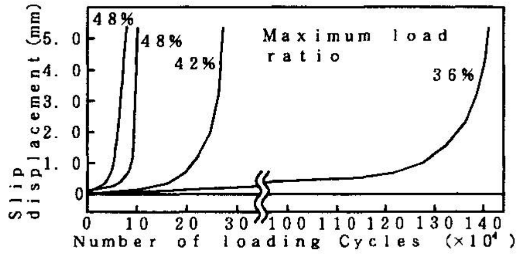

FIG. 7. NUMBER OF LOADING CYCLES VS. SLIP DISPLACEMENT CURVES
(AVERAGE LOADS)

As will be clear from Figure 7, almost every test specimen shows slip displacements proportional to the number of loading cycles if the loading cycle ratio is within about 70% where the slip displacement is about 1.0 mm. The fact that the rate of change in slip displacement increases when a structure using box dowels is subjected to repetitive loads suggests that the rigidity effect changes irrespective of whether the box dowel fails or not. It is therefore considered that the number of cycles ($N_{1.0}$) for a slip displacement of 1.0 mm may be taken as the composite functional life under repetitive loading conditions. Accordingly, $N_{1.0}$ was determined from the number of loading cycles vs. slip displacement curves, and $N_{0.3}$ number of loading cycles run to reach a slip displacement of 0.3 mm taken as a slip displacement limit life under static loading conditions was also determined. They are shown in Table 2. Given in Figure 8 are S-N curves showing slip displacement life ($N_{0.3}$), functional life($N_{1.0}$), and fatigue life (Nu) in relation to maximum loading ratio.

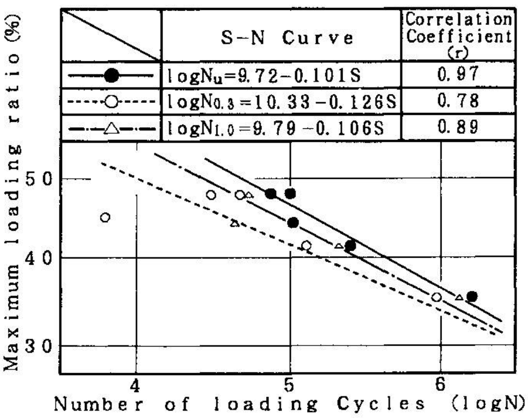

FIG. 8. S-N CURVES

5. AN APPLICATION EXAMPLE OF THE NEW JOINTING METHOD TO AN ACTUAL BRIDGE

It was determined to apply the newly developed jointing method to Kamishima-bashi Bridge, a simple composite steel plate girder bridge, situated in Sasebo City, Nagasaki Prefecture. It had reinforced concrete slabs with a thickness of 18 cm. Constructed in 1968 with a design load of TL-20, length (L) of 22.0 m and width (B) of 18.45 m, it was classified as a first-class bridge. It concrete slabs were damaged serious by heavy traffic, and had to be replaced with new precast concrete slabs according to the box dowel process. Stress frequency and deformation of main girders under actual service conditions before and after slab improvment were measured in the safety assessment of the bridge. Stress frequency of main girders was measured using a histogram recorder for 24 hours under actual service conditions, for the purpose of evaluating the safety of the bridge. The technique used for safety assessment of the newly developed dowel joint was called the peak-valley method in whitch the frequencies of extremes (peaks and valleys) of stress (strain) waveforms developed by passing vehicles within a 24-hour period were counted for each specific stress level. Before and after slab improvement, the stress frequency was measured at the same midspan position on the underside of the lower flange of each of the main girders G1 through G7. The maximum values of the stress intensity and deflection developed in each of the main girders under actual live traffic loads before and after slab improvement are shown in Figure 9,10. Table 3 shows the live load-to-stress intensity ratios measured for respective main girders under actual live traffic load conditions after slab improvement. Figures 11 and 12 show an example of changes in frequency of stress and deflection of main girders before and after slab improvement.

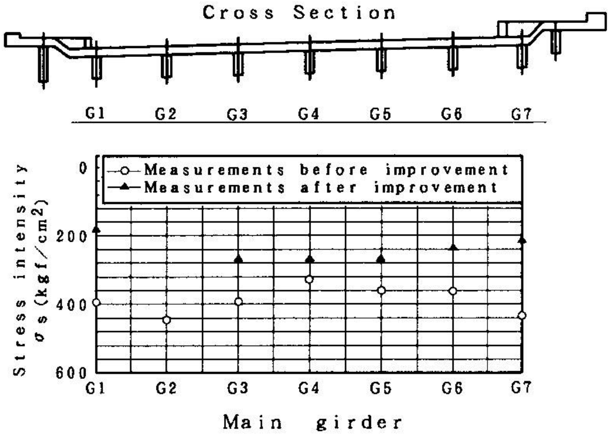

FIG. 9. MAXIMUM STRESS INTENSITY
(UNDER ACTUAL LIVE TRAFFIC LOAD CONDITION)

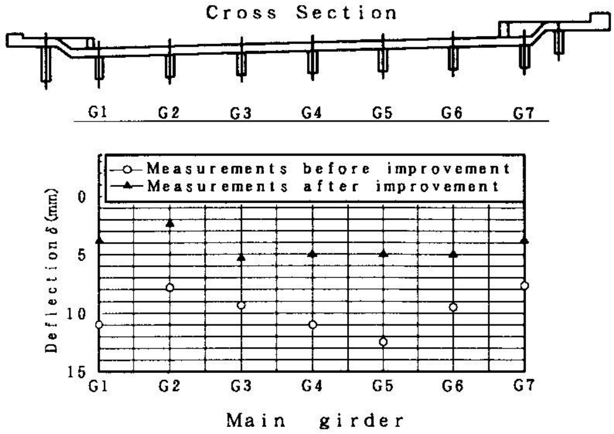

FIG. 10. MAXIMUM DEFLECTION
(UNDER ACTUAL LIVE TRAFFIC LOAD CONDITION)

Main girder No.	Stress intensity ratio under live load condition(σ max/σ 1)		Deflection ratio under live load condition(δ max/δ 1)	
	Before improvement	After improvement	Before improvement	After improvement
G1 girder	0.71	0.32	0.91	0.37
G2 girder	0.51	0.07	0.45	0.17
G3 girder	0.44	0.33	0.54	0.41
G4 girder	0.38	0.33	0.63	0.38
G5 girder	0.41	0.33	0.72	0.38
G6 girder	0.41	0.29	0.54	0.38
G7 girder	0.78	0.46	0.65	0.41

σ max:maximum stress intensity (measured value),
σ 1:calculated stress intensity
δ max:maximum deflection (measured value),
δ 1:calculated deflection

TABLE 3. STRESS INTENSITY RATIO AND DEFLECTION RATIO UNDER LIVE LOAD CONDITION (TL-20)

The measurements above indicate the following:

(1)It is apparent from Figures 9 and 10 that the maximum values of stress intensity and deflection measured for 24 hours after improvement under actual live traffic load conditions were improved by stress intensity and deflection as compared with pre-improvement values.

(2)Table 3 indicates that the measurements of stress intensity and deflection of main girders after slab improvement under actual live traffic load conditions were below half the calculated values (46 to 47% and 41 to 47%, respectively), or well below allowable values.

(3)The changes in frequency of stresses and deflections in main girders in Figures 11 and 12 show a substantial reduction in amplitudes of post-improvement stresses and deflections, suggesting that the rigidity has been improved markedly, and that the bridge is safer after than before improvement.

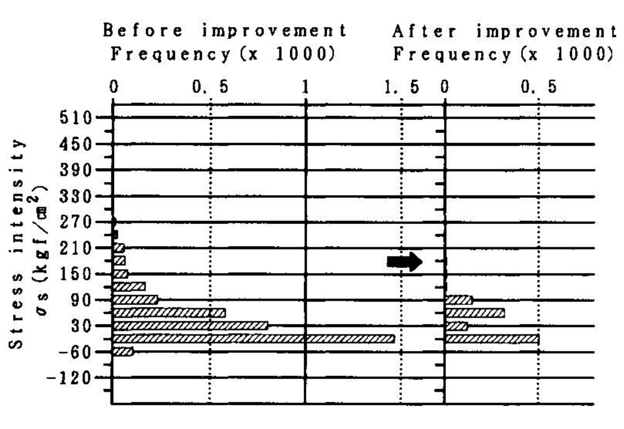

FIG. 11. STRESS FREQUENCY OF G1 GIRDER BEFORE AND AFTER IMPROVEMENT

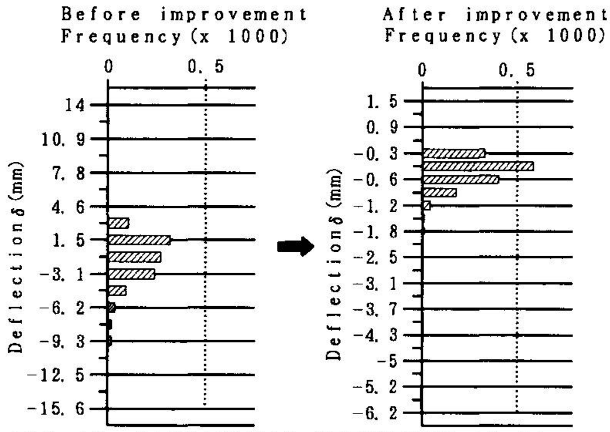

FIG. 12. DEFLECTION FREQUENCY OF G1 GIRDER BEFORE AND AFTER IMPROVEMENT

6. CONCLUSIONS

The experimental study of the newly developed box dowel in terms of performance and rigitity improvement effect, and the measurements of stress intensity and deflection in the main girders of a bridge to whitch the new jointing process was applied, have led to the following conclusions:

(1)The box dowel exhibits a sufficiently high capacity to check the dislocation of concrete slabs from the steel girders and to withstand shearing deformation, and is expected to perform its duty satisfactorily.

(2)The box dowel is practically warrantable for application to actual bridges if the shearing strength of the welds between the box dowel and steel girder is properly designed to effectively withstand the critical fatigue rupture strength due to repetitive loads.

(3)The measurements of stress intensity and deflection in the main girders of a bridge after slab improvement were smaller than those taken before improvement, bearing out the improvement in rigidity and safety of the bridge.

All these recommend the newly developed jointing process as a practical solution to the simplification of site work, improvement of slab structural characteristics, and reduction of construction period for steel road bridges by taking advantage of the excellent features of precast concrete slabs unavailable from the conventional in-situ reinforced concrete slab process.

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Deck Cracking of Shelby Creek Bridge

Fissuration du tablier du pont de Shelby Creek

Rissbildung in der Fahrbahnplatte der Shelby Creek Brücke

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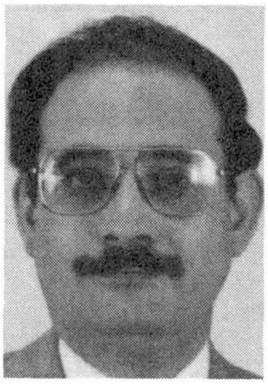

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SUMMARY

Early cracking of a cast-in-place reinforced concrete overlay of a sandwich concrete deck of Shelby Creek Bridge is investigated. Independent numerical checking proved that the bridge design was correct. Shrinkage cracking was enhanced by several factors. This enhancement was due to the special features of the bridge.

RÉSUMÉ

La fissuration prématurée de la couche supérieure du béton armé dans le tablier composite du pont de Shelby Creek est discutée. Une vérification numérique, indépendante, montre que le calcul était correct. Les fissures de retrait du béton ont été influencées par plusieurs facteurs, dûs aux particularités de ce pont.

ZUSAMMENFASSUNG

Die frühzeitige Rissbildung in der Fahrbahnbetonplatte von der Shelby Creek Brücke wurde untersucht. Die unabhängige numerische Prüfung hat bewiesen, dass der Brükkenentwurf korrekt ist. Die Schwindrisse sind durch mehrere Faktoren verstärkt. Diese Vergrösserung ist durch die spezielle Brückenbeschaffenheiten verursacht.

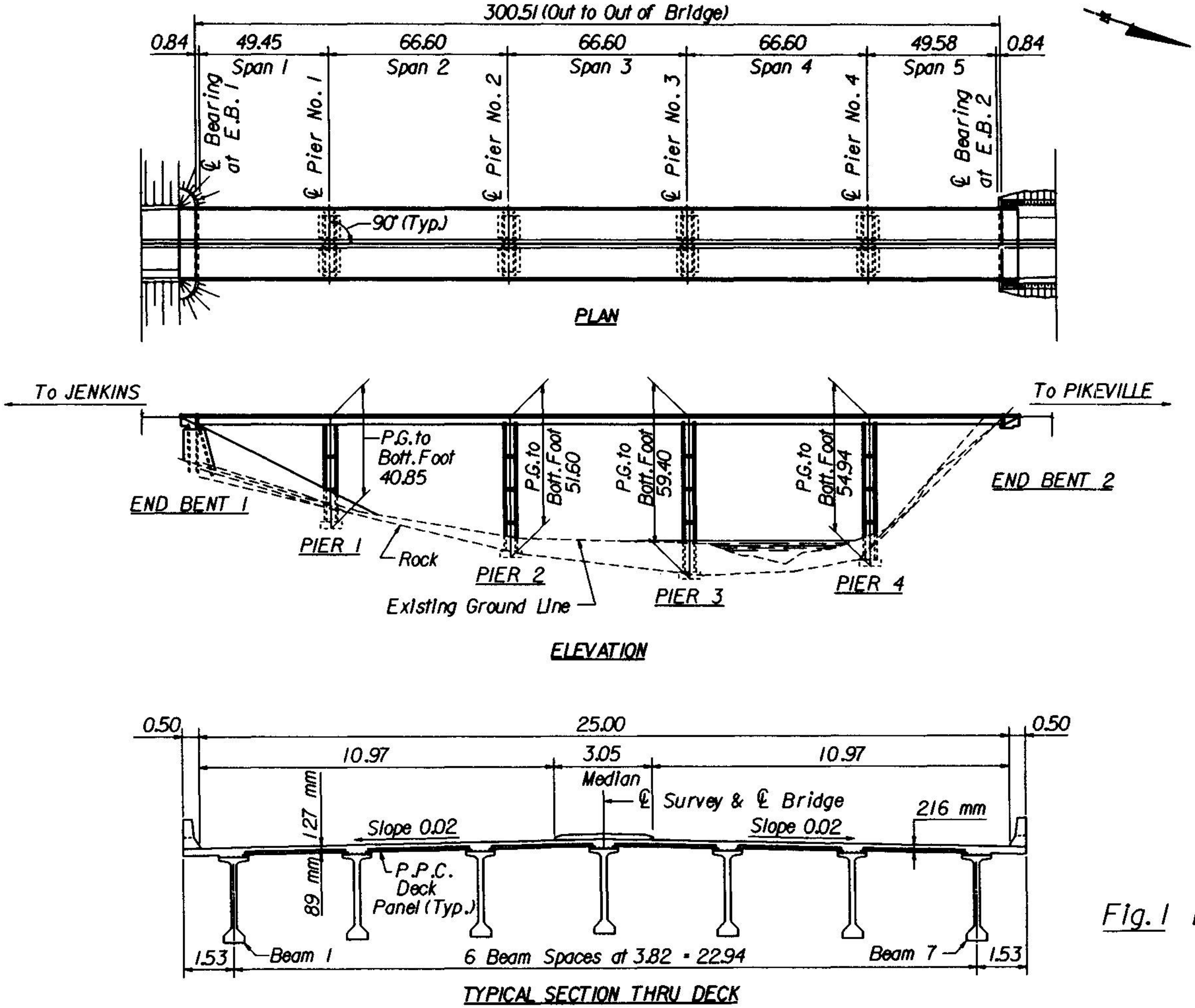
300.51 (Out to Out of Bridge)
0.84
49.45
Span 1
66.60
Span 2
66.60
Span 3
66.60
Span 4
49.58
Span 5
0.84
℄ Bearing at E.B. 1
℄ Pier No. 1
℄ Pier No. 2
℄ Pier No. 3
℄ Pier No. 4
℄ Bearing at E.B. 2
90° (Typ.)
PLAN
To JENKINS
To PIKEVILLE
P.G. to Bott. Foot 40.85
P.G. to Bott. Foot 51.60
P.G. to Bott. Foot 59.40
P.G. to Bott. Foot 54.94
END BENT 1
END BENT 2
PIER 1
Rock
PIER 2
Existing Ground Line
PIER 3
PIER 4
ELEVATION
0.50
25.00
0.50
10.97
3.05
10.97
Median
℄ Survey & ℄ Bridge
127 mm
Slope 0.02
Slope 0.02
216 mm
89 mm
P.P.C. Deck Panel (Typ.)
1.53
Beam 1
6 Beam Spaces at 3.82 = 22.94
Beam 7
1.53
TYPICAL SECTION THRU DECK

Fig. 1 Layout

1.0 INTRODUCTION

Over the last 15 years of the highway bridge construction history in U.S.A., occurrences of early concrete deck cracking have been recorded although no errors in bridge design and construction ever occurred. Shelby Creek Bridge is one such case.

Shelby Creek Bridge on US 23/US 119 in Kentucky is a four lane, five span (49.45m + 3 x 66.60m + 49.58m) beam type segmental bridge with two-way traffic as separated by a 3m wide raised median, Fig. 1. The bridge deck is 25m wide and it is composed of precast prestressed 41 MPa concrete panels (PPC) 9 cm thick used as formwork for 13cm cast-in-place reinforced 34 MPa concrete deck overlay (Fig. 4). The superstructure consists of seven precast 48 MPa lightweight concrete beams of constant depth at a spacing of 3.82m. The bridge is supported on four piers and two end-bents. The height of the piers varies from 35.74 to 50.09m. Piers are supported on spread footings on rock. Each pier consists of two twin curtain walls 0.90 x 3.66m spaced at 4.57m. They are battened together at 11.60m height intervals and post-tensioned.

The vertical curve, 213.36m long between the grades +0.5% and 7.0%, starts 16.84m before pier 3. This curve corresponds to a velocity of 80-90 km/h. Heavy loaded coal haul trucks, going mainly downhill in southbound lanes, are traveling with a speed of 100-110 km/h.

1.1 Description of Deck Cracking

General. The construction of Shelby Creek Bridge was opened to traffic at the end of December, 1991 and was fully completed on April 20, 1992. The cracks developed in the bridge deck were noticed in Fall, 1992. These cracks are developed only in the top, poured-in-situ, 13 cm concrete deck.

Cracks are mainly transverse and/or longitudinal with some which are semicircular. Two distinct patterns of cracks are identified: 1) above the piers, i.e. between their two diaphragms

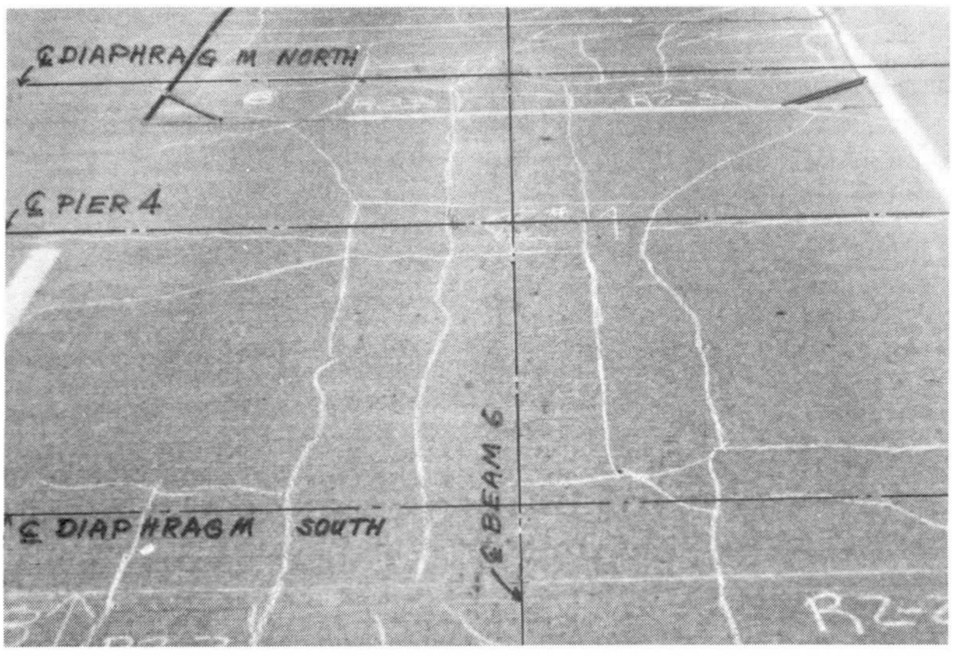

Fig. 2 Cracks at Pier 4

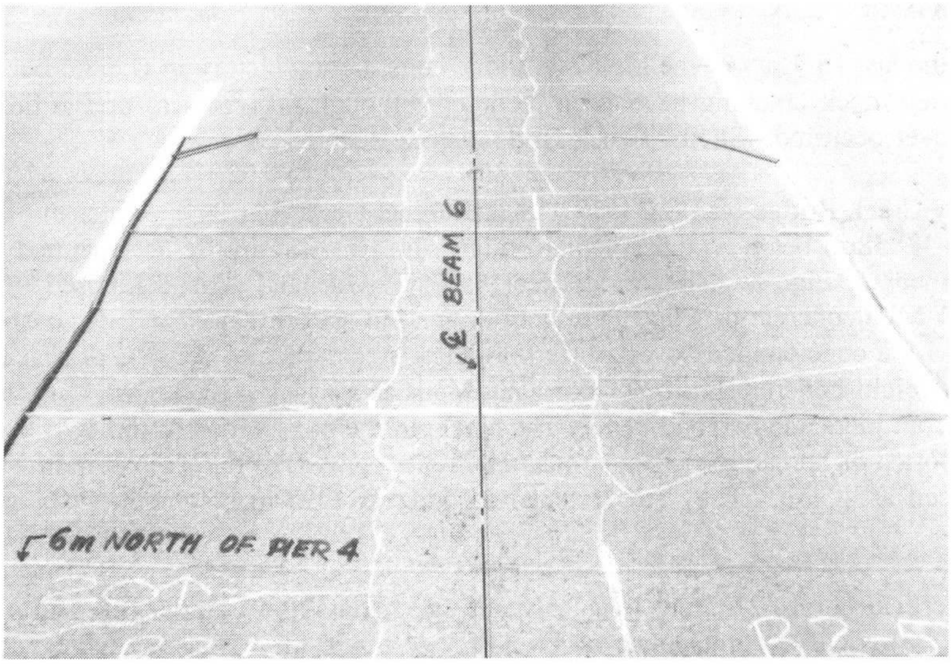

Fig. 3 Cracks Away of Piers

(curtain walls) Fig. 2 and 2) in the spans, away from piers, Fig. 3. Common to all cracks is more intense cracking in the exterior lanes of both traffic directions, than in the other two interior lanes, i.e., between the second and third beam of both bridge side faces . Next, cracks are more pronounced at piers 3 and 4 than piers 1 and 2, and the longitudinal cracks in spans are mainly developed along one or both sides of top flanges of both first interior beams.

Cracks at Piers. In Fig. 2, the typical pattern of enhanced cracks at pier 4 are shown. Both external lanes are almost centered on the first interior beams, and the longitudinal cracks are developed on both sides of beam's top flange. Transverse cracks crisscross them in the deck within the two pier curtain walls (diaphragms) in a way almost symmetrical to the pier centerline. Longitudinal beams and transverse diaphragms (curtain walls) produce deck panels 3.82 by 4.57m exposed to two dimensional restraints for any overlay displacement, including one due to the shrinkage. Due to this two-dimensional restraining effect, the corresponding stresses, developed in deck, produce tensile principle stresses at variable angles with bridge axis, i.e. the corresponding cracks in the overlay are semicircular.

Cracks in Spans. In Fig. 3, enhanced longitudinal cracks in the last span from 6 to 11m north of pier 4 are shown. Cracks are almost parallel, and they are centered above the first interior beams, i.e. beams 2 and 6. They are spaced about 0.65 to 0.85m apart. This distance is approximately equal to the space (0.75m) between ends of precast prestressed concrete panels (P.P.C.) above the beam flanges. Later on in the span, the interior crack eventually stops and only the exterior one runs further and then stops.

Major transverse cracks are developed only in interior bridge spans, leaving two end spans with mainly longitudinal cracks. There are about 26 cracks in southbound lanes and 23 cracks in northbound lanes. Very few of the cracks reach the raised median. Their spacing is between 0.60 and 2.50m and they are again more pronounced above piers than in spans.

2.0 SPECIAL BRIDGE FEATURES

No errors that could explain the deck cracking were found through an independent numerical checking of the design of composite deck slab, an analysis of live load static and some dynamic effects, and a partial checking of construction documents and activity.

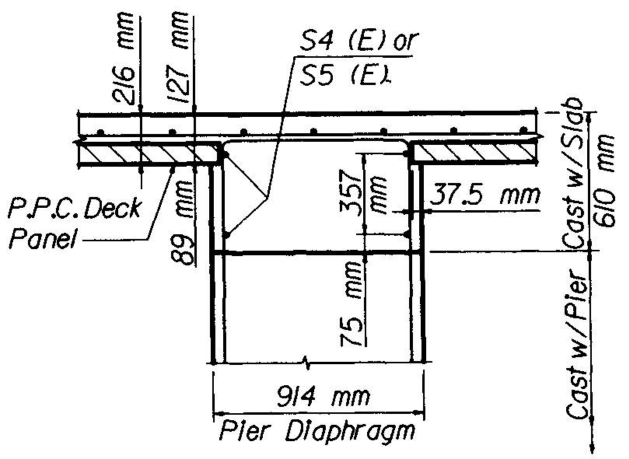

Fig. 4 Section at pier diaphragm

Therefore, differential shrinkage, as enhanced by some bridge characteristics, resulted in the described cracking. The special features and their contributions to cracking are:

- PPC Panels. First, they produce above main beams (between panels' longitudinal edges), longitudinal gaps 0.64m wide, where overlay depth is suddenly increased from 13cm to 22cm. Second, at their beveled transverse edges having a spacing of every 2.5m, the similar increase is 10.2cm. This means that a rectangular pattern of differential shrinkage is already set-up by use of PPC panels. Third, the end of panels resting on beam flanges are not rigidly fixed by the overlay and small rotations under live load have enhanced longitudinal cracks, which are started by shrinkage, to be more pronounced in the lanes with higher traffic volume, i.e. in the outside lanes.

- Two diaphragms at curtain walls of piers. These massive diaphragms, 0.91cm thick and 4.57m apart, Fig. 4, with main beams produce rigid two dimensional restraints in addition to the previous rectangular pattern. Therefore, cracking above piers is more intense than in the spans.

- Bridge vertical alignment. Heavily-loaded 623 kN trucks travel southbound downhill with speeds over the design speed for vertical curve as stated in Section 1.0, producing more crack enhancement by PPC at Piers 3 and 4, than 1 and 2 and more in the outside southbound lane than in the northbound lane.

- Bridge pier height and configuration. Due to the bridge almost north-south orientation, each pier curtain wall facing south is exposed to direct sun radiation as opposed to the other twin wall being always shaded. Therefore, a differential temperature of 10 to 20°C is created between the two walls. This produces a relative raising in the deck slab, e.g. at pier 3 of 0.7 to 1.4cm at a distance of only 4.57m and therefore a hogging moment as well. This introduces a slab tension of 0.57 MPa to 1.14 MPa respectively.

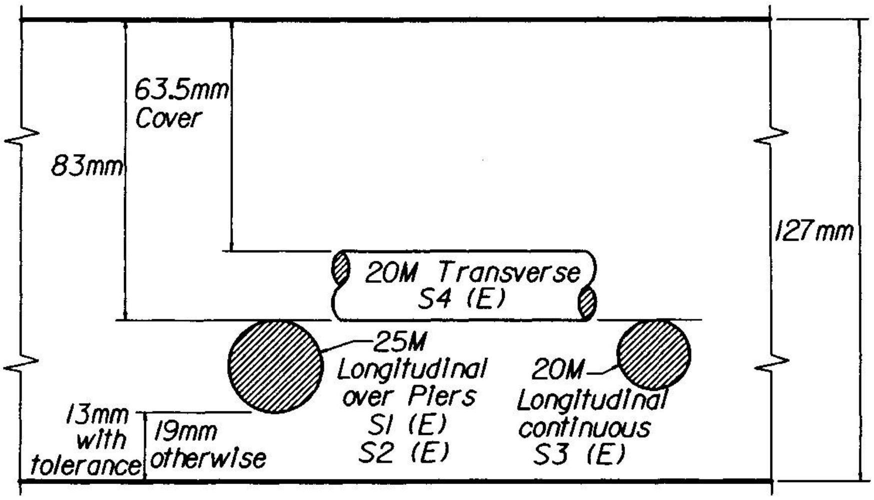

Fig. 5 Detail of topping overlay

3.0 CONCLUSION

Unwanted cracks caused by inadequate design or construction were not present. The cracks are "restraining cracks" caused by the restrained volume changes due to shrinkage, creep, elastic shortening and temperature gradient. It is known that about two thirds of any slab shortening is due to shrinkage. Many of the restrainers previously discussed, as well as sudden volume changes, produce differential shrinkage and hydration temperature gradient, which consequently can cause the cracking. Large clear cover of epoxy-coated bars also contributed to the cracking, Fig. 5.

A percentage-wise distribution of the causes for the overlay cracking on the Shelby Creek Bridge was estimated to be as follows: 65% shrinkage, 10% PPC panels, 5% temperature gradient between overlay and PPC, 10% temperature in walls, 1% pouring sequence, 4% large concrete cover and 5% still unknown.

As the cracks proved to be stabilized after three months of observation and measurement (March 1993-May 1993) and the concrete was already 1.5 years old, the only repair required was sealing the cracks.