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Session A1

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

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Strengthening of the 120-Year-Old Substructure of a Railway Bridge

Renforcement de l'infrastructure d'un pont-rail de 120 ans

Verstärkung des 120 Jahre alten Unterbaus einer Bahnbrücke

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SUMMARY

The 120-year-old substructure of a railway bridge has been strengthened to allow for a considerably heavier new superstructure throughout a projected service life of 100 years. The joints of the original masonry piers and abutments have been reconditioned, the foundations of two piers have been strengthened by the soilcrete technique, and an abutment has been anchored in the bedrock in order to resist the high horizontal loads.

RÉSUMÉ

L'infrastructure d'un pont-rail de 120 ans a été renforcée pour supporter une nouvelle superstructure, nettement plus lourde, pendant la durée d'utilisation prévue de 100 ans. La maçonnerie en pierre naturelle des piles et des culées a été remise en état, et les fondations de deux piles ont été renforcées avec des pieux. Une culée a été ancrée dans la roche pour reprendre les importants efforts horizontaux.

ZUSAMMENFASSUNG

Der 120 Jahre alte Unterbau einer Bahnbrücke wurde verstärkt, um den beträchtlich schwereren neuen Überbau über eine geplante Nutzungsdauer von 100 Jahren tragen zu können. Die Pfeiler und Widerlager aus Natursteinmauerwerk wurden instandgesetzt, und der Baugrund unter zwei Pfeilern wurde mit Hilfe von Jetting-Pfählen verstärkt. Für die Aufnahme der grossen Horizontalkräfte musste ein Widerlager im Fels verankert werden.

1. INTRODUCTION

The steel superstructure of a 233 m long double track railway bridge over the river Aare at Brugg, Switzerland, is being replaced by a post-tensioned concrete box girder (Fig. 1). The substructure built in 1875 has been strengthened and the pedestrian suspension bridge is being reconstructed.

The new girder was constructed span by span on temporary concrete piers and scaffolding on the downstream side of the existing bridge. After having removed the five single span steel truss girders for either track of the existing bridge, the new continuous girder will (in two stages) be shifted laterally into its final position. Thus, except for one complete shut-down and two periods of single track use, the railway traffic is not affected by construction operations.

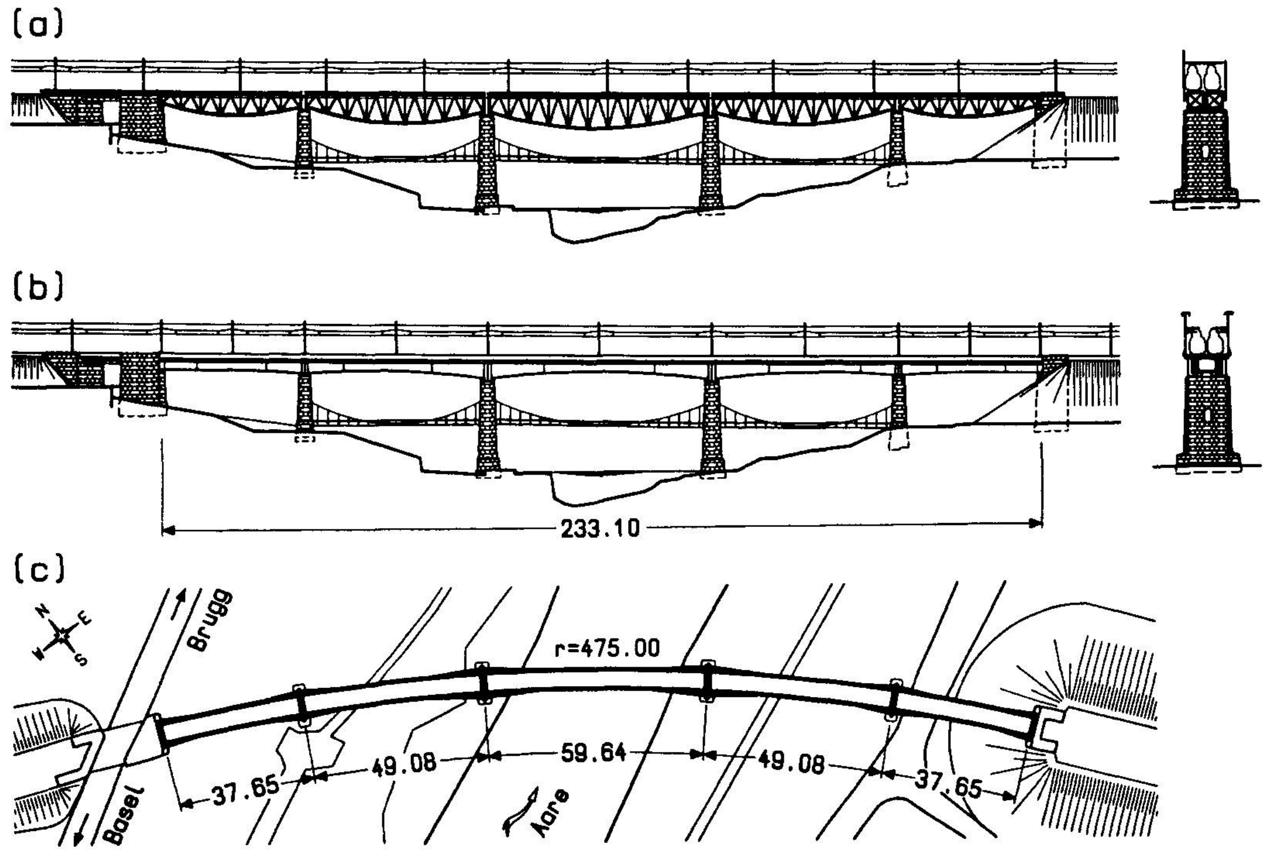

Fig. 1 Overview: (a) Steel superstructure on natural stone masonry substructure; (b) New post-tensioned concrete box girder on strengthened substructure; (c) Plan view of new bridge. Note: Dimensions in metres.

Compared to the open-decked steel structure the average dead load of the new concrete girder (including ballast) increased from 69 to 490 kNm^{-1}, i.e. by 610%. Design values for usual live loads increased from about 110 to 160 kNm^{-1}, i.e. by 45%. Strengthening measures for the substructure had to take into account these increased loads, the projected future service life of 100 years and the condition that all construction work had to be executed under full railway traffic.

This paper presents the techniques used to strengthen the different parts of the substructure, i.e. the foundation of the two side piers, the natural stone masonry of the piers and abutments, and the West abutment. Assessment of the existing structure, strengthening concepts and experiences gained during execution are discussed

2. FOUNDATION OF SIDE PIERS

While the two 25 m high central piers are founded on solid rock the somewhat shorter side piers are founded on layers of weathered molasse with a high clay content. These soft layers have a variable thickness of 1 to 8 m (Fig. 2).

Currently, average (extreme) soil pressures without traffic and with maximum traffic loads amount to 0.38 (0.45) and 0.55 (1.29) MPa, respectively. In the future, these values will increase to 0.62 (1.16) and 0.80 (1.50) MPa, respectively. In order to avoid unacceptable settlements and inclinations of the piers it was decided to replace the weathered molasse by a material with considerably higher modulus of elasticity and strength.

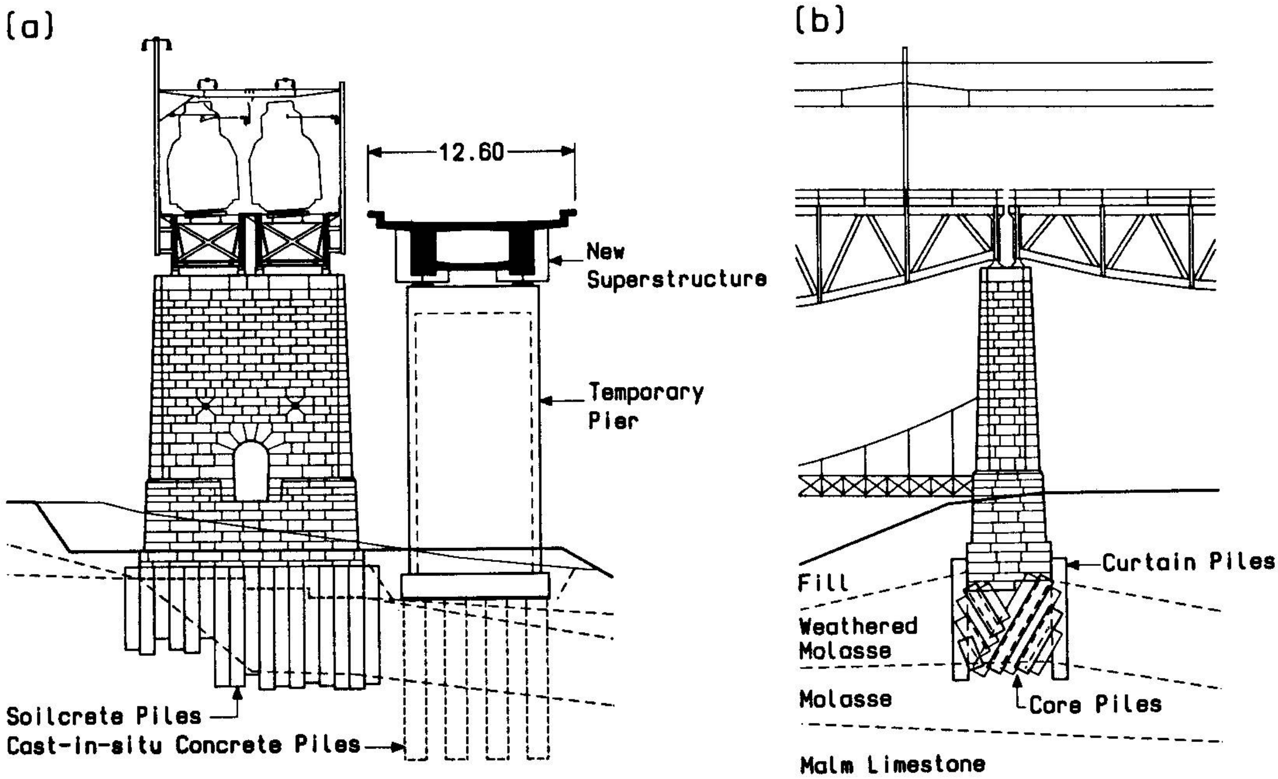

Fig. 2 Soilcrete pile foundation of side piers: (a) Cross-section; (b) Elevation.

Traditional underpinning was discarded because of high costs and risks for railway traffic during execution. The soilcrete technique was chosen after a feasibility test had shown that piles of sufficient quality could be made despite the soil's high clay content. It was decided to complete a curtain along the perimeter of the pier base prior to filling the core underneath the foundation. The curtain piles served to encase the material underneath the pier and allowed to gain further experience on the actual soil conditions. Construction of the core piles began only after the curtain piles had passed severe quality control test. Pier settlements and inclinations were continually monitored during the execution of the soilcrete piles.

The soilcrete piles were made by three-phase jetting. After having drilled down to the rock, the soil was cut from the bottom to the top by a high-pressure water jet followed by a cement injection. Retraction speed and jetting pressures for water and cement injection amounted to 1.1 mms^{-1}, 470 bar and 80 bar, respectively, and the injection material contained 690 litres of water and

870 kilogrammes of cement per cubic metre. The cement used allowed for early strength gain. The feasibility test had shown that soilcrete piles with a diameter of about 1 m could be expected, and thus, a spacing of 0.8 m between adjacent piles was prescribed. Pile production had to follow an alternating sequence to avoid concentration of fresh pile material. The main difficulty encountered during the execution was the adjustment of machine and injection parameters due to the variation of the clay content in the weathered molasse.

The quality of the soilcrete piles was checked by means of coring. While the average (minimum) values of the modulus of elasticity and of the compressive strength were found to be equal to 5 (2) GPa and 6 (2) MPa, respectively, the average density of the piles amounted to 16 to 20 kNm^{-3}. These values met the specifications and it was verified that the quality was sufficiently uniform throughout the soilcrete body.

3. NATURAL STONE MASONRY OF PIERS AND ABUTMENTS

In the 19th Century, most bridge piers and abutments were made of natural stone masonry according to the principle illustrated in Figs. 3(a) through (c). The circumferential masonry is responsible for carrying the loads. The inside core is an irregular and voided filling containing rubble and excess mortar from the construction of the exterior wall. Because of its low modulus of elasticity, the core does not carry any significant loads. The masonry blocks were placed on two strips of bed mortar having a 5 to 10 times lower modulus of elasticity than the stone material [Fig. 3(b)]. Joints between blocks were sealed with a weather resistant finishing mortar [Fig. 3(c)].

Examination of the condition of the piers and abutments revealed the usual damage due to weathering, i.e. deterioration of the outermost 100 to 200 mm of the joints, cracking of a few single blocks and overall wear of the surface. The exterior masonry wall has a thickness of about 750 mm and is made of limestone blocks with a compressive strength of 150 MPa. Due to the new superstructure, the average permanent stress in the exterior wall will increase from 0.35 to 1.6 MPa in the upper portion of the pier and from 1.4 to 2.4 MPa at its base.

The exterior walls have been prepared for the higher loads by reconditioning the existing mortar joint and by filling the voids in the mortar to allow for a smoother stress distribution in the joint. The head of the piers will be replaced by a massive 2.5 m high reinforced concrete block that transfers the forces from the bearings to the exterior walls. This concrete block also serves as a "roof" preventing water from entering into the pier.

Execution followed the steps illustrated in Figs. 3(d) through (f). First, the weathered surface was cleaned and the weak bed mortar, i.e. the outermost 100 to 200 mm, was removed by water jetting at a medium pressure. Second, the remaining mortar was supplemented by new bed mortar with similar mechanical properties. In the third step, remaining voids in the joints were grouted. Core grouting in Step 4 aimed at filling the voids immediately behind the masonry to achieve a back-anchorage of the wall in the core. In Step 5 the joints were sealed with a 20 to 30 mm thick layer of weather resistant finishing mortar. Where necessary, single masonry blocks were consolidated or replaced.

The requirements for the new bed mortar included a maximum value of the modulus of elasticity of 10 to 15 GPa and a limitation of the alkali-content of the cement to 0.6% to avoid crystallisation with subsequent damage in the stone material. Modern mortar and injection materials were used and adjusted to achieve the low modulus of elasticity required in the present application. In hindsight, the authors would have preferred to use more traditional materials, containing fewer additives and leaving less doubt regarding chemical compatibility with the natural stone masonry.

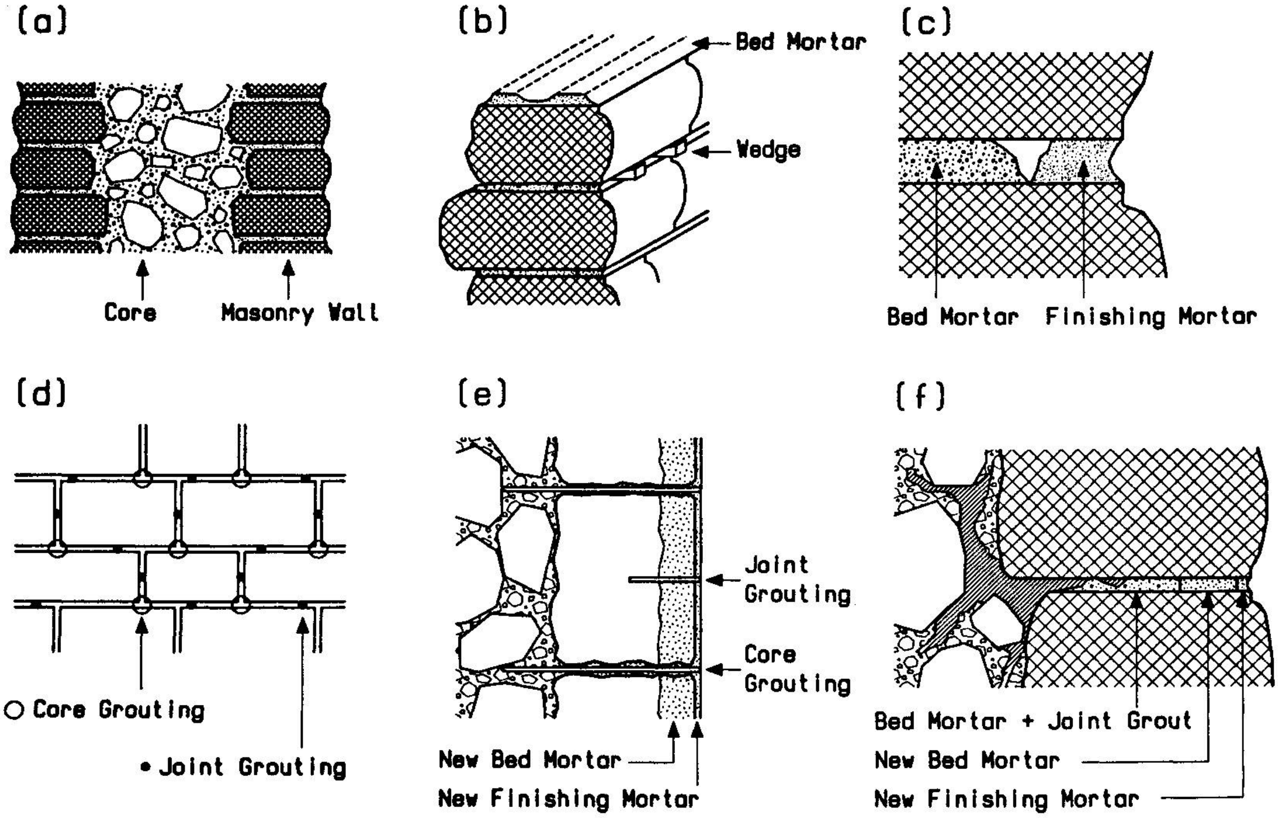

Fig. 3 Natural stone masonry: (a) Vertical section through pier; (b) Placing of masonry blocks; (c) Bed joint; (d) Elevation showing grouting locations; (e) Plan view; (f) Cross-section through bed joint.

Work was carried out in portions of about 40 m^2 of pier or abutment surface, alternating between front and side faces. The East abutment was used to gain experience and to refine the working procedures for the piers and the West abutment. The main difficulty was to find the right workability and technique to bring the new bed mortar into the joint. For quality control, cores were taken from the joints; remaining voids and mechanical properties of the new joint material were found to be satisfactory.

4. WEST ABUTMENT

Due to the change of the static system from a series of single span girders to a continuous five-span girder, the West abutment will be subjected to drastically increased horizontal loads. Breaking forces and friction forces induced in the bearings due to temperature changes are the primary actions in this regard. In passing, it is interesting to note that, compared to former provisions, the current design value for breaking forces of rail traffic has increased by 40%.

In spite of the congested space for the execution, [Fig. 4(a)] the West abutment was chosen as the fixed support of the bridge since it is of moderate height and founded on rock, while the pier-like East abutment is embedded in an earthfill dam.

After reconditioning the masonry, the abutment was strengthened by horizontally placed steel reinforcing bars to obtain a solid abutment block [Fig. 4(b)]. In addition, vertical steel reinforcement is placed to interlock the abutment block and the concrete block housing the bridge bearings. The required resistance to the horizontal design load of 11 MN, acting 13 m above the solid rock, is provided by the weight of 38 MN of the abutment block as well as by a number of inspectable and replaceable ground anchors with a total anchor force in service of 16 MN [Figs. 4(c) and (d)].

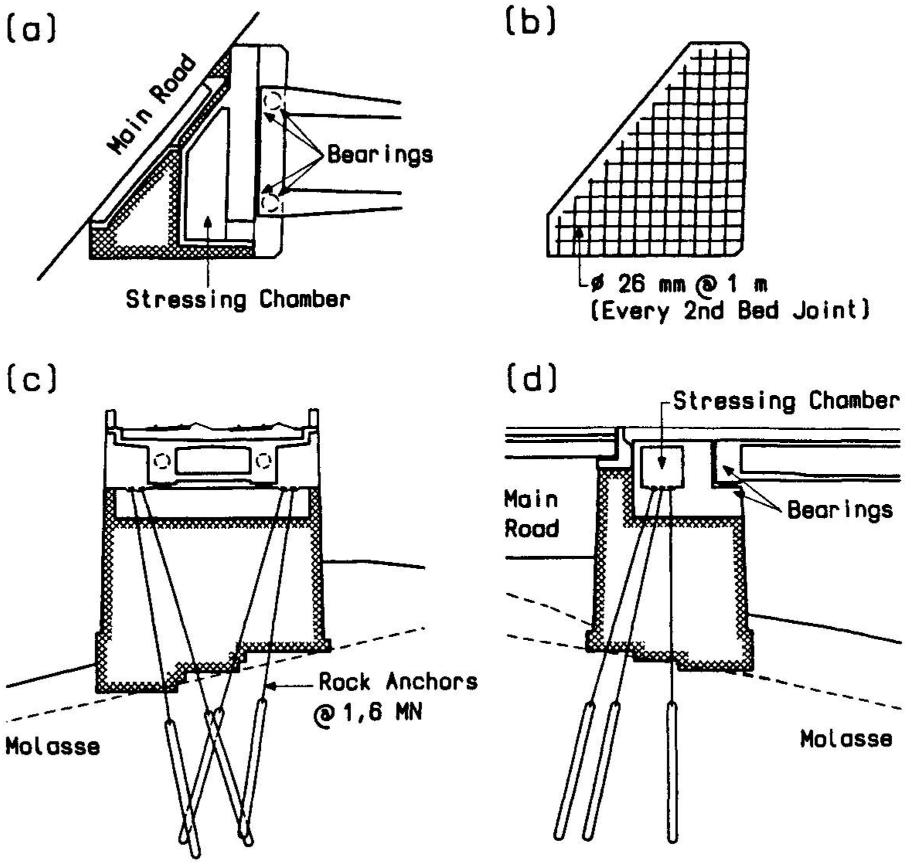

Fig. 4 West abutment: (a) Plan view; (b) Orthogonal reinforcement in bed-joints; (c) Cross-section; (d) Elevation.

5. CONCLUSION

The chosen project management has contributed much to the success of this project. The contractual arrangements between the client, the proof engineer, the consulting engineer (who was responsible for design and on-site management), and the contractors have allowed for the necessary close and efficient collaboration. In an area where no textbook solutions are available, a consensus based on engineering judgement is necessary among all partners in order to arrive at technically sound and cost effective solutions.

Structural Rehabilitation of a Reinforced Concrete and a Prestressed Concrete Bridge

Réhabilitation de la structure de deux ponts, en béton armé et précontraint

Erneuerung einer Stahlbeton- und einer Spannbetonbrücke

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SUMMARY

Two examples of works on reinforced and prestressed concrete bridges are described, each involving different aspects of problems faced by the maintenance programs that the Italian Roadway Board has carried out in recent years. The first case involves an old reinforced concrete bridge whose heavily damaged structures required radical repairs and anti-seismic measures. The second case concerns prestressed concrete span cantilever cables to be inserted in the cross section in order to restore safety margins and correct some of the deflections due to creep.

RÉSUMÉ

Deux exemples de travaux sur des ponts en béton armé et précontraint sont décrits, chacun impliquant différents aspects de problèmes rencontrés par les programmes de maintenance mis sur pied par l'Administration Italienne des Routes. Le premier cas concerne un vieux pont en béton armé dont les importants dommages structuraux ont demandé des réparations essentielles ainsi que des mesures antisismiques. Le deuxième cas concerne le renforcement de travées en porte-à-faux d'un pont en béton précontraint par l'ajout de câbles dans la section transversale, afin de rétablir les marges de sécurité et de corriger une partie des déformations dues au retrait.

ZUSAMMENFASSUNG

Aus dem Unterhaltsprogramm der Italienischen Strassenamtes der letzten Jahren werden beispielhaft zwei Brücken mit unterschiedlichen Problemstellungen beschrieben. Eine alte Stahlbetonbrücke wies so schwere Schäden auf, dass sie radikale Sanierungs- und Erdbebenschutzmassnahmen benötigte. Im zweiten Fall waren in einer Spannbetonbrücke Kragarmvorspannkabel in den Betonquerschnitt einzubauen, um Sicherheitsreserven wiederherzustellen und einen Teil der Kriechdurchbiegung auszugleichen.

1. INTRODUCTION

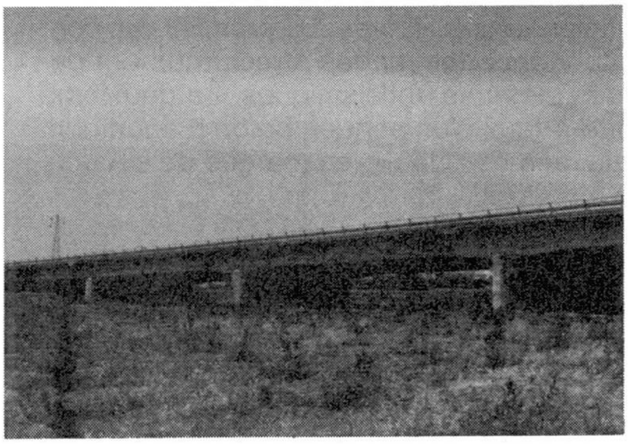
Increased traffic rate and seismic damage in many sites have led to numerous interventions to repair and restore the structures of Italian highways. Deterioration and alteration of materials, deformations caused by creep and need for additional anti-seismic measures give rise to different types of problems. Two examples of strengthening measures are described. The first concerned twin 30-year old reinforced concrete cantilever and Gerber beam bridges on a highway in Southern Italy near the coast. The damage consisted mainly in degradation of the materials with alteration to the concrete, corrosion of uncovered steel bars and low-serviceability of the supports and joints associated with the extreme atmospheric conditions prevailing in the area. The viaducts were subjected to radical repairs (including the demolition of limited portions of the structure) and seismic rehabilitation by accomplishing the longitudinal continuity of the different spans through the elimination of the fixed supports on the piers and all the intermediate joints between the beams and transferring the seismic forces back to special supports constructed behind the abutments. The second example concerned a very long (90 meters) prestressed concrete five-span cantilever bridge that had suffered serious deformations due to creep in the concrete. In order to recover the safety margins of the structure and make it possible to restore the original elevations of the road surface, an additional prestressing cable system was inserted inside the cross section.

THE "GALLICO VIADUCTS"

2.1 Present situation

During the late eighties the Italian National Roadway Board (ANAS) carried out work on some of the viaducts of the Salerno-Reggio Calabria. This section runs through three mountainous regions of Southern Italy where environmental conditions are very severe. Traffic rates have increased heavily since the roads were originally built and it was feared that the present state of conservation of various structural elements was incompatible with the heavy increase in loads. The seismic nature of the area, the particular nature of alterations to materials and the very poor safety margins associated with the isostasy of the structure could have led to brittle collapse under the stress of traffic without warning. A detailed plan of interventions was therefore carried out on some viaducts, for example those known as Fiumara di Gallico and Piana di Gallico, which are near the coast towards the southern end of the Highway (figure 1) in one of the oldest (early sixties) and most threatened parts of the whole structure. The project aimed principally to improve the seismic behavior of the viaducts, adjust the structures to the increased loads and repair the areas, supports and the joints that had become altered. The operation was also intended to reduce the

FIGURE 1



SITUATION BEFORE THE INTERVENTIONS

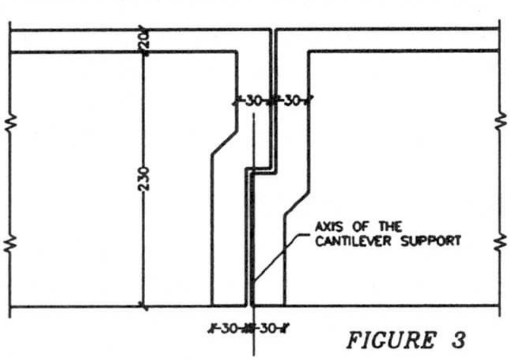


FIGURE 3

maintenance costs for the Board. The two viaducts, each of which has separate decks for each direction of traffic, consist of six 37.4-meter cast-in-place spans, three of which are cantilevered and three consist Gerber beams with 26.6-meter spans supported on six 6-7 meter high piers (photo n. 2). The Gerber beams are supported by very narrow joints girder (figure 3) with four ribs connected by a lower slab for a short distance from the pier along the cantilever.

2.2 Causes of decay and demage

On account of the severe environmental conditions (marine salts in the rain water, wide temperature ranges, heavy traffic) and poor maintenance, the degradation has affected mainly structural elements such as the Gerber joints, in which the steel bars are completely uncovered and deeply corroded

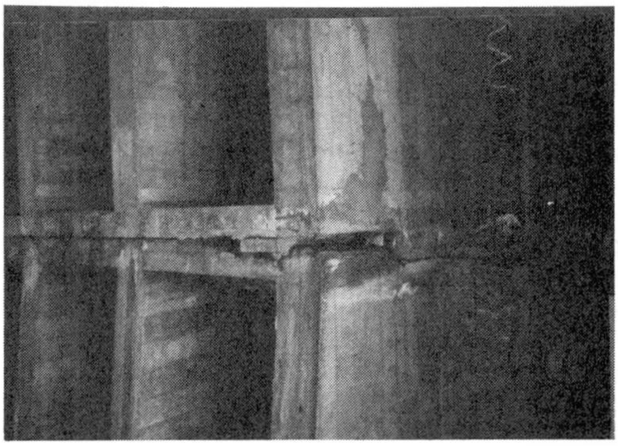


photo 4 - Fiumara di Gallico Viaduct
Gerber joint (lower view)

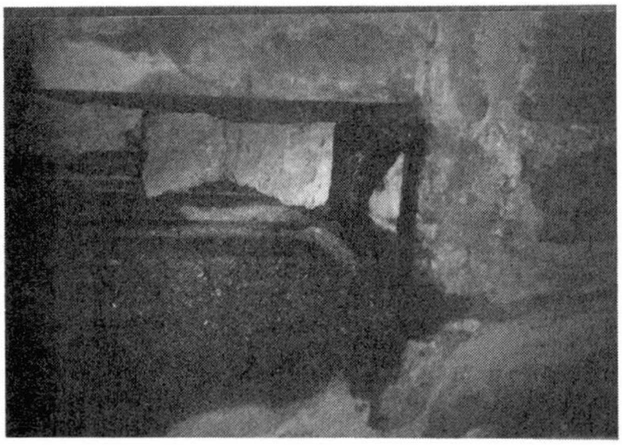


photo 5 - Fiumara di Gallico
Gerber joint (detail)

(photo n. 4-5). The seat of the supported beams - which is further restricted by degradation - is so reduced as to cast doubt on the stability of the beams. Weathering has also removed the concrete bar covering in many areas of the decks, particularly on the west side, towards the coast. A detailed program of in situ tests (pull-out tests, carbonation tests, microdrilling and sampling of concrete, endoscopic tests, etc.) on the structure was carried out to evaluate the actual state of alteration of the material, their strength and the efficiency of the supports apparatus.

2.3 Intervention criteria

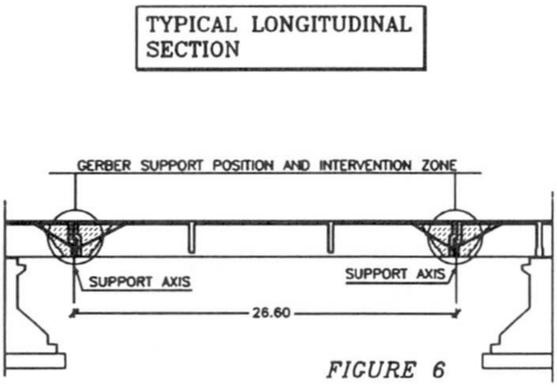


FIGURE 6

The principal objective of the intervention was to improve the seismic behaviour of the structure by creating a continuous kinematic chain between the various spans so as to transfer the horizontal actions to special purpose-built structures. In consideration of the state of the Gerber joints, it was decided to demolish these parts of the structure (figure 6) and rebuild them along more siutable geometric lines in order to ensure an adequate seat to the support, insert the bearing devices and accomplish the continuity of the structure by consolidating the upper slabs. The new structure was realized in Rheoplastic high-strength concrete connected to the remaining parts through connecting bars (figures n. 7-8). The supports on the piers were removed, after lifting the decks alternately, and replaced by mobile unidirectional or bidirectional devices. The deck was connected at the ends to blocks built behind the abutments on special foundations with micropiles and steel tie-anchors. The connection was ensured through special shock-transmitting devices placed horizontally so as to permit slow displacements due to temperature variations and prevent impulsive actions. Two 1200 KN shock transmitters were inserted behind the abutments on each deck, capable of resisting an overall seismic force of little less than 5000 KN without damage (figure n. 9). In this way, the structure became an axially continuous beam whose longitudinal seismic forces are transferred to the blocks; the size of the foundations can resist such a force. Transversally, the continuity of the slabs allows most of the seismic forces to be transferred to the abutments - the most rigid supports - thus ensuring a more uniform distribution of the residual forces among the different piers. The forces acting on the head of the piers, up to about 1500 KN longitudinally and transversally, were reduced by the measures to tens of KN longitudinally (mainly friction forces) and to about 300 KN transversally. The measures thus led to an unloading of the piers in seismic conditions, so that no specific strengthening measures were required. In the National Roadway Board's strategy, the elimination of the surface joints will reduce the overall maintenance costs of the bridge. Special interventions on the deteriorated structures were also carried out. Three types of restoration were involved: the first involved the areas in which the surface concrete had been largely removed or which were affected by carbonation, high level of aggregates or a loss of serviceability of the corroded steel bars (thickness < 7-8 cm); the second concerned areas of damage limited to the covering concrete (thickness < 4 cm); the third essentially involved the protection of the concrete surfaces more exposed to aggressive agents and with minor cracks or deterioration.

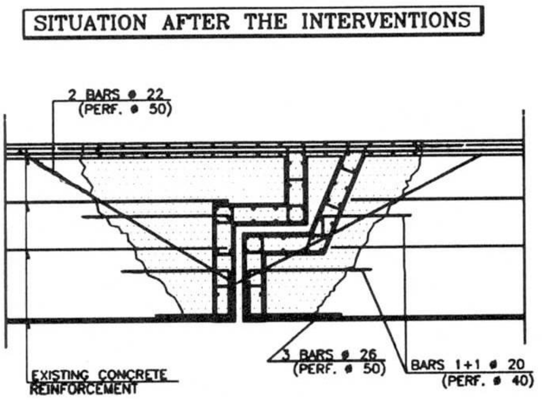


FIGURE 7

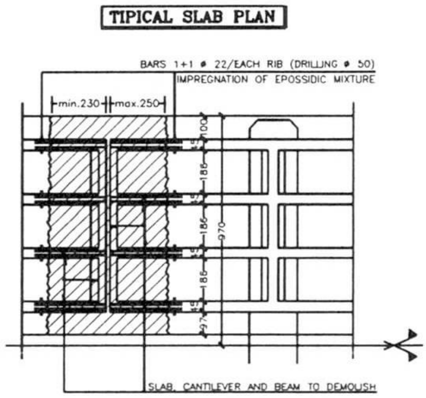


FIGURE 8

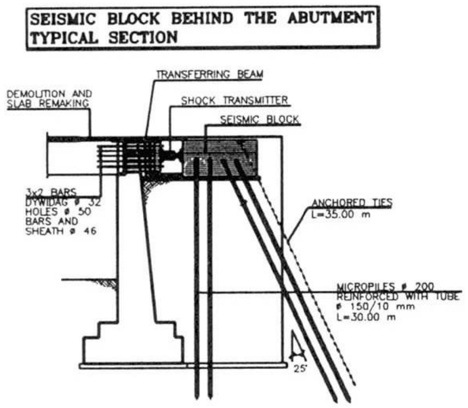


FIGURE 9

3. THE REGGIA VIADUCT

3.1 Present situation

The Reggia viaduct crosses the catch drain of the same name on the E45 Highway 30 km north of Perugia. It was built in the late seventies using the successive segment cantilever method on piers with a maximun height of 15 meters (photo 10). The cantilevers were then connected, so that the end result is a continuous 5-spans beam on 6 supports. The three central spans are 90 meters long and the lateral spans 45 meters. The supports on the central piers are of the fixed hinge type and those on

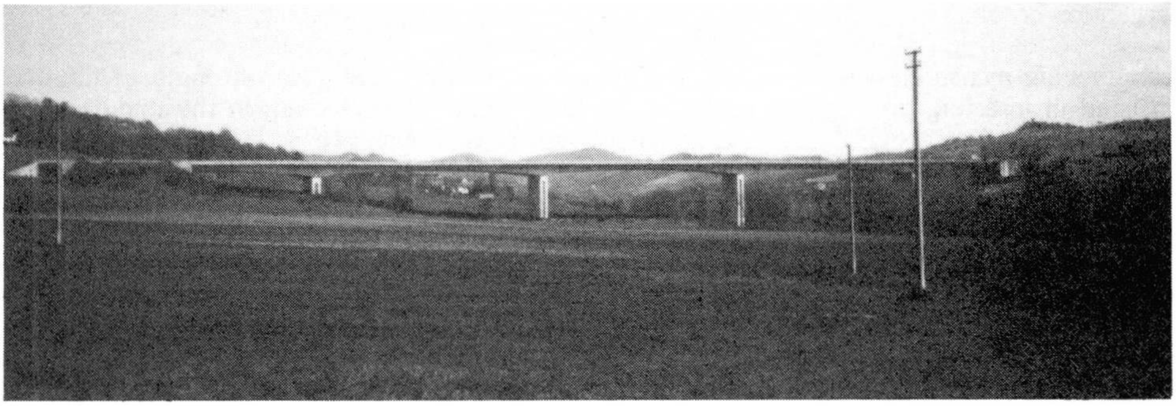


photo 10 - Reggia Viaduct - General view

the piers and on the abutments are simple supports. In order to avoid hyperstatic components during prestressing of the sewing cables of the central span, some provisional hydraulic jacks were inserted into the middle of that span. The cross section consists of a 3-cell box girders, 4.9 meters high at the level of the piers and 2.3 meters high in the midspan and on the supports of the abutments. Over the years the bridge has suffered some deflections in the lateral spans (piers 1-2 and 3-4), with a maximum amplitude of about 15 cm, while the central span shows no or minimum deflection (figure 11). These deformations called for a general re-examination of the structure, though they did not compromise the use of the viaduct.

3.2 Causes of decay and damage

The phenomena observed are related to creep processes that cannot easily be predicted at the design stage. On account of the construction method, based on a cantilevered section-by-section procedure that involved building a structure with provisional boundary conditions that differed from the final ones, the creep effects have produced a redistribution of the stress for permanent loads and, in particular, an increase in positive bending moments in the middle of the spans and a reduction near the piers (figure 12). The individual sections of the cantilevers were poured in different stages, with non-homogeneous pouring and weathering conditions, so that they presented different creep functions. Even minor uncertainties in calculating the dead or prestressing loads could also have led to considerable variations in the creep deformations, bearing in mind the effective reciprocal influences between the creep and the steel relaxation of the prestressing cables. Another factor that should be considered is the possible effect of the insertion of provisional hydraulic jacks in the middle of the central span, which may have partially compensated the creep deformations of the third span, jeopardizing the second and fourth spans.

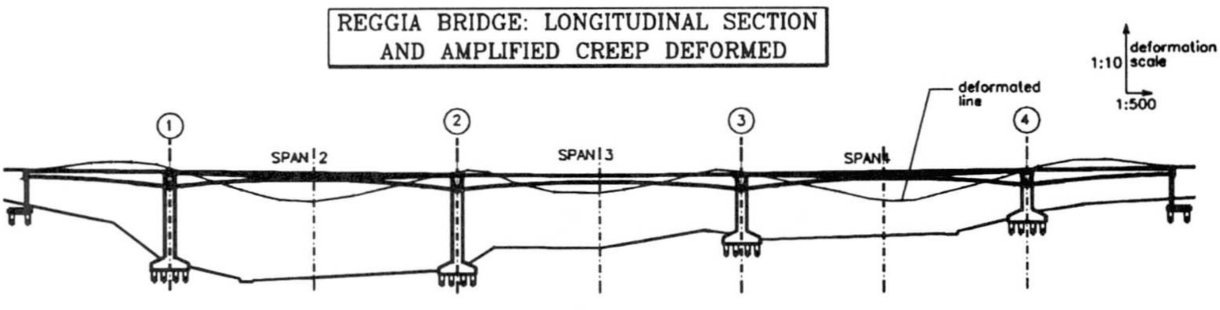


FIGURE 11

3.3 Intervention criteria

The strengthening measures were designed to enhance the bending and shear strength of the deck and consisted in inserting prestressing cables in the lower part of the section in the middle of the lateral spans and also, to a lesser extent, in the central span. Prestressing was achieved by inserting new cables within the box-girder and tensioning them prior to the casting the new levelling pavement. The new prestressing forces thus compensate either the creep effects or the loads corresponding to the new pavement. Each cable was protected by a grease sheath and anchored anchored to the diaphragms of the pierheads; they were placed along an almost rectilinear course to reduce the deviation devices. 192 six-inch wobblers were used, to provide an overall force of about 16000 KN for the lateral spans and about 10800 KN for the central one (photo 13). In order to differentiate the amount of prestressing, some cables are continuous from the first pier-head to the last, whilst others have only the length of one of the lateral spans. The only deviation device inside the box-girder, at midspan level (figure 14), consists of a steel frame connected to the ribs of the caisson to which the cable was connected after tensioning, so as to ensure continuity to the transversal section. Three devices were used for each span, one for each of the three cells of the box-girder. The frames were assembled with steel profiles and plates connected by high-strength bolts in order to allow transport and the mounting inside the box-girder. A horizontal strut with St. Andrew's crosses was provided in order to resist to the friction component of the prestressing forces. The work was successfully carried out without restriction traffic and restored about 10% of the previous deformations. The increase in safety levels was assessed at around 30%. Thus the measures adopted significantly increased the durability of the structure by closing the craks caused by creep. The load test performed on the bridge on completion of the work also showed an increase of the overall stiffness of the whole structure of about 20%.

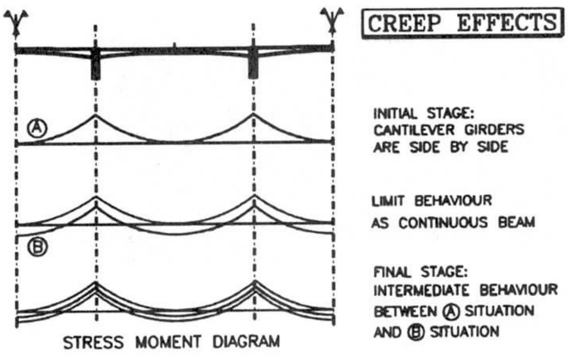


FIGURE 12

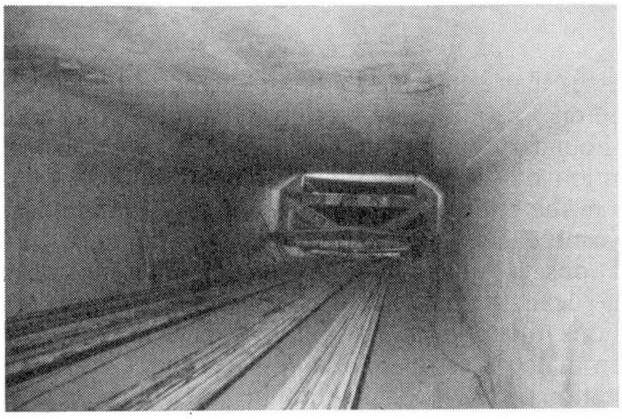


photo 13 - Prestressing cable system and deviation device frame

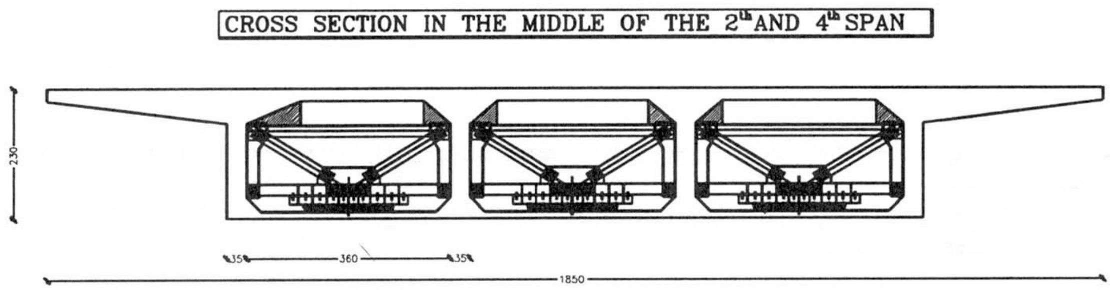


FIGURE 14

Extending the Useful Life of the Tower Bridge in London
Prolongement de la durée de vie du Tower-Bridge à Londres
Verlängerung der Nutzungsdauer der Tower-Bridge in London

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Associate
Mott MacDonald
Croydon, UK

Pat Stanley, born in 1945 in Ireland, graduated from Queen's University, Belfast, in 1966. He has specialised throughout his career with Mott MacDonald in the design and construction of bridges.

C. SNOWDEN
City Engineer
Corporation of London
London, UK

Colin Snowden has served in recent years as the City Engineer for the Corporation of London, responsible not only for the infrastructure of the City of London, but for many of the buildings and bridges which the City and Corporation hold in trust.

J.D. HAYWARD
Director
Mott MacDonald
Croydon, UK

John Hayward, born in 1940, is currently a main board director of Mott MacDonald and managing director of their Structural and Industrial Division which encompasses both structural and mechanical/electrical engineering.

SUMMARY
Tower Bridge was completed in 1894 and having survived the coming of the motor car and the bombing of the second World War, it continues to provide road access into the City of London and to open for shipping on a daily basis. Its longevity and the extension of its use for tourism are the result of regular maintenance and occasional repairs to the structure combined with a philosophy of ensuring that the bridge continues to serve the community. The paper reviews the most recent repairs and the provision of tourist facilities as an extension of the useful life of the bridge

RÉSUMÉ
Le Tower-Bridge, achevé en 1894, a survécu à l'apparition de l'automobile ainsi qu'aux bombardements de la Seconde Guerre Mondiale. Il continue à offrir un accès routier à la City de Londres et à s'ouvrir journellement pour la navigation fluviale. Sa longévité et son attraction touristique sont le résultat d'une maintenance régulière et de réparations occasionnelles combinées à une philosophie visant à ce que le pont continue à servir la communauté. L'article passe en revue les réparations les plus récentes et les aménagements touristiques pratiqués dans le cadre du prolongement de la vie du Tower-Bridge.

ZUSAMMENFASSUNG
Die Tower-Bridge wurde 1894 fertiggestellt und überlebte die Entstehung des Automobilzeitalters wie auch den zweiten Weltkrieg. Immer noch stellt sie die Strassenverbindung in die Londoner City sicher und öffnet sich täglich für Schiffsdurchfahrten. Ihre Langlebigkeit und touristische Bedeutung sind das Ergebnis regelmässigen Unterhalts und gelegentlicher Reparaturen im Verein mit einer Philosophie, den Gemeinnutzen der Brücke zu sichern. Der Beitrag berichtet über die jüngsten Reparaturen und touristische Einrichtungen zur Verlängerung der Nutzungsdauer der Brücke.

1. INTRODUCTION

Tower Bridge, which is now a national tourist attraction and one of the most distinctive trademarks of the City of London celebrated its centenary in 1994[2]. It is a hybrid structure with two suspended side spans and the steel towers are connected by a high level walkway. Twin leaf bascules rotate into chambers located within the piers on which the towers are built.

The bridge provides for two lanes of traffic between the London Boroughs of Southwark and Tower Hamlets and when opened for shipping there is 42.9 metres of clearance from high water level in the River Thames. Located adjacent to the Tower of London, the steel structure is clad in Portland Stone and Granite to harmonise with the ancient buildings.[3]

Recent years have seen the bridge being transformed with the aid of audio visual displays into a major educational and tourist attraction.

Tower Bridge opening for shipping

2. REPAIR AND MAINTENANCE

2.1 Repairs since 1931

In 1928 when the bridge was approaching 50 years of age it received a major refit to alleviate the effects of steel corrosion and to prepare it for the age of the motorcar. It was found to have a reserve of strength because the bridge's design was carried out soon after the Tay Bridge disaster and it has been adequate for all loading up to the present day. A 17 tonne weight limit and a 20 mph speed restriction are currently maintained in the interest of preservation and to reduce some vibration which continues to be a concern within the towers.

Cast iron panels on the high level walkways were replaced with steel and in the 1950's lattice girders were added to carry some 75 tonnes of the dead load on the ties between the towers.[4]

Originally the public was permitted to cross the river, even when the bridge was open to river traffic. In the early years the bridge appeared to be more open than closed and the staircases within the towers and the walkway across the top were open to the public and frequently used. The need for pedestrian access reduced as the road traffic dominated the river and the bridge became more closed than in the lifted position; the high level walkways were closed to the public in 1909.

The high level walkways in particular have had a considerable amount of engineering applied to them. In the 1960's suspenders were added to transfer more of the dead load onto twin 60mm diameter catenary cables formed from galvanised locked-coil wire ropes. Since the walkways are often used for receptions and corporate entertainment they are also serviced by two lifts. The lifts are located within the northwest tower and the southeast tower and can accommodate wheelchairs for the disabled and up to 40 persons or a maximum dead weight of 3175kg. The lift cars have 1200mm doors and a plan area 3725mm x 1375mm and each is driven by 40 HP motors which can generate a speed of 45m per minute.

In order to maximise the view out through the girders, double glazed windows have been fitted into the openings between the lattice girders and some of the original cast iron panelling has been replaced with steel and grp.

2.2 Regular Maintenance

A full time staff of fifteen, supervised by the Maintenance Manager provide not only for the regular maintenance but also the operating crews for bridge lifts. A Senior Technical Manager, a Technical Officer and three Technical Assistants make up a maintenance team and are on duty each day. A planned maintenance system was devised for the bridge following installation of modernised machinery in the 1970's. The system covers both the mechanical and electrical equipment and the bridge structure with service and inspection intervals ranging from once a day to once every five years.

2.3 Development of Tourism Facilities

In addition to maintaining Tower Bridge as an operational lifting bridge which plays a key role in the highway network around the City of London, the bridge's fame throughout the world and its role in promoting tourism has been recognised for more than 20 years. It is this extension of the bridge's use which can be a prototype for other structures around the world.

As a major tourist attraction and a public building many ideas have been developed through the years. It was proposed to surround the bridge and towers in a glass cocoon to provide large areas of office space which would have raised revenue in the same way as the shops and businesses which had occupied the space over the river in previous centuries. Proposals for a traditional pub and a restaurant to serve visitors' needs have been put forward and promoted, even to the tendering stage. The original engines which powered the bridge were recognised as part of the historical legacy of the 1894 bridge. One engine was found a safe home at the Forncell Industrial Steam Museum and others have been included in the provisions for tourism.

One of the original bascule drive engines remains within the machinery rooms and one is on display within the museum area located under the south approach. Two Armstrong Mitchell steam driven pumps each with a capacity of 360 HP are also on display within the original engine rooms.

The road decking has had considerable attention over the years. Originally there was timber laid onto steel buckle plates. Three layers of softwood and Greenhart were first replaced in 1949 and the entire deck was stripped out in 1962. Corroded buckle plates were replaced and foamed polyurethane blocks were grouted into position below Acme flooring panels. Water seepage into the decking continued to be a problem and during the 1970's further repairs to the decking were carried out. The present deck surface is based on Acme decking panels consisting of an 18mm base panel encapsulated in epoxy resin, an intermediate liquid applied waterproof membrane and another surface panel coated with a Cicol ET epoxy slurry and an anti-skid layer of Dynagrip.

Another problem which has occurred over the years is vibration induced by the passage of vehicles for which the bridge was not originally designed. A study carried out in the 1960's resulted in bracing rods and block walls being constructed at strategic places to tune out the most significant movements. Application of the 17 tonne weight limit and constant attention to the quality of the running surface both assist in restricting the vibrations to an acceptable level.

A significant replacement of the original hydraulic engines and the boilers and pumps which supplied them was carried out in 1976. The modern electric powered operating and bridge lifting system includes two low speed hydraulic motors, two independent electric motors driving piston pumps and a control panel. An 11 kV power supply is taken from the South Bank and an alternative supply is also available from the North Bank together with a 135 kVA standby generator, either of which can open or close the bridge at half speed.

From time to time throughout the years the ornamental balustrading and steelwork along the outer edges of the superstructure has been repaired and renewed. In a contract during 1978-9 most of the cast iron balustrading was removed and repaired, with new elements being cast where weld and metal infill could not repair the damage.

In 1992 further work was carried out to repair and renew balustrades on the lifting spans. Corroded sections of the steelwork supporting the balustrades and the decking under the bascule footways were the subject of extensive reconstruction works. Timber infilling was found to be retaining rainwater, leading to continued corrosion and increasing the bascule weight which was also overloading the lifting mechanism. A lighter fully sealed plywood decking was used as a replacement.

As a result of the most recent repairs and in order to minimise the demands on the lifting mechanisms a rebalancing and counterweight adjustment is to be scheduled in 1995.

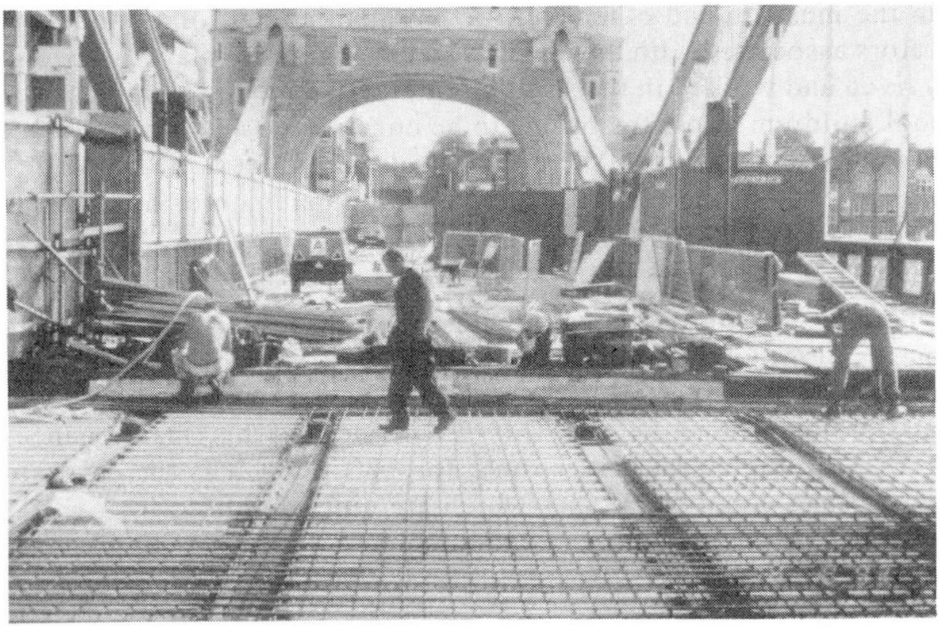
The most recent work to be carried out on the bridge structure has been an investigation of the vibration which continues to be of some concern although it has been established that the level of vibration is not structurally significant. It is proposed to update the vibration readings taken in 1960-70 by relocating gauges at the same locations. During a two day closure to vehicular traffic, isolated separate readings will be taken, while two vehicles of different weights (a single deck and a double deck bus) will travel across the bridge at different speeds.

Of particular interest to tourists are always the views from high places and Tower Bridge is particularly endowed with fine views and a fine viewing platform in the form of the high level walkways. Their use has been exploited in several ways.

3. RECENT MODERNISATION

3.1 Repairs to Road Deck

Extensive redecoration was carried out in 1992 in preparation for the celebration of the Bridge's centenary and it was during this operation that timber bulkheads were removed within the base of the towers, exposing parts of the steelwork not normally accessible.[5] Corrosion was observed and the removal of steel plating exposed internal voids and areas of corroded steelwork which had probably not been observed for 100 years. In some places there was no remaining capacity whatever and repairs were therefore necessary to restore the load carrying capacity while minimising the disruption to traffic and preserving both appearance and as much of the original structure was reasonably possible. For thirteen weeks in the summer of 1993 the bridge was closed to road traffic and the road deck under the towers was removed, exposing the underlying steelwork and enabling extensive cleaning, repairs and replacements to be effected.[6]



1993 Deck Repairs

3.2 Tourist Facilities

For the bridge's centenary a major modernisation of the tourism facilities within the bridge has been carried out. The new exhibition has been devised and designed by Bowes Darby & Associates in close co-operation with the City Engineers Department. Tourists enter by a new entrance in the northwest tower and leave by the southeast tower before walking along the south approach to the final experience, engine rooms and retail facilities under the southern arches.

The new entrance provides a waiting area adjacent to the ticketing equipment with a wall of video monitors which show the history of the bridge and the surrounding area. Tourists are taken by lift to the second floor of the tower to commence an educational and cultural tour which is illustrated in various ways.

An interesting audio visual display is provided, with the history of the structure being explained by an animatronic model of the chairman of the committee responsible for the bridge itself. The story and the engineering behind the bridge unfold in a variety of ways. Lifelike figures explain the detailed history and background of the bridge and models of the many alternatives considered are displayed. Theatrical lighting techniques and high technology projection equipment conjure up the ghostly image of Horace Jones whose brainchild the bridge was. The principle of the bascule is expounded on video.

Visitors pass over the river through the high level walkways, viewing the ever changing skyline of the City of London, and an extensive exhibition of pictures and exhibits which describe the construction of the bridge. A static display in the south tower includes tools and equipment set in and around an engineer's office of the 1880's and lighting exposes figures depicting construction work within the roof space of the tower itself.

In the south tower the visitors are shepherded into a model of the bascule chamber and having had a graphic explanation of the opening mechanism they experience the noises and communications signals associated with bascule movement. With a rumbling sound the roof of the viewing area begins to rotate intriguing the tourists and school children who visit to experience and learn.

On leaving the southeast tower by a lift festooned with contemporary newspaper cuttings visitors are led to the museum and other facilities. Again animatronic models depicting workers and operators associated with the original steam engines explain the circumstances under which they lived and worked in the 1880's. Interactive models, which are extremely popular with school children, allow the bridge to be constructed and the model machinery to be operated. Finally a reconstructed Victorian Theatre provides a further stage for more animatronic figures to explain the festivities and opening ceremonies held in 1894.

4. CONCLUSION

Although Tower Bridge, like most other bridges around the world, was intended to be a very functional means of providing a road and pedestrian crossing of the river its designers could not have envisaged it as the tourist attraction which it has become. The care and attention lavished on the bridge by the City Engineer and his maintenance staff is extending its life as a strategic cross river route for traffic but it is the extension of the bridge for educational and commercial uses which are quite unique.

5. ACKNOWLEDGEMENTS

The staff on the bridge itself and the engineers within the City Engineers Department who are responsible for the maintenance, together with the engineers at Mott MacDonald consider it an honour to be associated with the unique example of engineering which Tower Bridge has become. It is to the bridge that this paper is dedicated.

6. REFERENCES

1. WELCH, C. History of the Tower Bridge, Smith Elder & Co., London 1894
2. STANLEY, R.P. Tower Bridge - 100 Years On. British Construction Profile. Spring 1994. pp 11-16. McMillan Group Publications
3. TUIT, J.C. The Tower Bridge, The Engineer, London 1894
4. GROOME L.W. et al Tower Bridge, The Structural Engineer, London February 1985
5. Tower Bridge Corrosion Survey 1992, Final Report for Corporation of London, Mott MacDonald, Croydon September 1992
6. STANLEY, R.P. Tower Bridge London - History and Recent Repairs. Developments in Short and Medium Span Bridge Engineering 1994. Historical Bridges pp 1303-1313 CSEC, Canada

Crack-Free "Surround Concrete" Repairs of the Öland Bridge Piers

Renforcement des piles du pont d'Öland à l'aide d'une enveloppe de béton sans fissures

Rissfreie Betonierung in der Reparaturarbeit der Brückenpfeiler der Ölandbrücke

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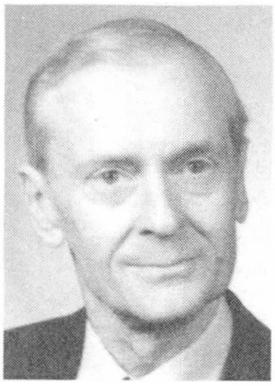


Ingvar Nilsson received his Master's degree in civil engineering in 1963 and Ph.D. in 1973 at Chalmers University of Technology, Göteborg, Sweden. Since 1976 he is at Nordic Construction Company, active in the fields of concrete structures, design, construction methods, materials, durability and repair.

SUMMARY

Damaged bridge piers have been repaired by casting new concrete around them. The use of a sliding layer between an old pier and its surrounding concrete is a new method of obtaining a crack-free structure. Based on measured load-deformation curves for the sliding layer in respect of both normal forces and shearing forces, surround concreting of the piers with a sliding layer can be dimensioned so that no cracks result from the heat of hydration and shrinkage.

RÉSUMÉ

Les piles de pont endommagées ont été réparées en y coulant une enveloppe de béton. Une nouvelle méthode, utilisant une couche de glissement entre la pile existante et le béton frais, a permis d'éviter la formation de fissures. Des courbes charge-déformation sous l'effet de l'effort normal et du cisaillement ont été établies pour cette couche de glissement. Ces courbes ont permis de dimensionner l'enveloppe de béton afin d'éviter la formation de fissures liées à la chaleur d'hydratation et au retrait.

ZUSAMMENFASSUNG

Schäden an den Brückenpfeilern sind mit frischen Betonschalen rund um die Pfeiler repariert worden. Die Benutzung von einem Gleitlager zwischen dem alten Pfeiler und der Betonschale ist eine neue Methode, um eine rissfreie Konstruktion zu erreichen. Die Dimensionierung, die auf abgemessene Last - Deformationskurven (für das Gleitlager unter beide Normalkräfte und Schubkräfte) basiert, wird ausgeführt, so dass die Betonschalen durch Reaktionswärme und Schwinden nicht gerissen werden.

1. INTRODUCTION

With a total length of 6072 m the Öland Bridge in Sweden was the longest bridge in Europe until the Great Belt bridges in Denmark were finished in 1994.
The Öland Bridge was built from 1968 to 1972. From 1990 to 1994 it has been undergoing one of the most comprehensive bridge repairs in the world.
112 of the 156 bridge piers have been repaired.

The piers spalled to a depth of 5-10 cm in the splash zone. The reinforcement was partially exposed in other places. The cause of damage was poor concrete quality. The concrete satisfied the specified strength requirements, but its durability was low. The Öland Bridge was built in accordance with regulations for concrete structures in force in the late sixties. Therefore, many other bridges built at this time also exhibit similar damage.

2. REPAIR METHOD

The piers have been repaired by casting new concrete around them down to the base plate. The existing piers are regarded as ineffective. The concrete shell has a thickness of 40 cm on the long side and 50 cm on the short sides. The work is done mostly below water level in special cofferdams, see Fig 1.

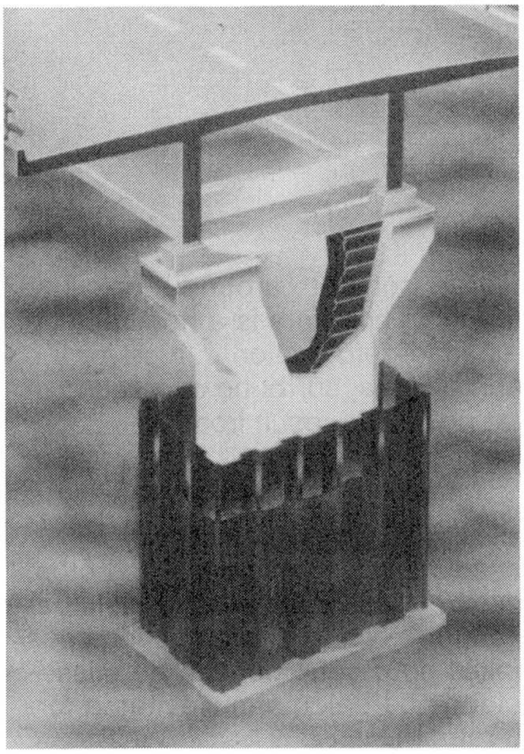
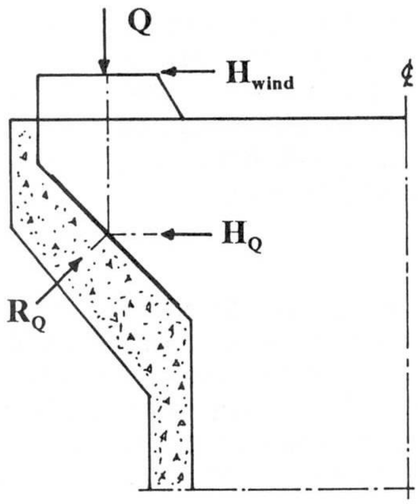


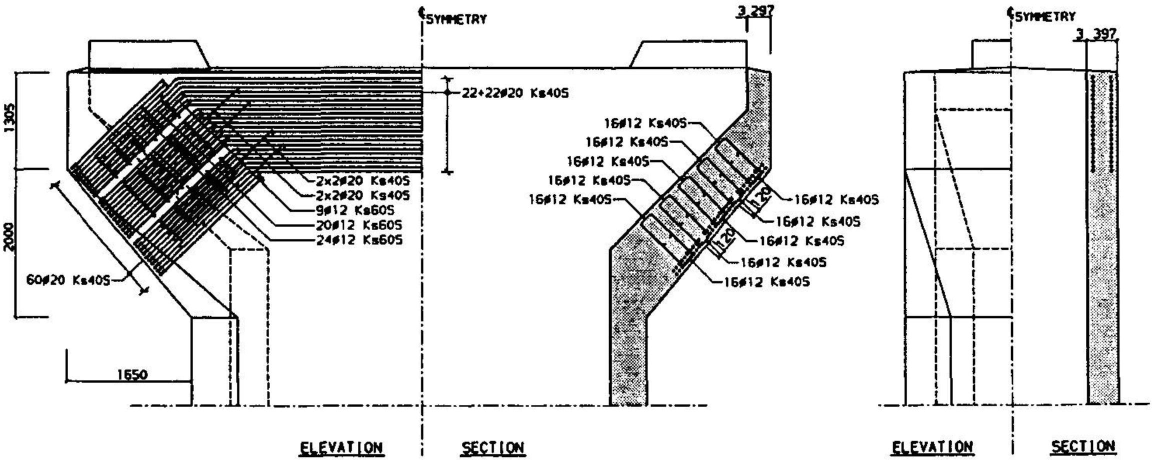
Fig. 1 The work is done under dry conditions in a cofferdam which stands on the base plate.
A sliding layer is placed between the old pier and surround concrete structure.

Fig. 2 Loads on pier top and reaction transferred to surround concrete structure below cantilevered beam.



2.1 Design

The load Q from the bridge bearing (Fig. 2) is transferred in the inclined contact surface. The reaction R_Q is carried by reinforcement in the concrete walls in the direction of the force. The reaction H_Q is a compressive force in the old concrete pier. Epoxy injection into the slit under the inclined surface of the cantilevered beam means that there is contact against the new surrounding pier. The injection is carried out not earlier than half a year after casting when most of the long time deformations have occurred. The rest of the surround concrete structure is separated from the old pier by a sliding layer based on asphalt. Fig. 3 shows the suspension reinforcement in the cantilevered beam.



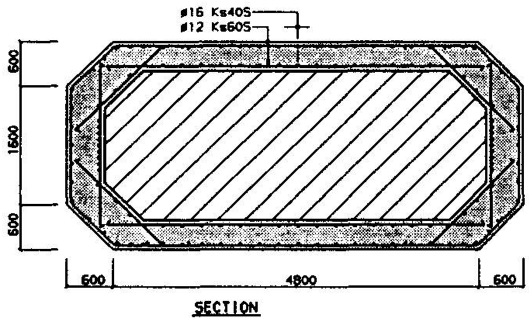


Fig. 3 Suspension reinforcement in the surround concrete structure of the cantilevered beam and a section of the pier shaft.

2.2 Concrete

In order to obtain surround concreting of good durability the following is required: 1. High quality concrete, 2. At least 50 mm concrete cover, 3. Crack-free surround concrete. Durability requirements demand that the surround concreting is crack-free. This is difficult to obtain and special measures are necessary.

The surround concreting is cast with quality K40 (40 Mpa on 15 cm cubes), Swedish cement, 420 kg/m^3 concrete, gravel size 18 mm, water-cement ratio $\leq$ 0.40, 5.5% of air (min 5.0%, max 6.5%), slump 80-100 mm normally without plasticiser.

Forms are only allowed to be removed when the new concrete has attained a compressive strength of 28 Mpa and has a temperature not more than 5°C above that of the concrete in the old pier and the ambient air. The aim is to limit the risk of surface cracks. Therefore, forms are usually removed after 15-30 days. Stripped surfaces are protected in temperature controlled environment and water cured continuously with fresh water until the concrete cube strength has reached 40 Mpa. This is reached after 35-55 days, depending on the time of the year. The pier is then painted with a silane. The cofferdam is now removed and the brackish water of the sound comes into contact with the new piers.

3. TEMPERATURE CRACKS

When surround concreting piers there is a risk that the surround concrete will crack. It contracts when the heat of hydration starts to decrease and the surrounding concrete cools. Horizontal contraction is prevented by the old pier. A risk of vertical cracking will then arise. If vertical contraction is prevented there is a risk of horizontal cracking. The cracks will pass through the surrounding concrete.

Calculated temperature development in the surrounding concrete and the old pier corresponds well with measured temperatures. The surrounding concrete heats up the old pier. As a consequence the old pier expands at the same time as the surrounding concrete cools down and contracts. This results in dangerous high tensile stresses in the surrounding concrete with a risk of cracks. In order to reduce the cooling and the risk of cracking, the concrete casting temperature is first lowered by cooling with liquid nitrogen.

Initially one pier, No 108, was tested without a sliding layer. The casting temperature was 7°C. Fig.4 shows measured temperatures in a central section. While the surround concrete is cooling, the old pier is heating up.
The accompanying contraction and expansion result in a demand for measures to ensure that the concrete would not crack. Fig.5 shows how pier 108 cracked.

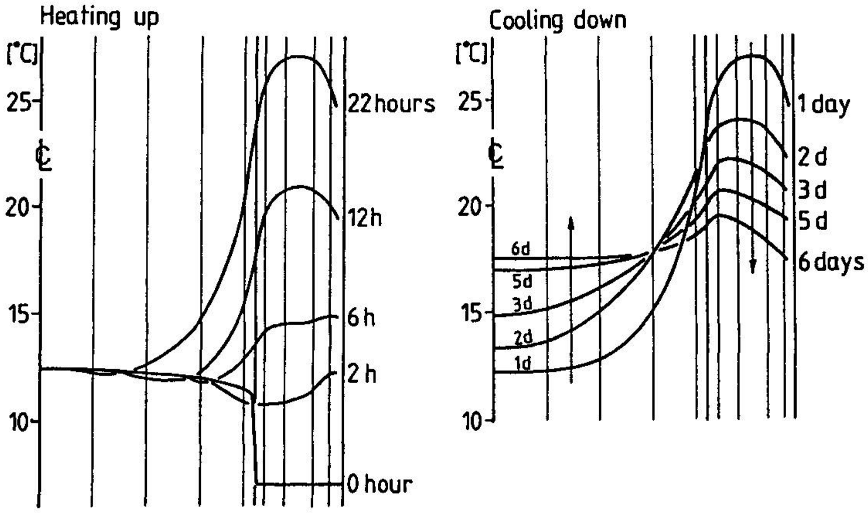


Fig. 4 Measured temperature distribution in a central section of pier No 108 during heating and cooling of the surrounding concrete.

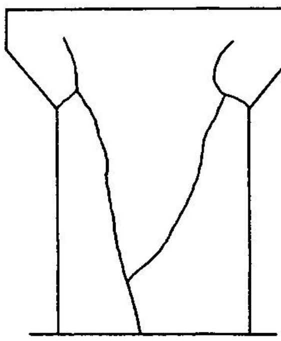


Fig. 5 Crack pattern for test pier No 108.

4. SLIDING LAYER

The use of a sliding layer between an old pier and its surrounding concrete is a new method of obtaining a crack-free result. Cooled concrete is used in order to limit the contraction during cooling.

On the long sides of the piers the sliding layer consists of two layers of 1.6 mm asphalt carpet. On the short sides of the piers where the movement requirements are biggest, a 3.0 mm cellular plastic layer is placed between the asphalt carpets. Sliding layers are not used in the load transfer areas.

Under compressive load the material flows out into pores in the concrete surface which after 20 years exposure has got a rough surface structure. In this way the sliding layer can take up a small movement, even when perpendicular to the surface.

4.1 Movement in the sliding layer

The movements which need to be taken up in the sliding layer are mainly caused by the cooling contraction of the surrounding concrete when the maximum curing temperature is exceeded. That part of the surrounding concrete which is positioned above the water also experiences a slight drying-out shrinkage.

When casting with cooled concrete the difference between max curing temp and curing temp after long time became about 20°C. The cooling contraction coefficient = $8.19 \cdot 10^{-6}$.

Horizontal movement requirements (Fig.6a)

Cooling contraction = $8.19 \cdot 10^{-6} \cdot 20 = 0.16 \cdot 10^{-3}$

Drying out shrinkage = $0.10 \cdot 10^{-3}$

Total = $0.26 \cdot 10^{-3}$

Vertical movement requirements (Fig.6b)

Vertical contraction at 20°C cooling:

For 10 m high pier = 1.64 mm + shrinkage

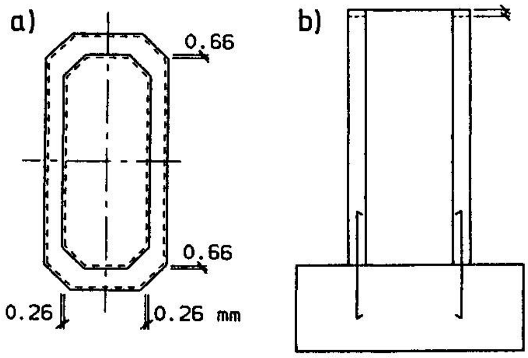


Fig. 6 Horizontal and vertical movement requirements for the surrounding concrete.

4.2 Load-deformation relationship

Which stresses occur in the surrounding concrete when there are movements as outlined above? When the sliding layer is compressed tensile stresses occur. Is there any risk that the surrounding concrete will crack? The load-deformation curves must be determined experimentally for different combinations of normal force and shearing force in order to provide a basis for dimensioning.

5. COMPRESSION ON SHORT SIDES

When the concrete rising rate is 0.6 m/hour there is a concrete pressure of 0.048 Mpa from 2 m concrete before setting. Fig. 7 shows that this corresponds to a compression of 1.34 mm. Cooling and shrinkage need, according to Fig. 6, a 0.66 mm movement requirement, in all 2.0 mm. When the sliding layer is compressed 2.0 mm against the pier side a counter pressure of 0.093 Mpa will occur according to Fig.7. According to Fig. 8 the tensile stress p in the long sides of the surrounding concrete is

$$p \cdot (0.4 + 0.4) = 0.093 \cdot 2.0 \qquad \text{I.e. } p = 0.23 \text{ Mpa}$$

The concrete reaches a tensile strength which is higher than this tensile stress after less than 1 day. Accordingly, it will not crack.

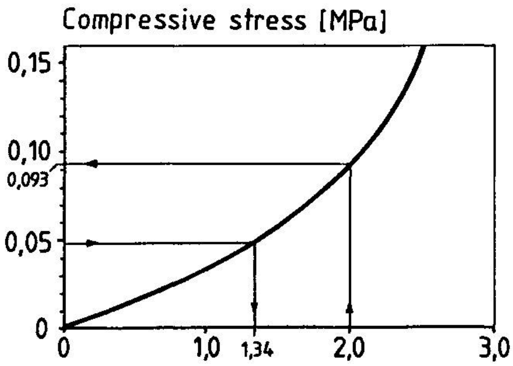


Fig. 7 Measured working curve for a sliding layer with a cellular plastic layer.

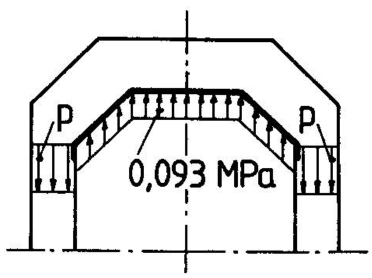


Fig. 8 Tensile stresses in the long sides of the surrounding concrete.

6. COMPRESSION ON LONG SIDES

The concrete pressure is according to the previous chapter assumed to be 0.048 Mpa. Fig. 9 shows that this corresponds to a compression of 0.32 mm. Cooling and shrinkage require according to Fig. 6 a movement need of 0.26 mm, in all 0.58 mm.

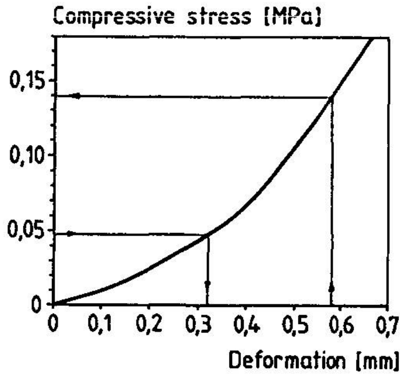
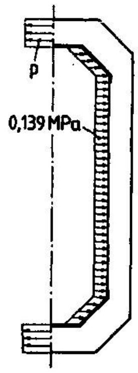


Fig. 9 Measured working curve for a 3.2 mm thick sliding layer on the long sides.

Fig. 10 Tensile stresses in the short sides of the surrounding concrete.



When the sliding layer is compressed 0.58 mm against the pier side a reaction pressure of 0.139 Mpa will occur according to Fig. 9. The tensile stress p in the short sides of the surrounding concrete is according to Fig. 10

$$p \cdot (0.5 + 0.5) = 0.139 \cdot 5 \quad \text{i.e. } p = 0.695 \text{ Mpa}$$

The concrete reaches a tensile strength which is higher than the calculated tensile stress after less than 1 day. Accordingly, it will not crack.

7. SHEARING MOVEMENT

The surrounding concrete structure is anchored to the bottom slab. When the surrounding concrete cools down it contracts vertically. Working curves for shearing force-sliding were measured (not shown here). The shearing stress along the pier decreases against the bottom slab. The calculated tensile stress in the surrounding concrete becomes much less than the concrete tensile strength. Accordingly, it will not crack for vertical sliding.

8. CONCLUSIONS

The use of a sliding and compressible layer between the existing piers and the surrounding concrete opens up possibilities of taking up the movements related to surround concreting. Based on measured load-deformation curves for the sliding layer in respect of both normal forces and shearing forces surround concreting of the piers with a sliding layer can be dimensioned so that there are no cracks resulting from the heat of hydration and shrinkage.

If a cellular plastic layer is included in the sliding layer one must select a material with closed cells and low water absorption, otherwise there is a risk of bursting under freezing conditions. For the Öland Bridge cellular plastic is used only at the pier ends where the movement requirement is greatest. Furthermore, the cellular plastic is enclosed between two watertight asphalt membranes.

REFERENCE

1. Nilsson, Ingvar, Crack-Free "Surround Concrete" Repairs Öland Bridge Piers. Concrete International, Detroit, July 1994.

Rehabilitation of a Historic Bridge over the Sand River in South Africa

Consolidation d'un pont historique sur le Sand River, en Afrique du Sud

Erneuerung einer historischen Brücke über den Sand River in Südafrika

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V3 Consulting Engineers
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Rudolph Kotzé, born 1956, obtained his civil engineering (honours) in structural engineering at the University of Pretoria. He is involved in bridge design and rehabilitation.

SUMMARY

The old Sand River arch bridge near Virginia in the Free State, which was built in 1926, was totally inundated during the floods in 1988 and, apart from the approaches which were washed away, the bridge superstructure was severely damaged. Future planning of roads in the immediate vicinity of the bridge made provision for a new bridge over the Sand River, thus replacing the existing bridge. Closer investigation revealed that the bridge exhibited far greater inherent strength than previously believed. Through innovative design, a solution was found which obviated the construction of a new bridge and led to substantial cost savings to the client.

RÉSUMÉ

Un pont construit en 1926 sur le Sand River près de Virginia fut entièrement submergé par les inondations de 1988 qui emportèrent non seulement les voies d'accès, mais endommagèrent également le tablier. La planification des routes dans les alentours du pont prévoyait déjà un nouveau pont sur le Sand River, remplaçant ainsi l'ouvrage existant. Les études révélèrent que le pont possédait une résistance beaucoup plus grande qu'on ne croyait. Une solution innovatrice fut trouvée qui évita la construction d'un nouveau pont et réalisa d'importantes économies pour le client.

ZUSAMMENFASSUNG

Die Freistaat Provinz von Südafrika wurde 1988 von verheerenden Überschwemmungen heimgesucht. Die alte Bogenbrücke über dem Sand River in der Nähe von Virginia, 1926 gebaut, wurde ganz überschwemmt. Die Auffahrten wurden weggespült, und der Oberbau der Brücke wurde schwer beschädigt. Die Planung von Strassen in der unmittelbaren Nähe der Brücke sieht eine neue Konstruktion über dem Sand River vor, also eine Ersetzung der alten Brücke. Eingehende Untersuchungen zeigten, dass die Brücke mehr Tragfähigkeit, als früher vermutet, besass. Eine innovative Lösung wurde gefunden, die den Bau einer neuen Brücke vermied und zu erheblichen Kosteneinsparungen für den Kunden führte.

1. INTRODUCTION

During February and March 1988 the Free State province of the Republic of South Africa was hit by devastating floods. The provincial road network was disrupted to such an extent that Bloemfontein, the provincial capital, was cut off from the rest of the country for several days. Although several bridges were completely washed away, the approaches of 20 other bridges were also washed away. The biggest problem was the reconstruction of the bridge approaches to re-establish road links.

One of the bridges which was damaged during these floods was the historic Sand River Bridge No.96 near Virginia which was built in 1926. The importance of this bridge lies in the fact that it forms part of the link between the Gold Fields of the Free State and Lesotho which provides labour for the mines. Owing to a sharp increase in traffic volumes, the route was to have been upgraded as a matter of course. The planned new road alignment also made provision for the construction of a new bridge which would have made the existing bridge obsolete.

The prohibitive cost of a new bridge resulted in a closer investigation into the feasibility of repairing and untilising the existing structure rather than building a new bridge at a greater cost.

2. EXISTING BRIDGE

2.1 General

The existing structure was built in 1926 and consequently very little as-built information was available. No structural drawings could be located and the only drawing found was a location plan which did not even show the bridge position. The bridge is of an arch-type construction and due to its age is deemed to be historic.

2.2 Founding conditions

A visual inspection of the bridge site revealed that the bridge piers were founded on sandstone rock. Sandstone was also found at various locations in the river bed in the vicinity of the bridge. Based on the visual assessment and the absence of any scour in the river bed, it could be assumed with reasonable certainty that the bridge was firmly founded on rock and further geotechnical investigations were deemed unnecessary. The estimated safe bearing pressure of the sandstone is 1000 kPa.

2.3 Structure

The sub-structure consists of mass concrete piers, abutments and wingwalls. The total length of structure is approximately 90 metres, made up of nine arch spans of reinforced concrete. Mass concrete walls constructed on the sides of the arches kept the rubble infill placed on the arches in place. The single roadway was provided by placing premix on the infill and constructing sidewalks on the walls. Steel handrailings were added. The total height of the structure above the river bed was 11 metres and the bridge width was 4.6 meters with an effective roadway width of 3 metres.

An interesting feature of the bridge was the fact that the two outside openings between the abutments and the piers were closed up by means of concrete walls. The reason for this is unknown. It is presumed that the abutments needed structural support but the closure of the openings would definitely have had a negative influence on the hydraulic capacity of the structure.

3. FLOODS OF 1988

3.1 Extent of floods

The flood in the Sand River was not investigated by the Department of Water Affairs as it was not deemed to be as severe as the flooding in the rest of the province. Hence it is dificult to give accurate figures. However some calculations were made and are summarised :

Hydrological data:

Catchment area:	6113 km^2
Average slope :	,57 m/km
Average rainfall:	615 mm / year

In order to determine the magnitude of the flood in the Sand River all available data was analysed including the level of water during the floods which could accurately be determined from eye witness accounts and debris. It was determined that the floodwater overtopped the bridge by approximately one metre.

The hydraulic and hydrological investigations revealed that the flood which overtopped the bridge was in the order of 2650 cumecs, which constitutes a flood with recurrence interval of more than 50 years. Recurrence intervals of more than 200 years were recorded in the rest of the province.

3.2 Damage to bridge

The bridge was overtopped by the floods and consequently the approaches were washed away, allowing water to flow around the structure as well. Furthermore the water removed large portions of rubble infill on the arches which effectively made the bridge impassable. Apart from the damage described, tonnes of debris, including large trees, were left on the bridge.

An inspection of the bridge after the floods showed that the sub-structure, in contrast to the super-structure, showed no signs of damage or structural distress. Although cracks were found in the wingwalls and arches it could not be proven to have been caused by the floods.

The fact that the bridge withstood the flood forces indicated that the structure posessed far greater inherent strength than the visual appraisal indicated. The existence of large cracks in the arches and wingwalls also indicated that the existing structure should not be subjected to any additional loads.

4. REHABILITATION OF THE EXISTING BRIDGE

4.1 General background

Due to severe cuts in financial budgets for roads in the province, the possibility of utilising the existing structure in order to obviate the construction of a totally new bridge downstream had to be investigated. The cost of a new bridge was estimated at $ 800 000. The client was willing to accept recommendations regarding speed restrictions and lower geometrical standards if necessary.

4.2 Design criteria

The rehabilitated structure had to comply with the following :

- The structure had to accommodate two-way traffic.
- Pedestrian traffic had to be accommodated.
- Minimum costs were to be incurred on the demolition of the existing structure.
- The existing sub-structure had to be used and no additional loading onto super-structure was allowed.
- The cost of strengthening the existing structure had to be minimised.
- The existing wingwalls were not to be built higher.
- The structure had to be designed to withstand a 50 year flood as well as NA & NB24 loadings.

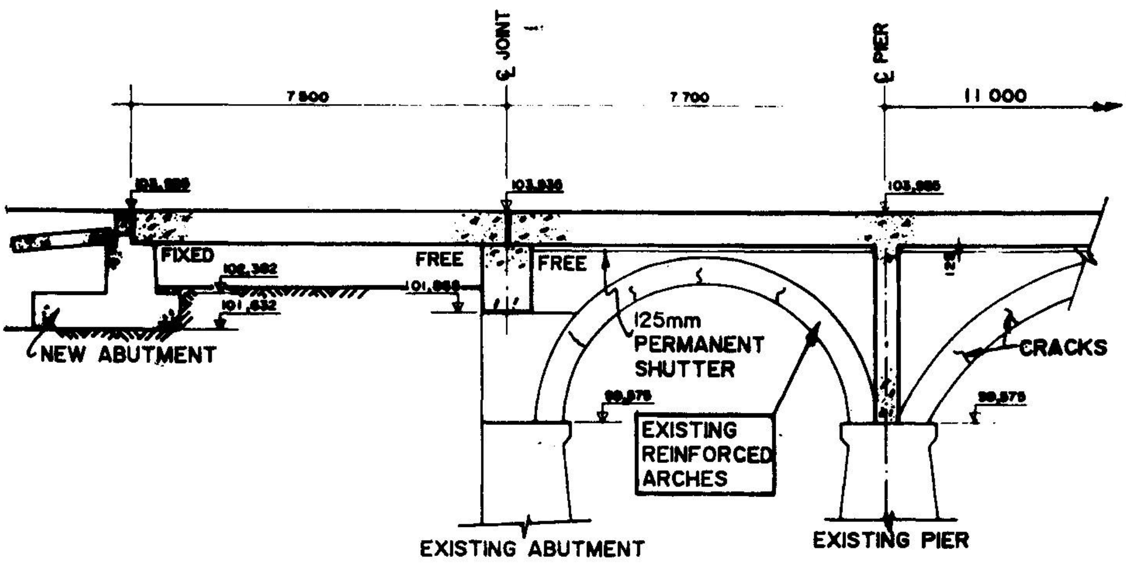
- The minimum cross-section of the deck was :
  - 2 x 3.1 metre lanes.
  - 2 x 0.3 metre shoulders.
  - 2 x 0.425 metre New Jersey balustrades.
  - 1 x 1.2 metre walkway for pedestrians.
  - Total width = 8.85 metres.

4.3 Final structure

In order to comply with the abovementioned criteria a structural solution was proposed and subsequently built which made optimal use of the existing structure while still providing a functional and economical alternative to a new bridge.

The solution will now be discussed briefly with reference to Figure 1.

The existing sub-structure was kept intact while a new continuous reinforced concrete deckslab, supported by piers on the existing piers, was added. All rubble was removed from the existing arches leaving the side walls intact. The new piers were designed to be cast composite with the deckslab forming a portal structure which rested on the old piers. A continuous structure was needed to ensure the stability of the new super-structure due to the relatively large cantilevers needed to provide for the total deck cross section.



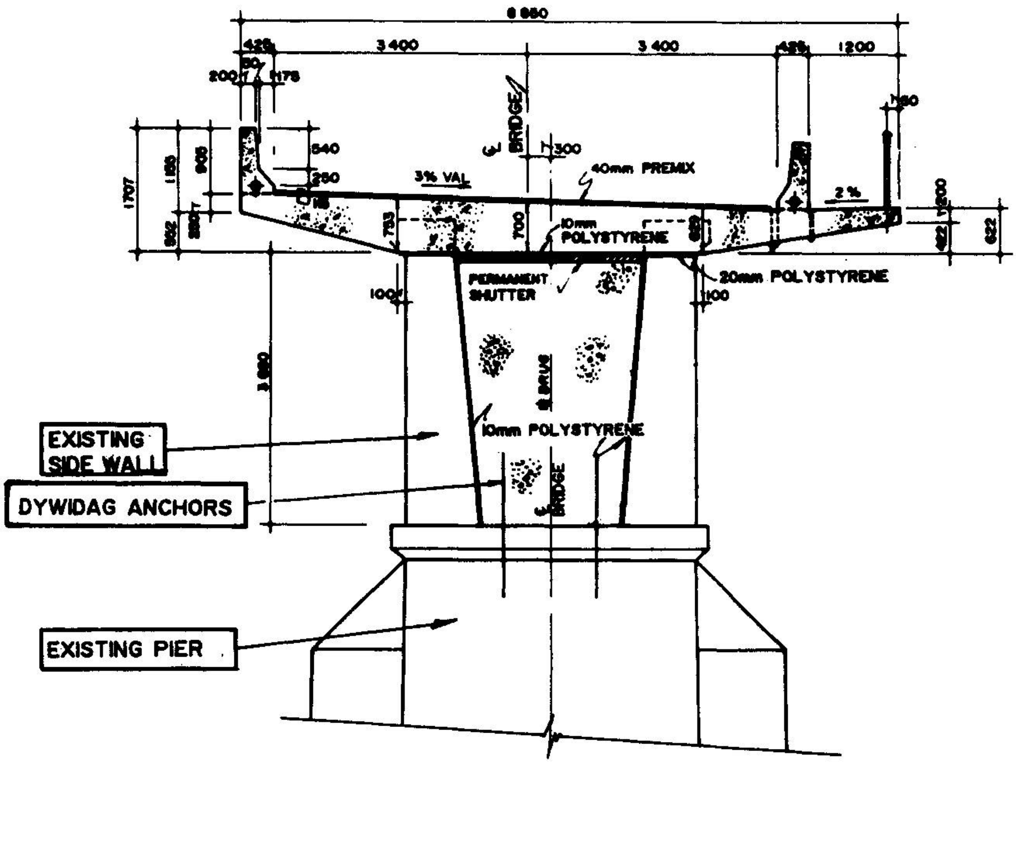


FIGURE I

TYPICAL SECTION

The new piers were designed as pinned joints resting on the existing piers and fixed by means of unstressed Dywidag bars grouted into the existing concrete. To ensure a minimum load transfer onto the existing structure, care was taken in providing for movement gaps between the old and new structure.

In order not to extend the existing wingwalls vertically, it was decided to rather build two additional short spans on either side of the bridge. These spans are simply supported and rest on the existing abutments and new abutments founded on the reconstructed approach fill. Expansion joints were provided at the ends of the portal structure.

Before construction commenced and after the rubble was removed from the structure, cores were taken at all the existing pier positions to determine the in situ strength of the supporting material for the new piers. Results obtained showed the existing concrete to have a minimum strength of 20 MPa.

4.4 Construction

Numerous problems were experienced during construction. The biggest problem was contending with the flow of water and necessary stagework needed to shutter the large cantilevers. Due to the lack of as-built data the actual dimensions of the structure once cleared of rubble and debris differed from the design assumptions to such a degree that it was necessary to re-analyse and adapt certain dimensions and reinforcing. A special support system for the cantilever shuttering had to be designed to ensure that no additional loads were placed on the arch walls. The formwork had to be precambered to compensate for deflections during concreting.
It was however the team work from all parties involved which ensured that the project was successfully completed on time and within budget with no claims from the contractor.

The final cost of rehabilitating the structure was $125 000 and was completed within six months and within the original estimates and tender price.

5. CONCLUSION

South Africa, like many other countries in the world, is experiencing a lack of funding for road construction and maintenance. This means that innovative solutions will have to be found to provide the much needed infrastructure to ensure a growing economy. This project, even though it does not classify as a major engineering achievement, does however show that through innovative engineering large savings or greater return on investment is possible.
This solution not only saved money but also provided an aesthetically and functional alternative. The principles adopted can be used in many other larger projects and could provide for substantial savings.

Rehabilitation of the Gardiner Expressway, Toronto, Canada

Réparation de l'autoroute Gardiner à Toronto, Canada

Instandstellung des Gardiner Expressway, Toronto, Canada

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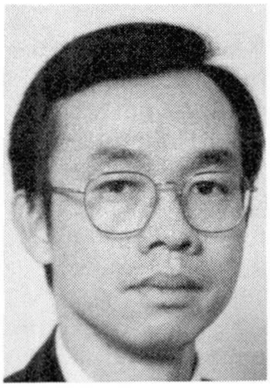

Edward Li, born in 1953, obtained his degree at the University of Hong Kong in 1976. He joined Morrison Hershfield in 1989 and is responsible for designing and managing bridge projects for the firm, including the F.G. Gardiner Expressway Rehabilitation Project.

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Carl Woodman, born in 1945, received his civil engineering degree from the Technical University of Nova Scotia in 1968. He joined METRO Transportation in 1969, and has been responsible for many of the F.G. Gardiner Expressway maintenance and rehabilitation projects.

SUMMARY

The Gardiner Expressway is the major link to the downtown financial district of Toronto, Canada. After 16 years of service, the 8 km elevated structure of the Expressway already required considerable structural repairs. As the condition of the structure continued to deteriorate, major work was required to extend the life of the structure. This paper describes the strategic planning of the rehabilitation programme and the adoption of a repair method that involved a significant alteration of the original structure.

RÉSUMÉ

L'autoroute Gardiner est la liaison principale du centre ville de Toronto. Après 16 ans d'utilisation, les 8 kilomètres de la voie surélevée ont nécessité de nombreuses réparations. L'état de la structure continuant à se détériorer, il a fallu entreprendre des travaux considérables pour prolonger la vie de cette autoroute. Ce document explique les moyens stratégiques utilisés dans le programme de reconstruction et l'adoption des modalités pour les réparations requises. Les moyens utilisés impliquent des modifications importantes à la structure actuelle.

ZUSAMMENFASSUNG

Der Gardiner Expressway ist eine Hauptverkehrsader zum Finanzzentrum in der Innenstadt von Toronto, Kanada. Die 8 km lange Hochstrasse wurde bereits nach 16 Betriebsjahren reparaturbedürftig. Kostspielige Reparaturen wurden daraufhin ausgeführt, jedoch der Zustand verschlechterte sich weiterhin. Dieser Bericht beschreibt die strategische Planung für das Sanierungsprogramm und die Anwendung einer Reparaturmethode, die eine bedeutende Umänderung der ursprünglichen Baustruktur erfordert.

1. INTRODUCTION

1.1 The F.G. Gardiner Expressway in Toronto located on the north shore of Lake Ontario was built in the early sixties. Figure 1 shows a bird's eye view of the elevated portion of the Expressway. The most prominent feature of the 14 km long Expressway is an 8 km elevated roadway, composing of mainly simply supported spans resting on concrete bents. A 6.2 km length of this elevated roadway carrying a major portion of the traffic became the centre of the subsequent rehabilitation study. The main deck of this section was made up of 507 spans with a total deck area of 240,000 square metres. These spans were principally concrete slabs on steel I-girders (79%), concrete overlay on prestressed concrete box girders (16%), or in-situ concrete slab and beams (5%). An expansion joint was generally located at every supporting bent.

1.2 The ineffectiveness of the waterproofing membrane and leaking joints did not provide adequate protection against de-icing salt attack. This resulted in the rapid deterioration of concrete and steel members. Corrosion of steel reinforcement led to severe cracking and concrete delamination, which in time turned into extensive spalling. Falling concrete fragments became safety concerns to drivers, pedestrians and adjacent properties. Additionally, the deteriorating condition of concrete and steel components was substantially reducing the capacities of the structural members.

1.3 Beginning in 1972 the Transportation Department carried out various maintenance efforts to repair and prevent further damage caused by salt corrosion. These efforts included expansion joint replacement, asphalt resurfacing, concrete patching and shotcreting. However, it was recognized that continuation of maintenance activities only was insufficient to address the increasing deterioration problems. In 1984, the 1 km section of the elevated roadway on concrete box girders, which showed the worst signs of deterioration, was repaired. Details of the repair can be found in [1]. It was then realized that a long term repair strategy was required.

2. THE MAJOR INVESTIGATION AND THE STRATEGIC PLAN

2.1 To develop the strategic plan, an investigation was carried out in 1985 [2]. The purpose of the investigation was to determine the condition of the structure, to formulate the technical solutions to the problems, and to estimate the quantities of work involved in the rehabilitation. A representative section of the elevated roadway utilizing deck-on-steel-girders was selected for surveying. The investigation took three months to complete, encompassing extensive visual inspections, corrosion potential measurements, sawn asphalt samples, coring and chloride content tests.

2.2 The Strategic Plan for the rehabilitation of the elevated section of the Expressway was prepared and approved by Metropolitan Toronto Council in 1986. The study, considering the previous work undertaken and the experience gained, developed a long-range rehabilitation program, to maximize the life of the structure and to minimize the future maintenance cost of the Expressway [3]. Recommendations of the Strategic Plan included:

1. A strategy by which consecutive events should follow with cost estimates for a period of 40 years;

2. A 20-year Construction Programme within which the main roadway deck would be repaired in 9 years; and

3. A principle of operation, which established the priority of rehabilitating the deck as the first activity.

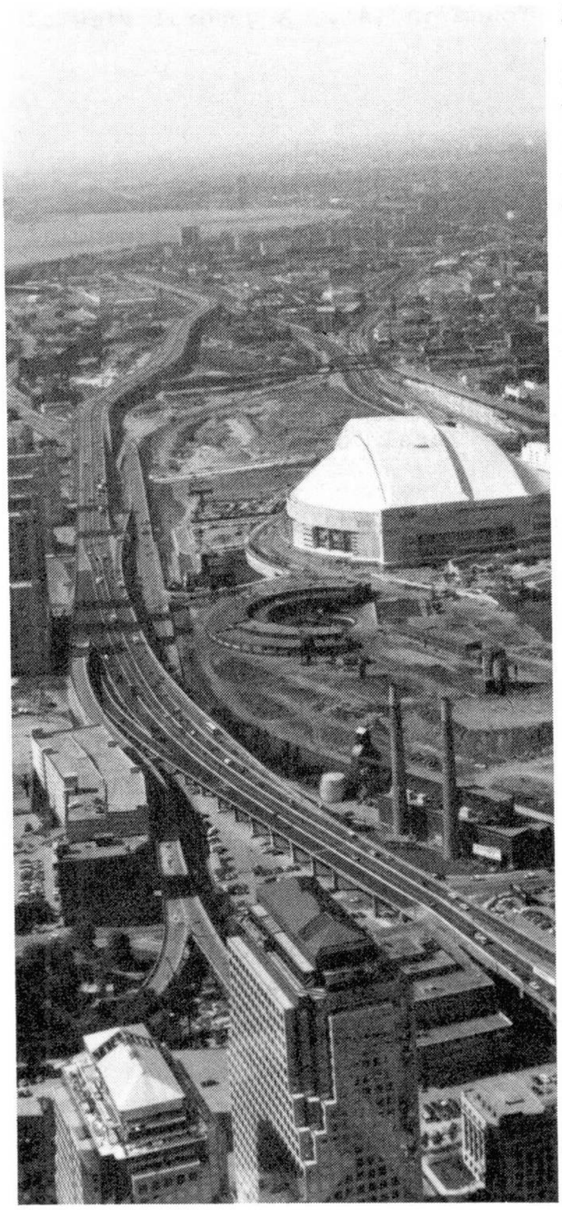

Figure 1
The F.G. Gardiner Expressway
Toronto, Ontario, Canada

3. DESIGN

3.1 The design was to produce a watertight deck to protect the substructure from further salt damage. In addition, after repairing all of the existing defects, the deck had to provide a service life of at least 15 years before further rehabilitation would be required.

3.2 To accomplish this, it was decided to minimize the number of existing joints . The longitudinal joint between the twin bridges was replaced with a concrete slab and additional cross-frames, essentially tying the 2 bridges together. Three out of four existing expansion joints were eliminated by making the deck continuous for a number of spans. The tying of originally simply supported spans required complete revamping of the bridge articulation system, by providing new bearings and expansion joints. Thermal movements of the tied deck were accommodated by flexing of the supporting bents, while ensuring that the existing fixed bearings would be strong enough to withstand such movements. The tying method chosen was called the "flexible link", which consisted of a thin and highly reinforced concrete slab. This link allowed the connecting spans to continue to behave as simply supported spans, with the end rotations due to traffic loading being accommodated by the flexing action of the link. A more detail description of the link slab can be found in [4]. Such re-arrangement of structural layout in rehabilitation projects was the first designed and built in Canada. All remaining expansion joints were replaced with joints incorporating readily replaceable seals with steel armouring.

3.3 An innovative "flood-free" drainage system was devised. The system utilized a fibre-glass hopper mounted on either the girders or the supporting bents. The hopper collected silt and allowed sediment-free water to enter the main drainage network. The inlet was designed to discharge water as quickly as possible and to accommodate a "vacall" type hose for the periodic removal of silt. At those times when the hopper was filled with silt, water could still drain off the deck by overflowing at the hopper.

3.4 The rehabilitation also involved

1. Repairing damaged deck and parapet areas, by patching and refacing;

2. Incorporating a new bituminous waterproofing layer and asphalt surface;

3. Reconditioning bearings to re-activate their movement capabilities.

3.5 Some of the above design details can be found in [4]. A general view of the repair scheme is shown in Figure 2.

4. TRAFFIC MANAGEMENT

4.1 The Expressway is a major East-West link across the City of Toronto. It also serves the centre of activities of this busy city, including the financial district and the SkyDome, one of the most popular sports complex in North America. Every contract had to be well planned to ensure the least disruption to the public. The mandate of these plans was to keep at least two out of the three existing traffic lanes open in each direction at all times, and to:

1. Ensure safety for drivers by providing adequate signing, sight distances, pavement markings etc. according to the local construction traffic guidelines;
2. Co-ordinate with other construction work to minimize impact;
3. Ensure sufficient structural capacity during any working stage, and provide adequate spaces for the lapping of reinforcement, waterproofing and asphalt;
4. Build additional working and detouring space for next contract by repairing areas beyond original limits of contracts;
5. Obtain approvals from various authorities, and carry out test drills for emergency conditions that could occur within the site;
6. Hold liaison meetings to keep all involved parties informed of developments, necessary closures, etc.; and
7. Publish pamphlets, notify media and, through telephone hot-lines, advise the public of the work, possible delays and recommended detours.

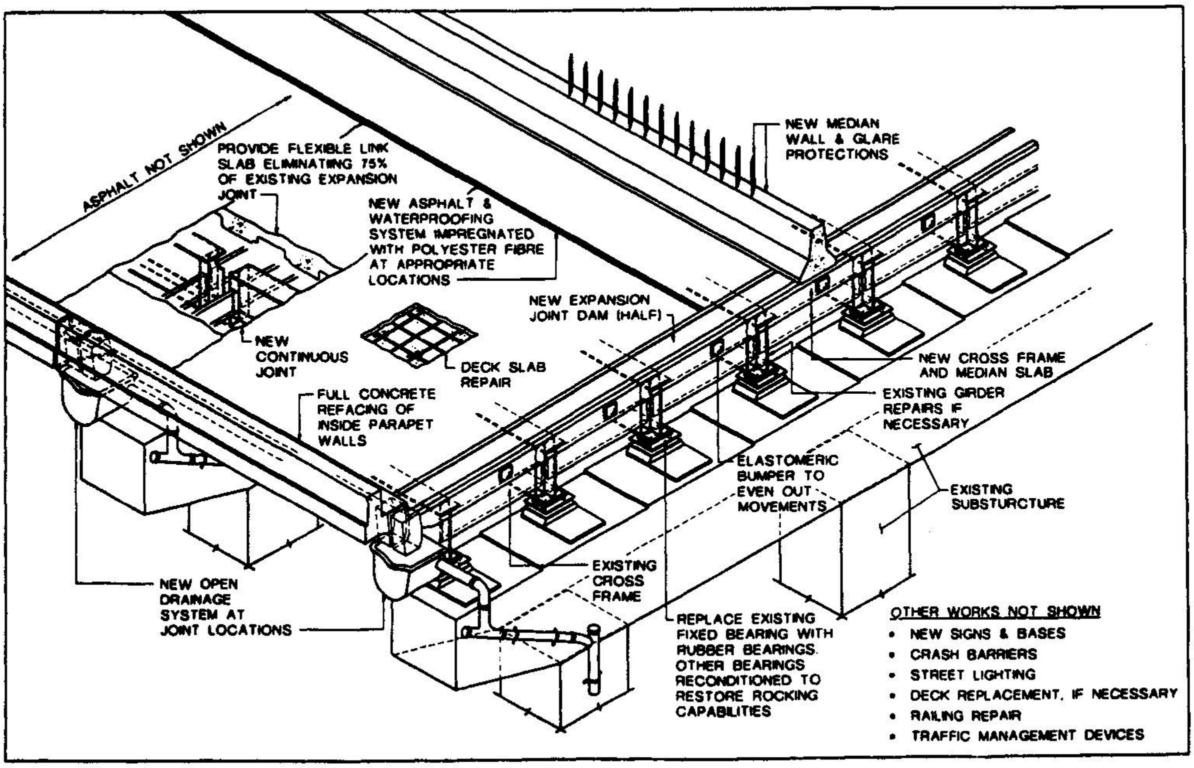

Figure 2 - General Deck Rehabilitation Scheme

5. CONSTRUCTION

5.1 In 1986, a pilot contract was tendered to gain experience and refine the details of the main innovations. From 1987 to 1995, the 1986 rehabilitation techniques were used to tender one major deck repair contract every year, for a total of 9 contracts. The annual expenditure and the approximate cost of repair per square metre of the deck are shown in Table 1.

TABLE 1

Year	Contract Sum	Deck Area Repaired	Cost / Sq.M.
1987	C$ 5,450,000	35,700 sq.m.	C$ 153
1988	C$ 5,900,000	25,600 sq.m.	C$ 230
1989	C$ 6,900,000	23,100 sq.m.	C$ 299
1990	C$ 6,542,000	21,900 sq.m.	C$ 299
1991	C$ 5,555,000	19,600 sq.m.	C$ 283
1992	C$ 5,493,000+	18,300 sq.m.	C$ 300
1993	C$ 5,159,000*	9,300 sq.m.	C$ 554
1994	C$ 1,850,000**	7,700 sq.m.	C$ 240

\+ plus C$ 0.8M for substructure repairs * includes C$ 4.7M for deck replacement work

**an additional C$ 0.4M was spent on substructure repairs

5.2 Construction began with rehabilitation of bearings by rust removal, new coatings, and bearing replacements. Repairing the bearings required slight jacking of all girders connected by cross-frames, and the erection of access platforms to carry out the work. Tying of the twin bridges together using additional cross frames was also carried out in this stage.

5.3 On the deck, stage 1 of the traffic detour was implemented. Work zones, which normally started within the median area, were delineated by concrete barriers, with appropriate signage and markings. Asphalt was stripped exposing concrete deck for sounding and repair. After removing the required deck areas, construction of new continuous deck (the flexible link slab), the expansion joint dams and the median slab were cast, with all necessary splicing details in place. At the same time, delaminated areas were removed and patched. On completion of all concrete and steel repairs, new waterproofing and the first layer of asphalt were then laid. The work was then switched to Stage 2 of traffic detour, and executed in the similar sequence. At this stage working was confined to the curb lanes where the joints were reconstructed along with the replacement of existing drains. Additionally, the inside face of the existing parapet walls were rehabilitated and surface-sealed. Stage 3 of the detour, involving work on the opposite curb lane of the bridge, followed and was executed in a similar fashion. Lastly, in Stage 4, the median lanes were re-occupied for the construction of median barrier walls and glare protections.

5.4 On a pre-arranged closure, the concrete barrier walls, for construction safety, were removed, the seals for the expansion joints were installed in complete pieces and the entire surface asphalt was laid in one operation. Traffic markings, lighting and electronic traffic management devices were installed / activated and the full width of the elevated roadway was opened to traffic. Under-deck work such as drains and pipes continued until final completion.

5.5 Some of the unanticipated new problems arose in the course of the rehabilitation program were:

1. Rubber membranes, normally sandwiched in the waterproofing to provide tensile strength where movement in the deck was expected, deformed. It was found that upon overlaying with hot waterproofing, the rubber deformed and wrinkled (known as the "braining" phenomenon), causing severe cracking in the asphalt pavement. The rubber membrane was subsequently replaced by polyester fibre sheets.

2. In many situations, the deterioration of the concrete bents required specific seating details for the jacking of girders. These problems were normally overcome by constructing additional steelworks attached to the existing columns for the jacking operation. At low bents and at abutments, jacking was sometimes done against kentledges laid on the ground.

3. The major ramps connecting the F.G. Gardiner Expressway with another expressway required complete deck replacement. A condition survey, carried out prior to tendering determined that up to 70% of the deck was delaminated, as well as on major portions of the parapet wall. Traffic was diverted to a single lane in each direction on one of the connecting ramps while the other was reconstructed. In addition other traffic had to be detoured through adjacent streets. While this rehabilitation provided a challenging and costly traffic diversion scheme, it afforded an opportunity to widen the connecting ramps and improve sight distances. The widening also provided additional space for disabled vehicles. The deck replacement is expected to have a life of at least 50 years.

6. CONCLUSION

6.1 By fall 1995 the main deck of the Expressway as identified in the Strategic Plan will have been rehabilitated, meeting the target set out in the Plan. Regular inspections on the new details such as the flexible link slabs and drainage works, some installed over 7 years ago, have shown no sign of cracking, leaking or distress. The total contract value of C$49.4 M (including deck replacement of the connecting ramps in 1993) for the past 9 contracts agrees well with the original estimate of C$44.7 M (before inflation adjustment). Pilot projects for the substructure repair have already started, and trial projects for new expansion joint systems have also been carried out. However, these are beyond the scope of this paper.

6.2 The writers would like to express sincere gratitude to the late Mr. Andres Tork who had contributed so much to the success of this project, and to **METRO**Transportation Department of Toronto and Morrison Hershfield Limited, who have allowed us to publish the information contained in this paper.

REFERENCES

1. TORK, A., Rehabilitation of Prestressed Concrete Box Beams Deck of an Elevated Roadway, The Canadian Journal of Civil Engineering, Feb., 1989.

2. Morrison Hershfield Limited, Report on F.G. Gardiner Expressway Jarvis Street to Don Valley Parkway, February, 1986

3. Morrison Hershfield Limited, Strategic Plan for the Elevated Section of the F.G. Gardiner Expressway CNE Grounds to Don Valley Parkway, September, 1986

4. TORK, A., Rehabilitation of Steel Girder Deck of an Elevated Expressway, 3rd International Conference on Short and Median Span Bridges, 1990, Toronto, Ontario, Canada

Renovation and Transformation of the Pérolles Bridge in Fribourg

Rénovation et transformation du pont de Pérolles à Fribourg

Renovation und Nutzungsänderung der Pérollesbrücke in Freiburg

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SUMMARY

The rehabilitation work performed on the Pérolles Bridge in Fribourg, Switzerland, is more than a simple refurbishment. The renovation represents a comprehensive transformation. By retaining the infrastructure, it will be possible to modify and improve the use of the bridge. The width of the deck will increase from 10m to 17.5m, which gives an indication of the increased performance needed to satisfy the new service conditions. Adequate design and construction procedures, in particular the detailed structural analysis for better understanding of the behaviour of the spatial structure, and the use of a temporary bridge during construction, will increase the lifespan of the bridge by approximately 80 years.

RÉSUMÉ

Les travaux de réhabilitation du pont de Pérolles à Fribourg, Suisse, ne se limitent pas à la simple remise en état. La rénovation constitue l'occasion d'une transformation. Par la réutilisation de l'infrastructure, il est possible de modifier et d'améliorer les caractéristiques du pont. Le tablier passe d'une largeur utile de 10.0m à 17.5m, ce qui montre le gain de performance en vue de son nouveau service. Le concept et la mise en oeuvre de mesures adéquates, notamment la méthode de construction au moyen d'un pont provisoire, et l'analyse structurale avancée permettant de connaître le comportement de la structure spatiale, prolongeront la durée de vie de l'ouvrage d'environ 80 ans.

ZUSAMMENFASSUNG

Die Erneuerung der Pérollesbrücke in Freiburg, Schweiz, beschränkt sich nicht nur auf eine Wiederinstandsetzung, sondern dient gleichzeitig auch als Grundlage für eine Nutzungsänderung. Durch die Wiederverwendung der Infrastruktur ist es möglich, die Eigenschaften der Brücke zu verbessern. Die Fahrbahnplatte wird von 10.0 auf 17.5 Meter verbreitert, was eine Verbesserung der Benutzung mit sich führt. Das Konzept und die Ausführung der geeigneten Massnahmen, unter anderem das Bauverfahren mit Hilfe einer provisorischen Brücke, sowie die erweiterte Tragwerksanalyse, die es erlaubt, das Verhalten des räumlichen Tragwerks zu erfassen, führt zu einer Verlängerung der möglichen Nutzungsdauer von ungefähr 80 Jahren.

1 INTRODUCTION

In 1922, the Pérolles bridge, situated at the entrance to Fribourg was the longest bridge in Switzerland. It had a span of 554 m, a height of 70 m and consisted of 40,000 m^3 of concrete. Only the deck slab was reinforced concrete (Figure 1).

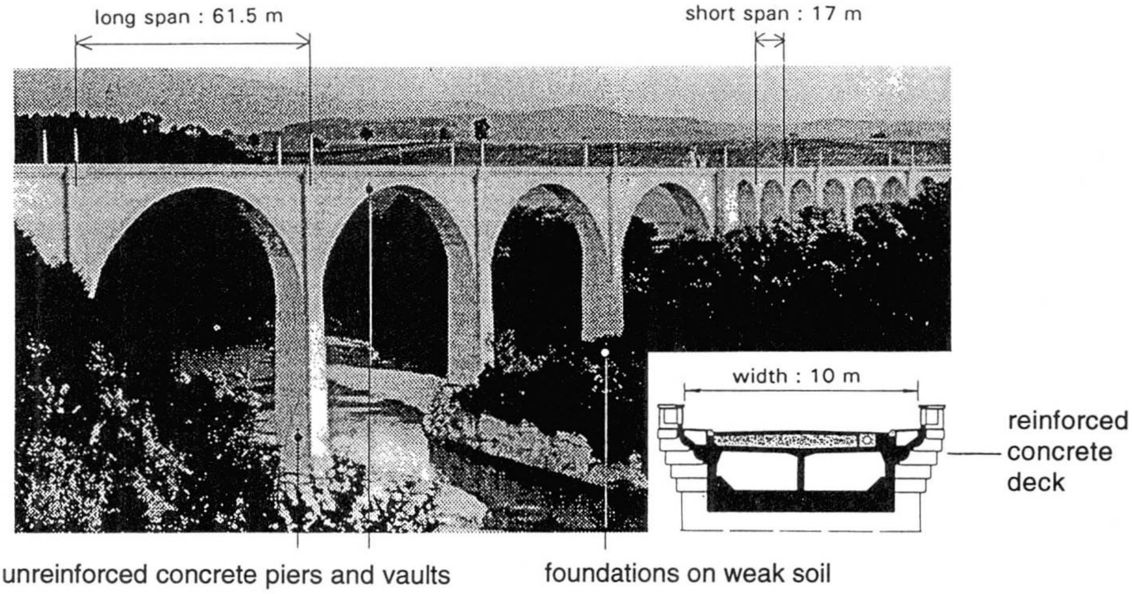

Figure 1 : Main features of old bridge

A detailed inspection of the bridge revealed severe degradation of the deck slab. The bridge had reached the end of its lifespan and consequently the deck slab had to be replaced.

Based on the evaluation of the existing bridge and the implications of such a large project, another serious question was raised : would it be possible to augment the resistance of the bridge to meet new demands? [1]

This paper considers the overall concept and the implementation of appropriate measures during the renovation and transormation of the Pérolles bridge in Fribourg.

2 PLANNING AND ORGANISATION OF THE PROJECT

Before effecting important changes on a structure, it is imperative that special attention be given to the global aspect of the problem, starting with the maintenance records of the bridge from when it was just put into service, and finishing with its renovation (Figure 2) [2].

A reconstruction of the history of the bridge, an understanding of the real behaviour of the structure, and a diagnostic of the present state of the bridge contribute to the selection of a concept, and the measures to adopt, in order to improve the characteristics of the bridge.

The adaptability of the bridge depends on the choices made during the conceptual phase and the construction phase. These choices influence the reserve in capacity, and create flexibility in terms of the new needs.

Renovation and transformation gives the structure a superior conformance to the requirements of the the SIA (Swiss Association of Engineers and Architects) standards and the applicable rules of common practice [3].

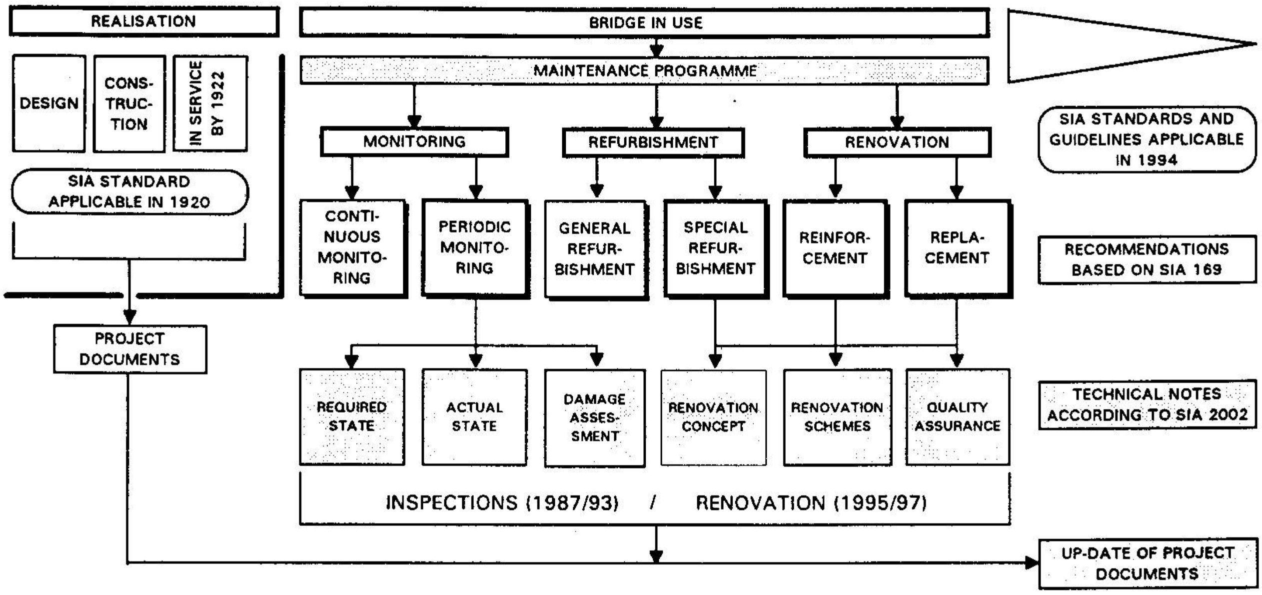

Figure 2 : Project organisation and planning

3 RENOVATION AND TRANSFORMATION

3.1 Concept

The general concept which influenced the renovation and transformation of the bridge is as follows :

- **optimise** use of existing structure as much as possible
- **satisfy** construction requirements through use of a temporary bridge
- **replace** the dammaged superstructure by one which is 70% larger
- **retain** the undamaged infrastructure but provide adequate reinforcement
- **preserve** the character of the bridge which is a part of Swiss heritage
- **ensure** high quality in terms of durability [4] [5]

Before the final concept was defined, another factor which needed consideration was the minimal disturbance to the existing structure.

The cross-section of the renovated and transformed bridge illustrates the modifications and improvements to the existing structure (Figure 3).

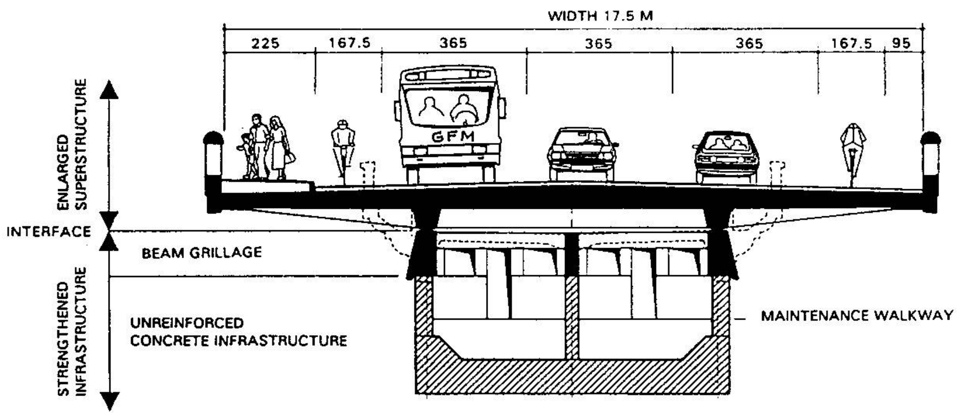

Figure 3 : Cross-section of renovated and transformed bridge

3.2 Course of action

Amongst the appropriate measures for prolonging the lifespan of the bridge, the choices of the construction conditions and method, and of the structural analysis approach were essential in order to satisfy requirements concerning respect of the environment, quality, cost and deadlines.

3.2.1 Conditions and construction method

The use of a temporary bridge provides optimal construction conditions by creating an independent site.

The construction method is based on the following criteria :

- **satisfy** safety requirements
- **ensure** concurrence and reliability of operations
- **maximise** use of repetition
- **reduce** complexity of procedures and introduction of new materials
- **monitor** quality-control, running costs and site schedule

Project management involves six construction phases as illustrated in Figure 4.

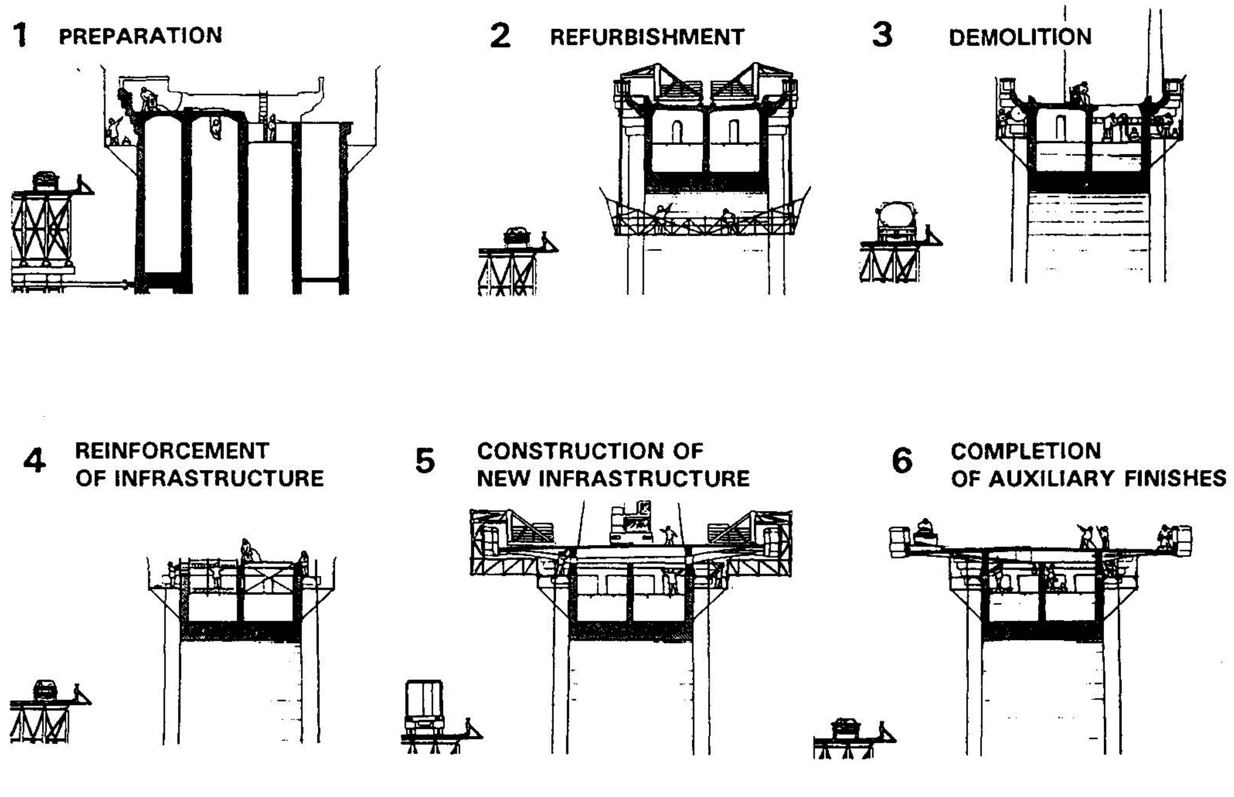

Figure 4 : Six construction phases

Interestingly, the construction method involves phases which require different loading conditions from those to which the completed bridge will be subjected.

3.3 Structural analysis

3.3.1 Evaluation of structural capacity

The construction of the vaults governs the design of the structure for the renovation of the bridge. The static model adopted at the time was a fully-connected arch, loaded by the fresh concrete of the walls and of the superstructure. The evaluation of the effective structural capacity, which fortunately had sufficient reserve, lead to the space frame analysis involving "arch, walls and deck-slab". Only the finite element method, through use of a precise model of the structure during the construction phases, provided a global approach to the problem by localising the critical zones. The MAPS computer program was retained for the comparative analyses performed by IBAP/EPFL (Figure 5).

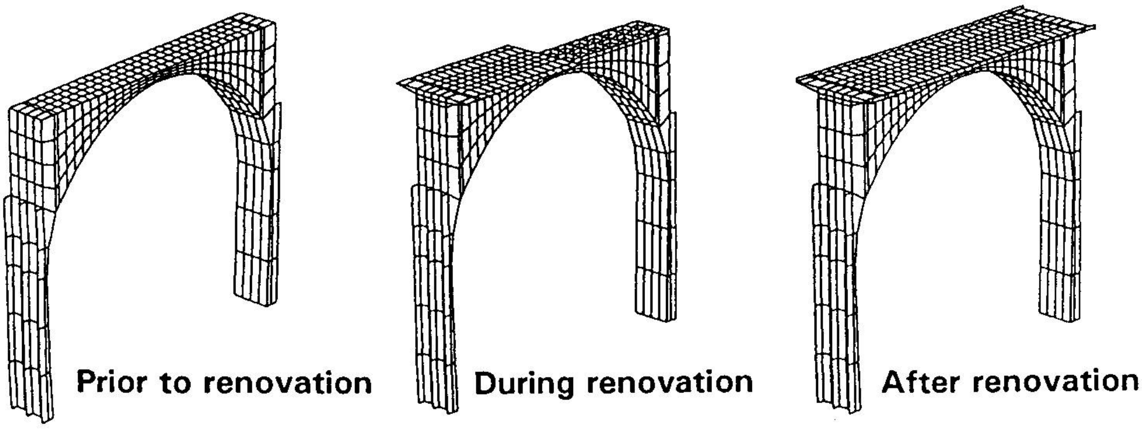

Figure 5 : Evolving structural models for a 60.5 m span according to the state of renovation

A parametric finite element analysis opened new avenues and lead to an optimal design of the structure. Measurements *in situ* contributed to a better understanding of the real behaviour of the structure and helped validate the reliability of the model (Figure 6).

3.3.2 Calibration of the model

The calibration was based on a large set of simultaneous temperature and deformation measurements of the structure. Comparison with the model helped to refine the material properties and the structural model (Figure 6).

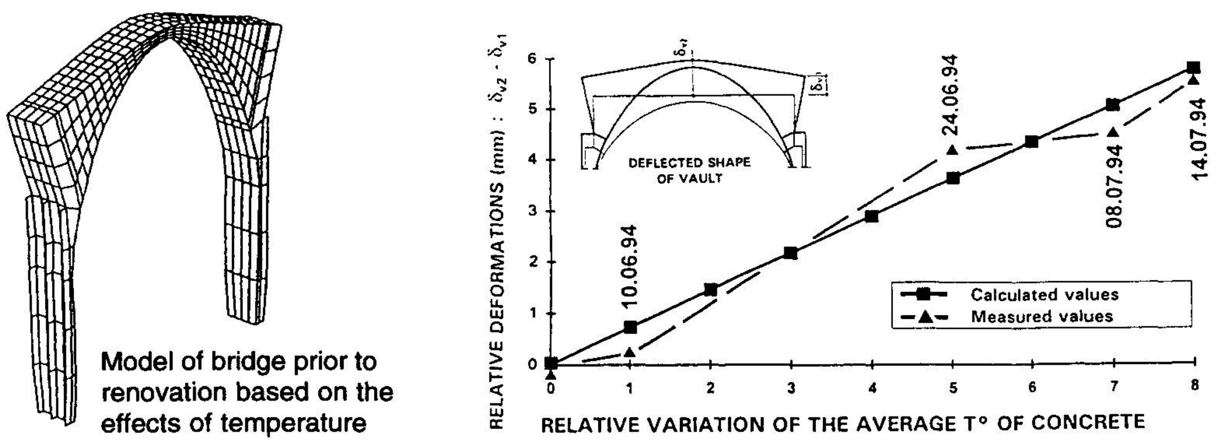

Figure 6 : Calibration of the model based on the effects of temperature

3.3.3 Identification of critical stress zones

The actual stress distribution in the infrastructure serves as a reference. The structural concept creates a minimum of disturbance to the unreinforced concrete elements. The evolution of the stresses during the renovation process was simulated in order to identify critical zones vulnerable to dammage. The set of tests performed by the LMC/EPFL laboratory, and the observations of the behaviour of the unreinforced concrete structure served as the basis for fixing a design value for the biaxial stress resistance in the vault, taking into account the discontinuous surface conditions of the concrete (Figure 7). The beam grillage is the only critical zone. It represents the only area reinforced concrete elements are used for strengthening the infrastructure. The non-linear analysis helped verify the cracking resulting from the imposed deformation restraints [6].

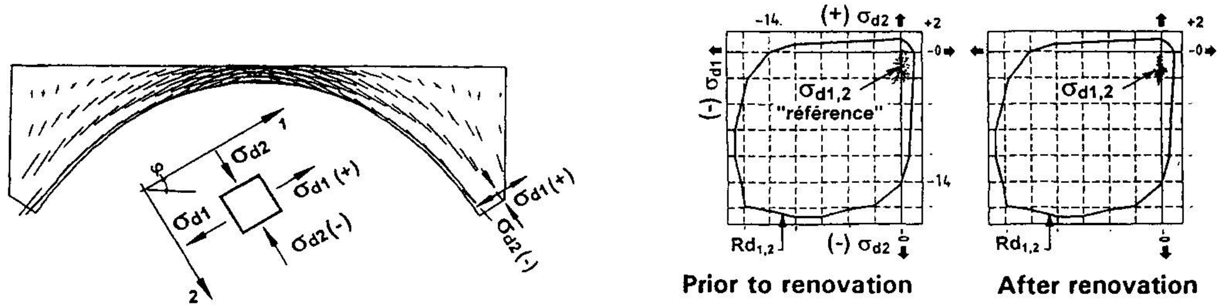

Figure 7 : Principal stress distribution in the vault prior to and after renovation, biaxial stress resistance (R_d) and normal stresses (σ_d)

4 CONCLUSIONS

For the renovation and transformation of the Pérolles Bridge in Fribourg, the selection of the concept and the appropriate construction measures, notably the method of construction using a temporary bridge and the advanced structural analysis procedure, helped optimise a set of criteria. These are : respect for the environment, aesthetics, reliability, durability, safety, deadlines and costs. The rehabilitation work will increase the lifespan of the bridge by 80 years.

5 PARTICIPANTS

Owner :	Direction of Public Works, Bridge Division, Canton of Fribourg
Design/Build team :	Consortium
• Renovation	Bongard & Zwick Consultants
• Temporary bridge	C. von der Weid Consultants
Expert :	H.-U. Frey, Ing. dipl. EPFL/SIA, Frey & Associates, Lausanne.

6 REFERENCES

[1] Kunz, P., Bez, R., Hirt, M.A. : *L'évaluation des structures existantes.* Ingénieurs et architectes suisses, Lausanne. 5/1994

[2] SIA 160, 162 (partial revision in 1993) and SIA 169 (1989 edition). Standards of the *Société Suisse des Ingénieurs et Architectes*, Zürich.

[3] Office Fédéral des Routes : *Détails de construction de ponts : directives*, Berne 1990.

[4] Favre, R., Andrey, Dl, Sutter, R. : *Méthodologie de surveillance*, EPFL/IBAP, Lausanne 6/1987

[5] OCDE : *Durabilité des ponts routiers en béton.* OCDE, Paris 1989.

[6] Jaccoud, J.-P., Charif, H., Farra,B. : *Cracking behaviour of HSA structures and practical consequences for design*, Third International Symposium on Utilization of High Strength Concrete, Norvegia, June 1993

Defects and Repairs at an Interchange Bridge in Tripoli, Libya

Dommages et réparations à un pont routier à Tripoli, Libye

Schäden und Reparaturen an einer Strassenbrücke in Tripoli, Lybien

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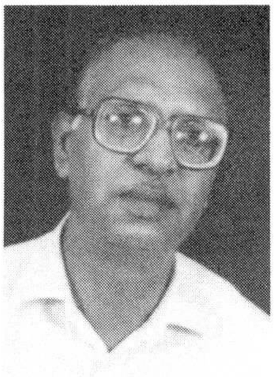

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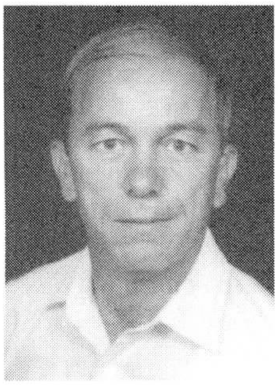

Prof. Tawil, born in 1945, received his doctorate at the Univ. of Colorado, USA. Prof. Tawil is a Univ. of El-Fateh staff member since 19 years. He has also been a part-time technical advisor to the National Consulting Bureau, Tripoli, for the last 17 years.

SUMMARY

Areas of large voids and honeycombing were detected during construction in the soffit of a continuous reinforced concrete voided deck structure. These defects were mainly due to the congestion of rebars. More areas of voids and honeycombing were investigated using hammer soundings and impulse radar non-destructive methods. It has been observed that the versatility of impulse radar in detecting all the areas of voids is offset by anomalies, and the difficulties in the interpretation of the graphical data for deep and heavily reinforced sections. Since repairs of defects were not sufficient to satisfy the client about the structural integrity, an acceptance load test was carried out.

RÉSUMÉ

Des zones non compactes et des vides ont été détectés lors de la construction d'une dalle évidée en béton armé. Ces faiblesses étaient principalement dues à la congestion des armatures. De telles zones ont été détectées par des méthodes non-destructives, grâce à un sondage au marteau et au radar à impulsion. L'utilisation d'un radar à impulsion présente certaines difficultés d'interprétation. La réparation des défauts n'ayant pas convaincu le client de la valeur de la structure, il a été procédé à un essai de charge.

ZUSAMMENFASSUNG

Schwächungen in Form von Löchern wurden in der Unterfläche der durchlaufenden Stahlbetondecke gefunden, meistens durch die zu dicht eingelegte Bewehrung verursacht. Weitere ähnliche Zonen wurden mit Hammerschlägen und mit Impulsradar nicht zerbrechenden Methoden untersucht. Es wurde beobachtet, dass die Brauchbarkeit des Impulsradars, um alle geschwächten Gebiete zu finden, wegen Unklarheiten gestört war. Weil die Reparatur der Fehler die Kunden nicht zufriedenstellte, wurde eine Probebelastung für die Akzeptanz der Konstruktion als Ganzes durchgeführt.

1. INTRODUCTION

As a part of Tripoli Corniche Road, three highway interchange flyover concrete bridges Waddan, Karamanli and Hospital have been constructed in eighties in a length of about 5 km. Hospital interchange is the farthest from the city centre at the eastern end of the Tripoli Corniche ring road, and consists of an elevated structure having 18 spans which include adjoining side spans. The interchange was designed by Rendel Palmer and Tritton of Britain in 1986 and, constructed by Daewoo Corporation of S. Korea under the supervision of National Consulting Bureau, Tripoli from 1986 to 1988. The deck of this interchange is a continuous reinforced concrete voided structure. When the first soffit shutters of the bridge deck were removed areas of voids and honeycombing were discovered. Investigations by hammer soundings, Schmidt hammer and by drilling suspect areas were carried out to detect other areas of voiding. Hammer soundings successfully revealed the other additional areas of voids not detected by drilling, like the voids in the deck above bearing No.28. (Fig. 1.). Repairs of voids traced by drilling were carried out as explained later in this paper. Repair of areas detected by hammer sounding during the final inspection survey had not been done pending further investigations to establish detection of all voids and to assess the effect of large areas of voids on the structural integrity of the deck. It is an accepted fact that timely detection, cause diagnosis and treatment of construction weaknesses will ensure full life span of the structure.

2. CAUSES OF VOIDS

From the survey report of the voids in the deck soffit, it became obvious that the voids on each span were located generally at the quarter span and in some cases the midspan. When compared with 'as built' drawings it was found that the voids at quarter span coincide with the laps in the reinforcement, specially for span 5 which is more complex structurally being on the curved section of the deck and at the intersection with a slip road side spans. Some spans also have reinforcement laps at midspan. On other spans also voids have occurred at midspan where there are no laps in the reinforcement. So it was concluded that the major cause of the voids in the deck soffit was due to congestion of reinforcement and inadequate compaction during construction. In the areas of reinforcement congestion, the concrete did not flow around the reinforcement or under the void formers.

3. THEORETICAL APPRAISAL

To assess the structural integrity of the deck due to the presence of voids at the quarter span and midspan, the designer has investigated for span 5 two conditions based on different assumption on the bond between the lapped reinforcement. Firstly, it was assumed that the whole area where the reinforcement bars are lapping are voided preventing transmission of bending stresses in the bottom longitudinal reinforcement. Results of the analysis indicated that under dead load only the deck section has sufficient capacity to withstand shear provided that the shear reinforcement have adequate anchorage to allow it to act as it is designed. Secondly, it was assumed that the bond between only 50 percent of the longitudinal bars are affected by voids because the laps in the reinforcement are staggered alternatively. According to 'as built' drawings, the amount of reinforcement in the bridge deck slab has been increased to control the crack width under service condition by decreasing the bar spacing from 125mm to 100mm. Calculations check as a continuous beam of unit width with this assumption of 50 percent effective reinforcement gave the maximum dead load moment capacity at ULS equal to 694 KNm/m against the maximum moment capacity of the section of 1550 KNm/m. Assessment of the full section capacity without voids yielded a maximum moment capacity of 2950 KNm/m employing F.E.M. analysis. The theoretical analysis revealed that for the full section the maximum DL moment is 21 percent of the maximum moment capacity of the section. When only 50 percent of the longitudinal reinforcement is considered effective then the DL moment increases to only 45 percent of the moment capacity.

Crack width calculations under the dead load with reduced reinforcement yielded crack width at the extreme fibre of 0.22mm compared to 0.09 mm for the full design section. So it can be concluded that the crack widths under dead load will be increased and under live load would exceed the design crack width for the section. However, site measurements taken of crack width at the mid span of deck soffit 5 and 6 using a microscope crack width recorder led to the detection of 0.1 mm wide cracks. These were in accordance with the calculations for crack width of the fully undamaged section under dead load with all the reinforcement effective.

Representatives of NCB, the designer and the contractor met to discuss the fears of the Client whether all the voids in the bridge deck have been found and repaired properly. To create confidence in the mind of the Client about the safety and the durability of the structure, another survey of the deck soffit using a reliable non destructive method was considered. It was agreed, on the suggestion of the designer, that the impulse radar technique will be the most suitable method for detecting these type of voids. Further, to satisfy the Client the decision for carrying out an acceptance load test on the bridge before opening to traffic will be taken after reviewing the results of the impulse radar method.

4. IMPULSE RADAR TECHNIQUE

Survey of the concrete surface by impulse radar is achieved by tracking a transducer radio anteunae over the surface to be investigated at a slow speed; pulses of energy are transmitted into the material which are reflected from any internal surface or structure change depending upon the difference in their permittivity or conductivity. The returning signals from each vertical scan build, as the transducer is moved, into a continuous transverse profile of the interior of the material as shown in Fig.2. A plane of energy is actually transmitted rather than a ray beam: this plane is normally set parallel to the direction of the survey such that all information streams refer to material directly below the survey line. More details about this method are given in references [1] and [2].

Signal attenuation is per unit wave length, and sets depth of penetration limit on this for any material, which will obviously be shallower for higher frequencies. Equally, target resolution decreases with increasing wavelength thus the selected frequency of the transducer is a balance between penetration and resolution.

The areas of the bridge deck selected for impulse radar survey were generally the areas of congested reinforcing steel, particularly around laps in the lower reinforcement. All areas investigated are shown in Fig. 1. All survey lines, both longitudinal and transverse, were set out on the soffit of the deck in areas marked out by the contractor. The majority of the data was collected from profile lines transverse to the deck and set at 250 mm centres. These line provided a general scan of the concrete condition to a depth of at least 250 mm. The transverse profiles were transacted by longitudinal profiles at 1 m centres arranged to examine the concrete condition immediately below the centre line of each void former. Survey lines were profiled using either the high resolution transducer with centre frequency set at 1000 MHZ, or the slightly lower resolution but deeper penetrating transducer set at 900 MHZ. In some cases both frequencies were used on the same line. This multi-frequencies approach was adopted in order to obtain as much information on the interior of the decks as possible.

5. TEST DATA

Figure 3 shows data collected with the 900 MHZ transducer along a longitudinal profile of deck 5. It can be seen that the hyperparabolic return from the reinforcement are interrupted: this is caused by the presence of a void reducing the energy coupled to steel thus sharply alternating the returning signal and creating a blank area in the data. The deeper void attenuating located between the reinforcement and the internal void former is not as readily identifiable within the data, but a disturbance or irregularity in the signals can be noted both as a cancellation or attenuation of signals from the reinforcement and also as a phase shift in the response from the metal skin of the internal void former.

The procedure of investigation by impulse radar at site included collection of data, interpretation and investigation of anomalies by drilling into specified target areas. The information obtained from the drilling investigations and further hammering testing has been used to eliminate or confirm anomalous signals interpreted from the data. According to the experience of the present authors, the versatility of the impulse radar technique has been offset by the difficulties in the interpretation of the collected graphic data. The final data analysis and subsequent location of defects have been the result of correlation between data interpretation and the drilling investigations. It has been found that hammer echo survey, although time consuming yielded either similar or better detection of voids and honeycombing areas compared to impulse radar technique due to complexity of graphic interpretation. After the completion of impulse radar survey, 5 new voids were detected on span 5 and one on span 16 by the representative engineer using hammer echo survey. Although it was concluded from the results of the impulse radar survey that all the significant areas of voiding and honeycombing of the bridge deck have been detected and repaired but it is still possible, as shown by latest hammer echo survey, that some of the voids and honeycombed areas have remained undetected. In order to satisfy the Client about the structural integrity of the bridge before the final handing over an acceptance load test has been carried out by the contractor.

6. REPAIRS

Following the detection of voids in the soffit of bridge deck an extensive program of repairs has been undertaken. Two types of voids were identified for which different strategies were employed. The first type of defect included extensive areas of voids between the reinforcement and the underside of the void former. The procedure of repairs consisted of breaking the defective concrete and saw cutting the edges of the repair area. Prior to concreting the repair the exposed reinforcement was cleaned by sandblasting. For the large volume repairs an access from the top surface of the deck was provided by coring to the top surface of the void former, cutting through the top and bottom sections of the void former and installing a plastic pipe. A soffit shutter was fixed and the void concreted through the plastic pipe using 10 mm maximum size aggregates. On completion of the concreting the plastic pipe was removed, the top of the void former repaired and the concrete above the void former reinstated.

The second type of defect involved thin section repair where voids were detected below or extending only to the bottom layer of reinforcement. Again, the damaged area was fully broken out and the edges of the repair area saw cut. The repair of the defect was carried out using Febset 'non flow' epoxy mortar.

Durability of the repairs was questioned by the Client due to the location of the interchange close to the sea. Keeping in mind the possibility of occurrence of shrinkage cracks around the perimeter of the repairs and to prevent the ingress of chlorides through the cracks, all the concrete surfaces have been coated with Dekguard elastic, produced by Forsoc of U.K.

7. ACCEPTANCE TEST

The load tests were carried out on spans 5,16 and 8 (without defects) to satisfy the Client that the defects that occurred during construction have been satisfactorily repaired and the deck is capable of carrying its design load. The load tests results satisfied the code acceptance criteria of maximum deflection for any span and for recovery which always exceeded 80%.

8. CONCLUSION

Appearance of defects like areas of large voids and honeycombing in the bridge deck during the construction stage is a matter of concern. Such defects should be thoroughly investigated and their effect on the structural safety evaluated. An acceptance load test following the repairs is recommended to restore the confidence of the Client.

REFERENCES

1. BUNGEY J.H., MILLARD S.G., Radar Inspection of Structures, Proc. I.C.E. Structures and Buildings Jnl., May 1993.

2. PITT J.B., Use of Impulse Radar as a method for the Non Destructive Testing of Concrete Slabs, Building Research and Information, Vol. 20, No.3, 1992.

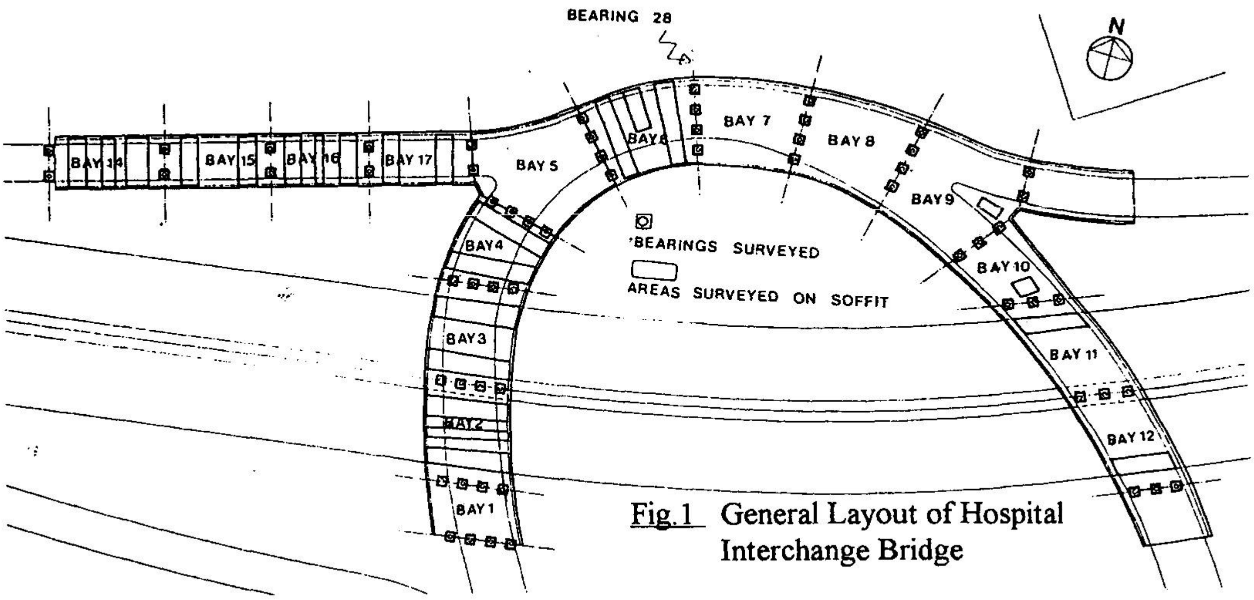

Fig.1 General Layout of Hospital Interchange Bridge

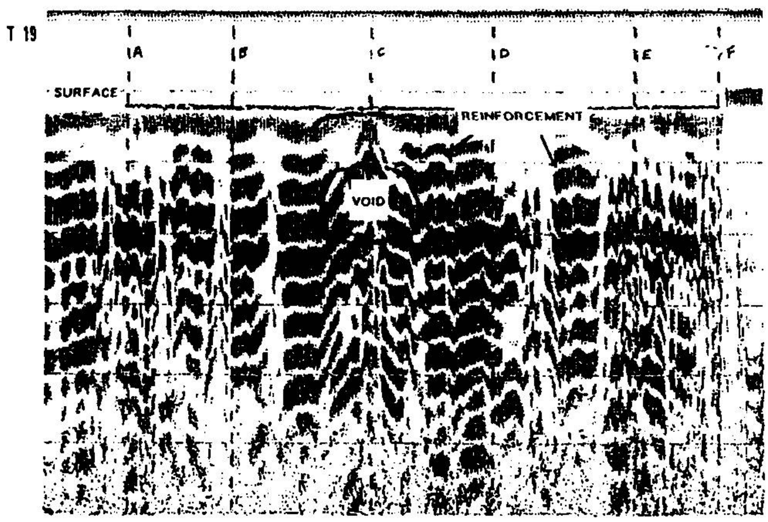

Fig. 2
Radar Output for Transverse Scanning Profile (Span 5)

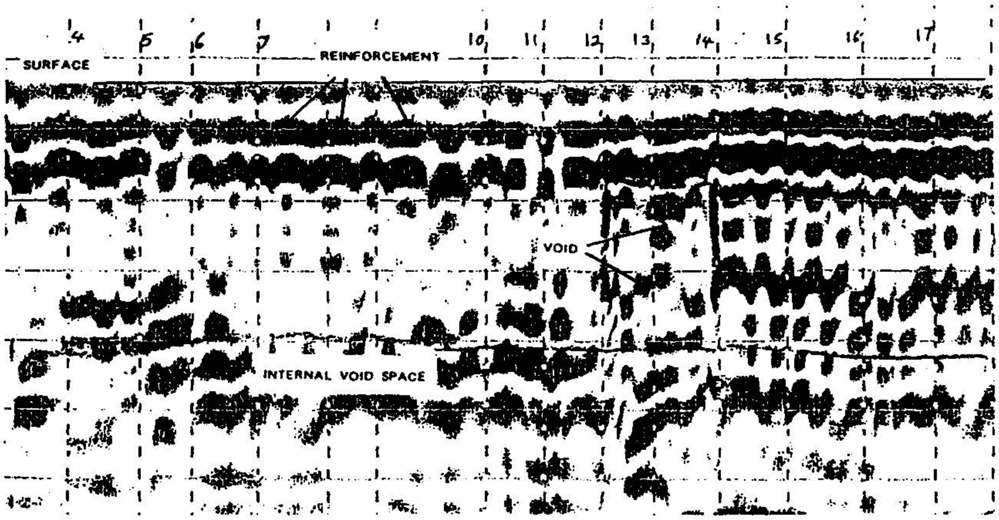

Fig. 3 Radar Output for Longitudinal Scanning Profile (Span 5).

New Approach for Strengthening Structural Concrete Bridge Girders

Conception nouvelle pour le renforcement de poutres de pont en béton armé

Neues Konzept für die Verstärkung von Stahlbeton-Brückenträgern

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SUMMARY

This paper describes a design model, including a new detailing approach for the stirrups, which has been used in the strengthening of reinforced concrete beams, such as bridge girders. In the proposed approach a new concrete top overlay containing confinement stirrups is cast on top of existing girders. The confinement stirrups are welded to the stirrups in the girders near the support region in order to prevent an inter-laminar shear failure. The results of tests have shown that each of the girders strengthened using this design approach has acted as a single unit, failed in a ductile manner and achieved its full flexural capacity compared with similar girders which have been cast monolithically.

RÉSUMÉ

L'article présente le principe d'une nouvelle conception pour la consolidation et le renforcement des poutres en béton armé. Le principe de cette méthode consiste à couler sur la poutre existante une nouvelle dalle de béton armé. Afin de réduire les risques de cisaillement, les armatures de cette nouvelle dalle sont fixées par des étriers soudés sur les armatures des poutres existantes. Des essais ont démontré que les poutres ainsi renforcées se conduisaient comme un ensemble homogène répondant aux mêmes critères de solidité et flexibilité que des poutres ayant été réalisés de façon monolithique.

ZUSAMMENFASSUNG

Diese Studie erklärt eine konstruktive und errechnete Methode für Bügel, die für die Unterzüge in Stahlbetonbauten benutzt wird. In dieser Methode wurde eine obere Schicht von Beton geschützt, mit zusätzlichen Bügeln auf die alten Bügel geschweisst, in der Nähe von den prägende Stellen, wegen den Querkräften. Durch die Ergebnisse wurde festgestellt, dass jeder zusätzlich verstärkte Unterzug als Einheit wirkte, duktil brach und dieselbe plastische Verformung erreichte, wie monolithisch hergestellte Träger.

1- INTRODUCTION

A large number of existing reinforced concrete structures, e.g. bridge girders, have been designed to carry loads which are well below present day requirements with respect to load carrying capacity and ductility. Also, there are many structures which have been weakened as a result of earthquakes, incorrect design and detailing, or poor construction practice. Such structures need to have their strength and ductility enhanced to enable them to carry more demanding loading levels. There is an obvious requirement for the development of methods to strengthen existing structures. In this context, several techniques for repairing and strengthening of existing structures have already been investigated, however, there is little information available on their effectiveness[1]. These approaches have included the use of ferrocement, epoxy injection, plate-steel bonding, a combination of the above, concrete overlays and underlays, and post tensioning. These techniques have proved that they may result in the restoration and maintenance of the original strength of a structure but are unlikely to result in a significant increase in the original structural capacity. Several of these approaches have also proved somewhat difficult to implement in practice. Moreover, the majority of research work in this area has been limited to laboratory tests to examine the comparative behavior of structures repaired using different techniques. The main problem encountered in using these techniques was the occurrence of a brittle shear failure in the strengthened beams. It is believed that there is a need for a rigorous design approach to be developed which could be applied generally to structural concrete members.

In this paper the use of a flexure-shear interaction design model with a new approach for detailing of stirrups, which has already been verified for the design of beams with different cross sections [2,3], is described for strengthening of structural concrete girders.

2- FLEXURE-SHEAR INTERACTION DESIGN MODEL FOR STRENGTHENING OF BEAMS

2.1 Outline of the model

Conventional design methods for structural concrete beams treat shear and flexural actions separately although they occur simultaneously in practice. The design approach for shear relies on the transverse reinforcement at the ultimate limit state to resist the shear stress v_s in excess of that assumed to be carried by the concrete section v_c. Such an approach assumes that two types of mechanisms act simultaneously in the beam structure, namely, truss and beam/arch actions corresponding to the shear stresses. The assumption of the presence of two mutually exclusive mechanisms thus leads to the confusion in the current methods for shear design. On the other hand, it is known that[4] the ability of a structure to carry additional load when subjected to large deformations due to additional load depends primary on confinement of the compression concrete. In flexural as well as in diagonal failures collapse of a beam occurs as a result of spalling of the compression concrete in either the inclined leg or the horizontal leg regions of the Compression Force Path (CFP) as a result of the development of transverse tensile stresses in the region of the path[5]. In order to prevent the collapse of beams, the compression concrete in the path regions must be provided with confining stirrups as shown in Fig. 1. The inclined leg regions are provided with conventional full length stirrups and the horizontal leg regions are provided with short stirrups. This detailing approach adopts the same mechanism for resisting moment and shear, irrespective of the level of loading. When the concrete capacity is exceeded, the transverse reinforcement is intended to enhance the strength of concrete in the compression zone, which is the main element in the beam structure which resists the imposed loading (axial, shear and bending moment). To elaborate further on the proposed unified behavioral mechanism, consider the case where the shear span to depth ratio a/d is within Kani's Valley. The capacity of each leg region M1, due to the development of transverse tensile stresses in the compression region, is less than the full flexural capacity of the beams Mf.

These tensile stresses reduce the concrete compressive strength by an amount equivalent to Δf_c which will lead to a corresponding reduction in the flexural capacity of each leg of the beam by ΔM where $\Delta M = Mf - Ml$. The full flexural capacity of the beam Mf can be restored by offsetting the reduction in the concrete compressive strength Δf_c. In this case this is achieved by utilizing the effect of confinement to enhance the concrete compressive strength by an equal amount i.e. Δf_c. After determining Δf_c, the confinement requirements and thus the amount of stirrups required to achieve this added strength is determined using Eqs. 1 to 3. The complete development and the experimental validation of the flexure-shear interaction design model for beams, including the determination of the leg capacity M1, have been reported elsewhere[3].

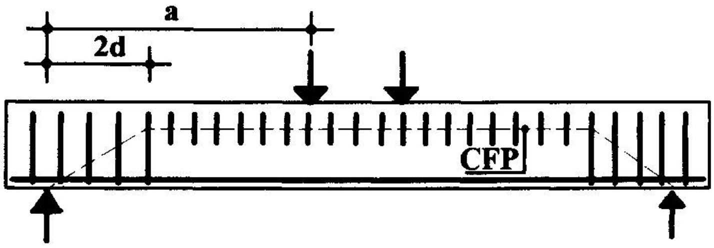

Fig. 1 Detailing arrangements for beams.

By definition

$$\Delta f_c = (K_s - 1) f_c \quad (1)$$

where K_s is the confinement enhancement factor[4]. Based on flexural theory

$$\Delta f_c / f_c = (Mf - M1)/Mf = \Delta M/Mf \quad (2)$$

The combination of Eqs. 1 and 2 can be expressed as follows:

$$K_s = \Delta M/Mf + 1 \quad (3)$$

The amount of confinement stirrups corresponding to K_s is found using a modified form[3] of the confinement model originally developed by Sheikh and Yeh[4].

2.2 The application of the model for strengthening of girders

The flexure-shear interaction design model can readily be adopted for the strengthening of girders. The detailing approach for the stirrups has a significant advantage over traditional approaches to detailing when applied to the maintenance and strengthening of existing structural concrete members. In the case of girders, a new concrete layer can be cast on top of the girders to act as a compression zone within the girder structure. The enhancement of the major parts of the girders, i.e. the horizontal leg region of the CFP, can easily be achieved since the top cast layers are only required to be provided with short stirrups which do not need to be extended into the existing part of the girders. The inclined leg regions do, however, require more careful consideration. The stirrups in the new concrete layer should be welded to the existing stirrups in the inclined leg regions thus avoiding the requirements to treat the overall depth of the section. Welding of the stirrups in the inclined leg regions would also prevent an inter-laminar shear failure.

3- TEST PROGRAM

3.1 Test beams

Twenty under-reinforced simply supported beams with different shear span to depth ratios a/d were included in a test program conducted at Birzeit University[6] for the verification of the proposed design approach for the strengthening of existing girders. All the test beams had a cross section of 150mm X 240mm, overall length of 2000mm, longitudinal reinforcement ratio ρ of 0.0075, and design concrete compressive strength f'_c of 20MPa. The test beams were divided into two types based on the design approach adopted as shown in Table 1 and Fig. 2. Traditional beams of Type T (beams T1, T2, T3, T4, and T5) were designed based on the ACI code of practice[7]. Strengthened beams of Type S (beams S1, S2, S3, S4, and S5) were cast in two stages. Beams of half of the overall depth were designed either based on the ACI code of practice to simulate existing beams in need of strengthening, or based on the proposed approach, whichever is critical. In this case, the upper horizontal leg of the stirrups were left exposed to facilitate welding. The remaining part of the beams i.e. the top cast overlay, was designed based on the proposed approach taking into consideration the whole section i.e. as if the beam had been cast monolithically. The stirrups in the top cast overlay and in the original beams were welded together over a distance equal to 2d from the support. This was to ensure the full interaction behavior between the upper and lower parts of the beams. The weld provided was sufficient to prevent an inter-laminar shear failure. It should be noted that two nominally identical beams were cast for each beam Type. High-strength deformed steel bars of 10mm diameter with f_y=420MPa were used for longitudinal reinforcement. The stirrups were fabricated from mild steel bars of 6mm diameter with f_y=280MPa.

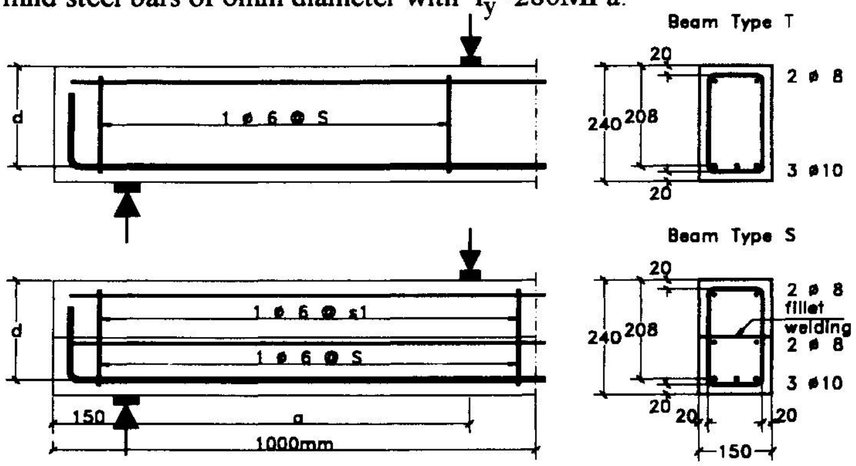

Fig. 2 Details of the test beams.

3.2 Test results

The measured and the calculated load carrying capacities of the beams are shown in Table 1. Fig. 3 shows typical load mid-span deflection curves for beams of Types T1, S1, T3, S3, T5, and S5. Fig. 4 shows a typical crack pattern and a typical strain diagram obtained from demec readings taken on the concrete surface for the strengthened beam Type S3 .

Beam type	a/d ratio	s mm	s1 mm	P1 kN	P2 kN	P1/P2
T1	3	50	-	37.5	30.2	1.24
S1	3	50	50	42.6	34.1	1.25
T2	3.25	75	-	34.1	27.9	1.22
S2	3.25	50	75	39.3	31.5	1.25
T3	3.5	75	-	29.7	25.9	1.15
S3	3.5	50	75	35.7	29.3	1.22
T4	3.75	75	-	28.9	24.2	1.19
S4	3.75	50	75	33.4	27.3	1.22
T5	4.0	100	-	28.2	22.4	1.26
S5	4.0	75	100	31.5	25.3	1.25

Table 1 Details and results of the test beams.

3.3 Mode of failure

All beams, regardless of the design approach used, failed in a flexural mode by spalling of the compression concrete in the region subjected to maximum bending moment as shown typically in Fig. 4.a for beam Type S3. Diagonal cracks developed in the shear span as an extension of the flexural cracks. All cracks proliferated and widened under increasing loads. In beams of Type S the short stirrups succeeded in preventing diagonal cracks from extending into the compression zone in the top overlay thus preventing diagonal failure in all strengthened beams. However, in beams of Type T the diagonal cracks bypassed the loading points and extended into the mid-span region, where stirrups had not been provided, thus entering into the compression concrete which spalled at the flexural failure. Very small horizontal cracks between the overlays and the original beams at a distance equal 2d from the support were found in the final crack pattern for some of the beams of Type S which had a/d ratios up to 3. Irrespective of these cracks, it is concluded that full interaction did develop between the overlays and the original beams. The concrete strain measurements, which were recorded on the sides of the beams, confirmed this conclusion. Typical strain measurements for the strengthened beam Type S3 shown in Fig. 4.b indicated that plane sections remained plane after bending. Furthermore, the crack widths and mid-span deflection measured at working load levels, which have been assumed at 65% of the ultimate load, satisfied the requirements of the serviceability limit state in the ACI code of practice.

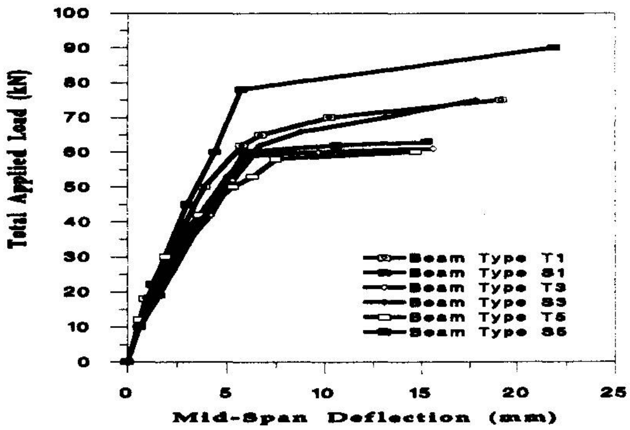

Fig. 3 Load mid-span deflection curves.

3.4 Load carrying capacity and ductility

All beams achieved their full flexural capacity and failed in a ductile manner which is a characteristic of under-reinforced beams as shown in Fig. 3. The load carrying capacity of the strengthened beams of Type S, regardless of the a/d ratios, were larger than the corresponding measured capacities of the traditionally detailed and cast beams of Types T as shown in Table 1. The increased strengths found in the strengthened beams were due to the effect of the reinforcing bars in the compression concrete in the original beams which behaved as tension reinforcement after the overlays had been added. The ratios of measured (P1) and calculated (P2) load carrying capacities (P1/P2) ranged from 1.22 to 1.25 and from 1.15 to 1.26 for beams of Types S and T respectively. The relatively large variation in the P1/P2 ratios (11%) for beams of Type T was due to the confinement being ignored in the design of the flexural capacity of the sections using the ACI code of practice. On the other hand, the variation in the P1/P2 ratios for beams of Type S was only 3% since they were designed using the flexure-shear interaction model. It is interesting to note that an increase in the load carrying capacity of up to 173% was achieved as a result of the strengthening approach adopted for these beams compared with the capacity of the original beams without addition of the confined overlays.

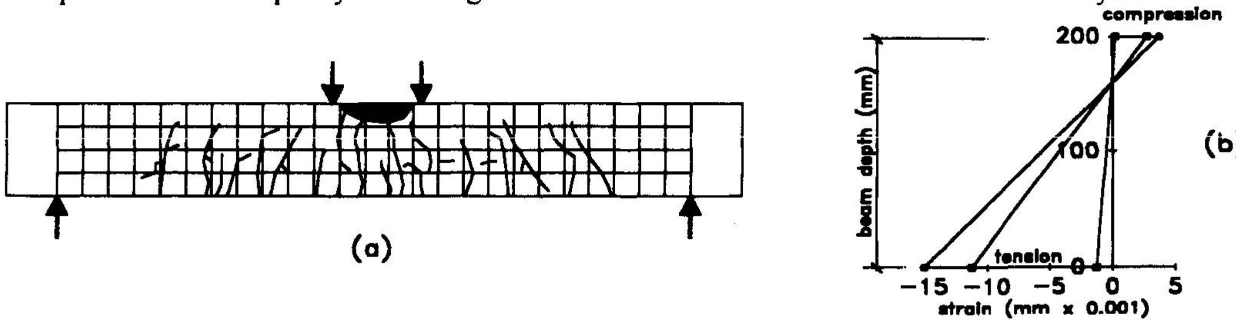

Fig. 4 Behavior of beam Type S3. a- Crack pattern. b- Strain diagram.

4- CONCLUSION

A flexure-shear interaction model has been used for the strengthening of existing beams. The beams were strengthened by casting confined concrete overlays on their upper surfaces. The inter laminar shear failure was prevented by welding the horizontal legs of the stirrups in the original beams to the adjacent legs of the stirrups in the overlays in the support regions. All of the strengthened beams failed in a ductile manner and achieved their full flexural capacities compared to similar beams which have been cast monolithically. An increase up to 173% of the capacity of the original beams was achieved using the proposed design approach. The variation between the calculated and measured capacities of the strengthened beams was 3%. The flexure-shear interaction design model has been confirmed to be suitable for use in the strengthening of existing reinforced concrete girders.

5- ACKNOWLEDGMENT

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5- REFERENCES

1- Trikha N. D., Jain. C. S., and Hall K. S., "Repair and strengthening of damaged concrete beams," Concrete International, June 1991, pp. 53-59.

2- Ziara M. M., Haldane D., and Kuttab A., "Shear and flexural strengths resulting from confinement of the compression regions in circular section structural concrete beams," Magazine of Concrete Research (London), V. 45, No. 164, September 1993, pp. 211-219.

3- Ziara M. M., "The influence of confining the compression zone in the design of structural concrete beams," Ph.D. thesis. Department of Civil and Offshore Engineering, Heriot Watt-University. Edinburgh, UK, July 1993.

4- Sheikh S., and Yeh C., "Analytical Moment-Curvature Relations for Tied Concrete Columns," Journal of Structural Engineering, ASCE, V. 118, No. 2, Feb. 1992, pp. 529-544.

5- Kotsovos M. D., "Compressive Force Path Concept Basis for Ultimate Limit State Reinforced Concrete Design, "ACI Structural Journal, V. 62, No. 1, Jan-Feb 1988, pp. 68-75.

6- Tannous E., and Naf'a M., "Strengthening of Existing Structures," Final Year Project, Department of Civil Engineering, Birzeit University, Via Israel, 1994.

7- ACI Committee 318, "Building Code Requirements for Reinforced Concrete and Commentary (ACI 318-89/ACI 318R-89)," American Concrete Institute, Detroit, 1989, 353pp.