Session D4

Strengthening of Structures by Prestressing
Renforcement des structures à l'aide de la précontrainte
Verstärkung von Bauwerken mittels Vorspannung

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Repair and Strengthening of Concrete Structures by Post-Tensioning

Réparation et renforcement de structures en béton au moyen de la précontrainte

Instandsetzung und Verstärkung bestehender Betonbauten mittels Vorspannung

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SUMMARY

Repair and strengthening often is a cost-effective alternative to demolition. One method which has significant advantages is the application of additional, or replacement of post-tensioning tendons. This paper presents a number of recent examples of bridges strengthened by application of external post-tensioning.

RÉSUMÉ

La réparation ou le renforcement de structures est souvent une alternative rentable par rapport à la démolition. L'application de précontrainte additionnelle ou le remplacement de précontrainte existante est une méthode qui présente des avantages certains. Cet article présente plusieurs exemples récents de renforcement de ponts par l'application de précontrainte extérieure.

ZUSAMMENFASSUNG

Instandsetzung oder Verstärkung sind oft kostengünstige Alternativen zum Abbruch und Neubau. Die Verstärkung oder das Ersetzen der bestehenden Vorspannung ist eine Methode, welche erhebliche Vorteile mit sich bringt. Dieser Artikel beschreibt einige neuere Beispiele von Brücken, welche mittels externer Vorspannung verstärkt wurden.

1. INTRODUCTION

A significant percentage of highway bridges anywhere in the world was constructed between 1940 and 1970. Traffic volumes, speeds and the average weight of heavy vehicles for which these bridges were designed have increased significantly since then. This, together with some design deficiencies, e.g. underestimating the effect of temperature gradients and of creep and shrinkage, and with the fact that many bridges were not maintained properly, has resulted in deficiencies of various degrees of a great number of bridges. Repair and/or strengthening often is a cost-effective alternative to demolition. One repair/strengthening method which has significant advantages is the application of additional, or the replacement of existing post-tensioning tendons.

2. PRINCIPLES, ADVANTAGES AND LIMITATIONS OF EXTERNAL POST-TENSIONING

The application of additional, or the replacement of existing post-tensioning tendons requires very little interference with the existing structure. The tendons, deviation saddles and anchorage blocks are placed inside the bridge girder so that the overall appearance is not changed, very little extra weight is added (especially when foundations are already fully utilised) and the work can often be carried out without significant restrictions in the normal use of the structure. Depending on the type of deficiencies which can be classified in two major categories (insufficient strength and excessive deflections and cracks) the post-tensioning can either follow a straight line or be draped. Obviously straight tendons do not markedly increase the shear capacity. The applied axial force and/or upward acting load-balancing forces from the post-tensioning tendons improve the performance of the structure in both serviceability and ultimate limit state. In the case of excessive deflection and cracks, one should however not expect that the opposing forces from the draped tendons would reverse the mid-span deflection by more than a fraction nor will wide cracks or open joints completely close after applying the additional prestress. This is because a cracked section is substantially stiffer for unloading, e.g. due to opposing moments, than for continued loading, and because the long-term creep induced part of the deflection cannot be reversed by a significant degree.

3. RECENT EXAMPLES

3.1 Oléron Viaduct, France

3.1.1 Background :

The Oléron Viaduct linking the Island of Oléron to the main land of France with a total length of 2862 m was built between 1964 and 1966. The bridge superstructure consists of a prestressed concrete single box girder constructed by the balanced cantilever method using precast segments 3.30 m long and 10.60 m wide. It rests on the abutments and 45 piers equipped with reinforced elastomeric bearings. The spans are 28.75 m / 7 x 39.50 m / 59.25 m / 26 x 79.00 m / 59.25 m / 9 x 39.50 m / 28.75 m. The viaduct is segmented by eight expansion joints into 9 sections of roughly 320m each. The girder has a constant height of 2.50 m in the spans up to 39.50 m and a variable height of 2.50 m to 4.50 m in the other spans.

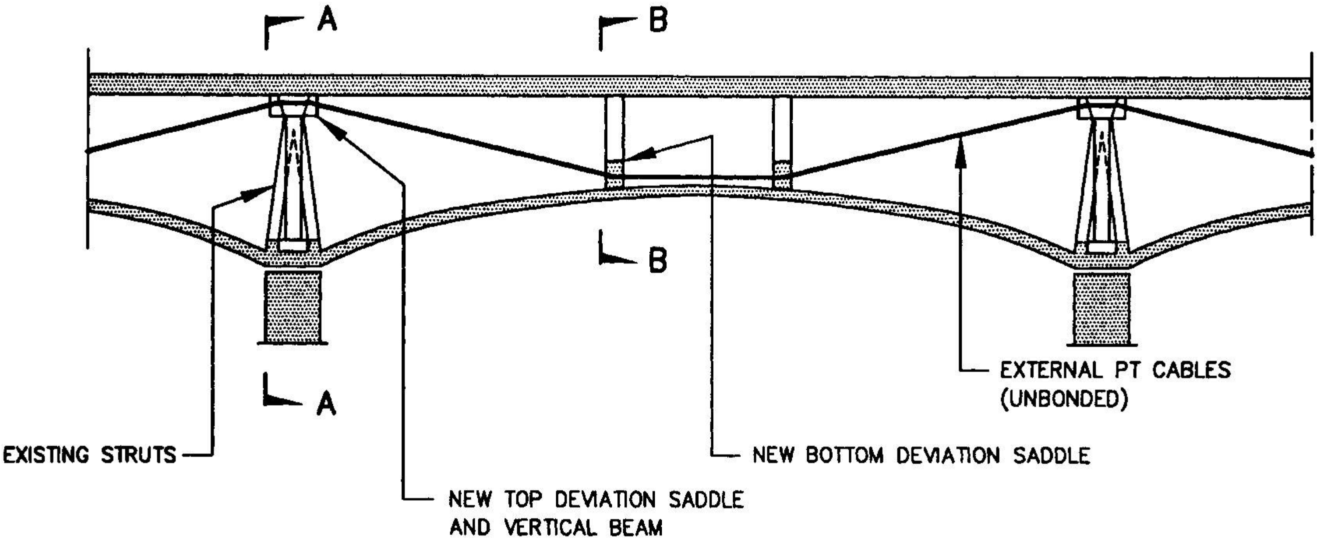


FIG 1 : OLERON BRIDGE : CABLE LAYOUT OF ADDED EXTERNAL POST TENSIONING

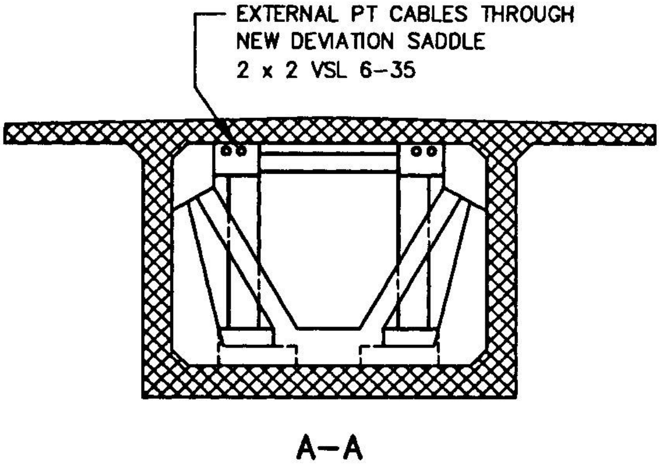


FIG 2 : OLERON BRIDGE :
DEVIATION SADDLE AT PIER

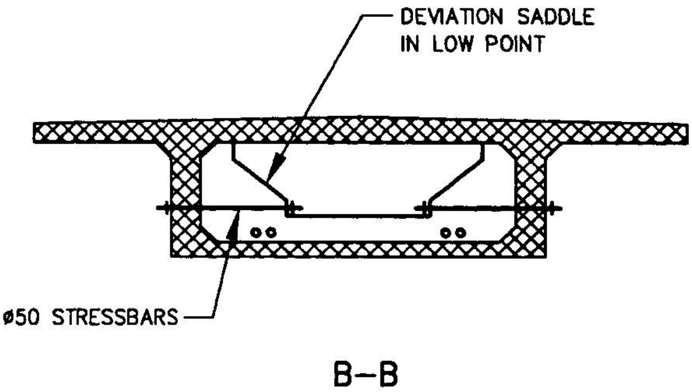


FIG 3 : OLERON BRIDGE :
DEVIATION SADDLE AT MID SPAN

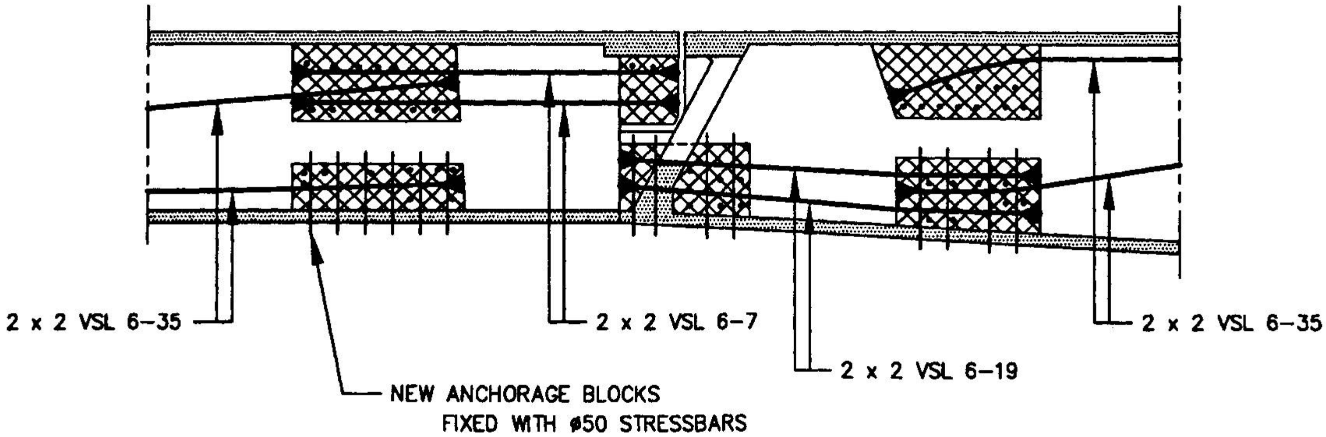


FIG 4 : OLERON BRIDGE : CABLE LAYOUT AT EXPANSION JOINT

3.1.2 Rehabilitation Concept

It was decided to use draped external post-tensioning tendons to strengthen the superstructure. Based on the official solution of the DDE of Charente Maritime, VSL, assisted by BOUYGUES TP consultant office, suggested some modifications in order to optimise the strengthening works.
The main modifications concern :

- Reduction of the number of anchoring blocks by using continuous tendons over 320 m instead of tendons over one span.
- Modification of the geometry and the nailing of the anchorage blocks. This is achieved by replacing stressbars spanning from one web to the other by shorter stressbars nailing directly each anchorage block so that the transmission of the friction force to the existing structure is improved and the local stresses are minimised. The prestress force losses in the stressbars due to the stiffness of the box girder are therefore eliminated.
- Modification of the pier deviator geometry in order to obleviate all nailing (autostable structures). This is achieved by replacing the top anchorage blocks nailed to the box girder by an autostable frame. The thickness is slightly increased in order to wrap the new frame around the existing diaphragm at the top of the box girder creating a longitudinal restraint (to cater for the differential in post-tensioning force).
- At last, the modification of the tendon geometry in order to avoid overstressing the hinges in the end spans which support the non-strengthened parts of the structure (spans 1 and 9)

Using continuous tendons over 320 m length calls for a few modifications in the choice of the post-tensioning : a total of 2 x 2 tendons of 35 strands 0.6" each (instead of 2 x 37 + 2 x 27)were placed in order to allow for the increase of friction losses and the steel grade had to be chosen slightly higher than originally planned (1860 MPa resp. 1770 MPa). The anchorage blocks near the expansion joints were adapted in order to allow stressing at both ends. The costs of these two measures were largely compensated by the saving of 40 anchorage blocks as well as a large number of anchor heads and stressing operations.
The feasibility of external tendons of such considerable length had been demonstrated, in principle by the successful placing of 400 m long straight external tendons to strengthen the Höll viaduct in Switzerland.

3.1.3 Execution of Work

The Oléron Viaduct is the only link to the main land for the traffic as well as for the services (telephone, water and electricity). It was therefore vital to maintain the traffic and the services at all time during construction. One traffic lane in each direction (7.0 m wide) was maintained and particular care was taken in the box girder when moving the equipment to avoid any damage to the 600 mm water supply and the 90'000 and 20'000 V electrical supply. Access for personnel and small equipment is provided through the existing man holes, whereas for the large equipment (jacks, pumps, etc.) the access is provided through an additional hole cut in the bottom slab near the abutment.
Once the position of the existing tendons was determined by gammagraphy, the stressbars were positioned and the anchorage blocks cast using a mobile pump placed on the deck. The HDPE sheath was then installed and supported temporarily approximately every 3 m. The coils of tendons were placed outside the box girder first at the abutment, then on the deck. The strands were then pushed individually through the sheath over the whole length (320 m). Openings in the sheath were provided 60 m from the end to enable a second pushing machine to be installed if need be. The tendons were stressed one by one from both ends using a multi-strand jack. Finally the tendons were injected with non-shrink grout.

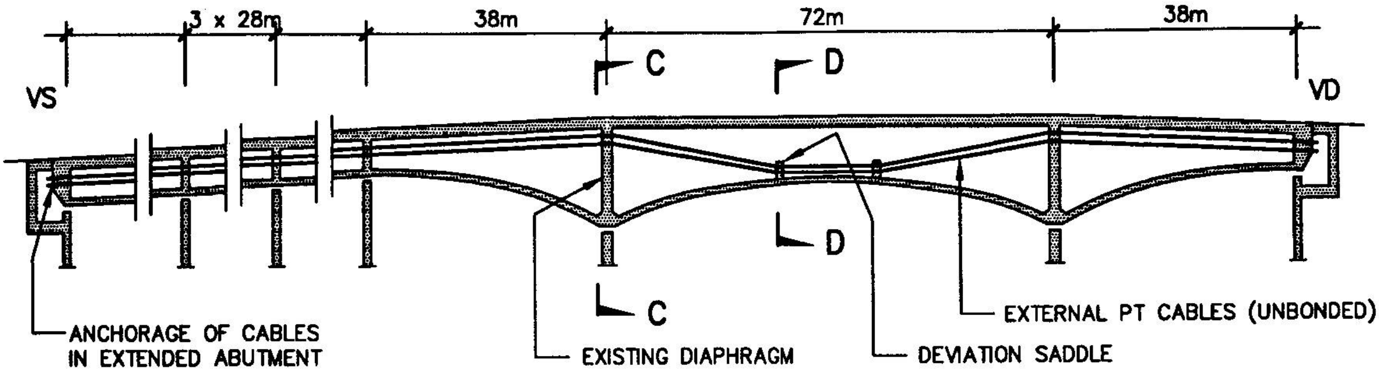
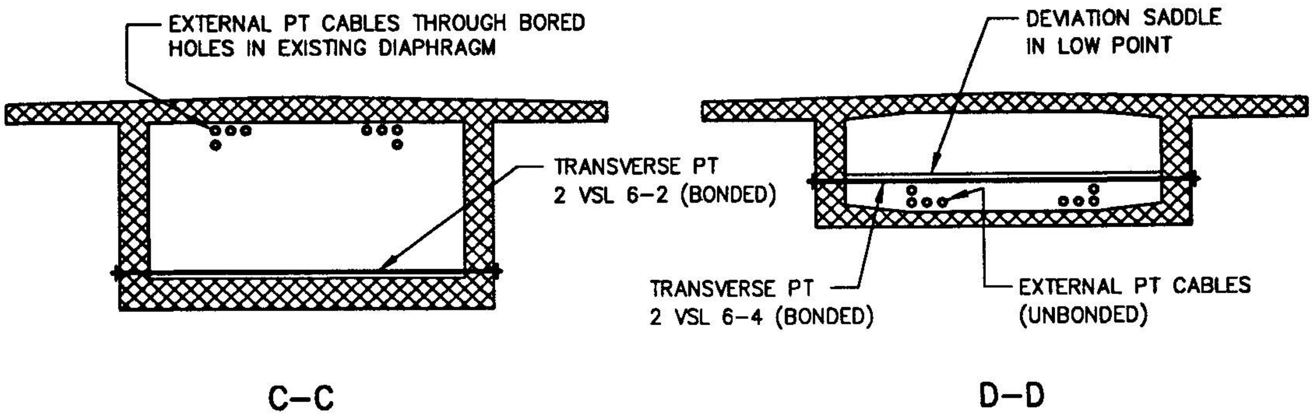


FIG 5 : MASSONGEX BRIDGE : CABLE LAYOUT OF ADDED EXTERNAL POST TENSIONING



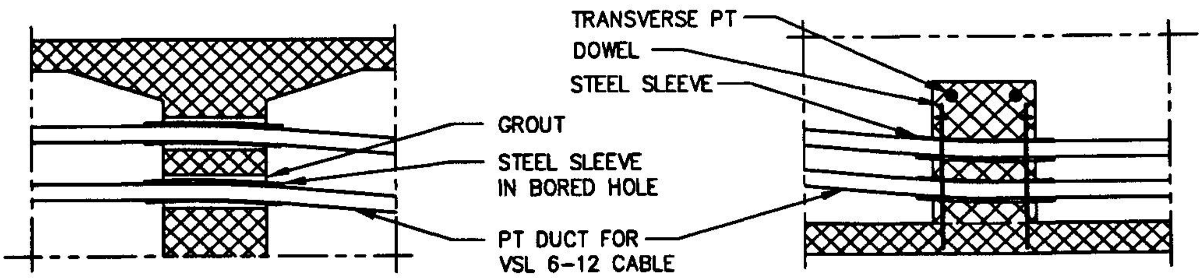


FIG 6 : MASSONGEX BRIDGE :
SECTION AT PIER
WITH DEVIATION SADDLE

FIG 7 : MASSONGEX BRIDGE :
LOW POINT SADDLE

3.2 Motorway Bridge across the Rhône at Massongex, Switzerland

3.2.1 Background:

This 232 m long box girder bridge with spans 3 x 27 - 38 - 72 - 38 m was built between 1978 - 1979. Since 1980 (8 months after final stressing) the superstructure has been surveyed regularly and in 1991 exhibited 113 mm mid-span deflection in the 72 m haunched main span, accompanied by a number of cracks up to 0.5 mm wide. The deflection had increased linearly with time and it was feared that this trend may continue unless strengthening was implemented. A substantial gradient of temperature due to the proximity of the very cold water of the Rhône, combined with strong sun radiation was thought to be the reason for large tensile stresses (2.5 N/mm2). Furthermore it was suspected that the amount of precompression provided by the existing tendons was appreciably lower than assumed in the design.

3.2.2 Rehabilitation concept :

It was decided to strengthen the structure using external post-tensioning. A total of 8 tendons VSL 12 x 0.6" composed of 12 monostrands individually greased and sheathed, then placed in a HDPE duct were installed over the full length. This type of tendons allows their re-stressing, de-tensioning and replacement if required. Hydraulic load cells were installed to continuously monitor the cable force. The tendons are draped with deviator tubes placed in the existing pier diaphragms Which were strengthened with additional transverse bonded tendons, and two low-point deviators consisting of prestressed concrete cross beams in the main span. The anchorages are located in prestressed buttresses added to the abutment diaphragms in order to allow the introduction of the additional forces and to avoid further damage to the already cracked end sections. New access chambers were constructed with sufficient clearances for the stressing jacks and strand overlength.

3.2.3 Execution of work :

The HDPE sheath was installed beforehand and attached to a single prestressed strand strung along the profile, thus avoiding any further intermediate supports. The monostrand bundle was assembled near the site and pulled in from behind one abutment. After taking the slack out of the strands by slightly stressing them grout was injected into the HDPE sheath under vacuum. The full prestress was applied after hardening of the grout. After stressing, an upward deflection of 20 mm was measured at mid-span, i.e. roughly 18% of the deflection was reversed. The surfacing was partially adapted and the remaining cracks were epoxy injected. More importantly, further deflection and crack growth was stopped by the added prestress. The repair work was carried out maintaining the traffic in both directions on one half of the bridge.

ACKNOWLEDGEMENTS

The information provided by the consultants responsible for the design of the rehabilitation of Massongex Bridge, REALINI, BADER ET ASSOCIES (Geneva, Switzerland) is gratefully acknowledged.

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Reinforced Concrete Framed Building Strengthened by Prestressing

Cadres en béton armé renforcés par la précontrainte

Rahmentragwerke aus Stahlbeton verstärkt durch Vorspannung

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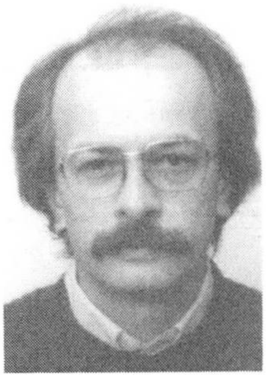

Vanja Alendar, born in 1947, graduated at the Faculty of Civil Engineering in Belgrade. Designer of buildings and long-span structures, his current interests are in prestressed concrete structures, seismic design and slender RC structures.

SUMMARY

Due to inadequate structural design and reinforcement detailing, a three-story reinforced concrete framed structure has suffered excessive cracks and deflection, followed by local failure of columns. The structure's bearing capacity has been improved by shoring each frame beam with prestressing tendons conducted below the beam soffit, in a "two-chord" system. Analysis of causes, details of strengthening and results of deflection measurements are presented in the paper.

RÉSUMÉ

Les cadres en béton armé d'un bâtiment à trois étages ont été gravement fissurés et déformés par suite de calcul et de disposition de l'armature constructive inadéquats. Le renforcement de la structure portante est effectué par des câbles de précontrainte extérieurs, en combinaison avec des plaques métalliques collées. L'analyse des causes, les détails de renforcement et les résultats de mesure des flèches après la mise de précontrainte sont présentés.

ZUSAMMENFASSUNG

Die unangemessene Berechnung und Bewehrung des Rahmentragwerks aus Stahlbeton eines dreistöckigen Gebäudes hat übermässige Risse und Durchbiegungen hervorgerufen. Die Verstärkung des Tragwerks wurde durch Vorspannung der Rahmen mit Aussenspanngliedern durchgeführt, so dass zweigurtige Konstruktionssysteme gebildet sind. Die Analyse von Ursachen, die Verstärkungseinzelheiten und die Ergebnisse der Durchbiegungsmessungen sind im Artikel dargestellt.

1. INTRODUCTION

Administration building within an industrial complex in Arandjelovac, Yugoslavia, was constructed in 1978. During construction, and very soon after the building was furnished, appearance of cracks and appreciable deformations of the structure was noted. The condition of the building got worst in course of time, caused fear and uncertainty, until the owner has decided to call for help.

2. LAYOUT OF THE BUILDING

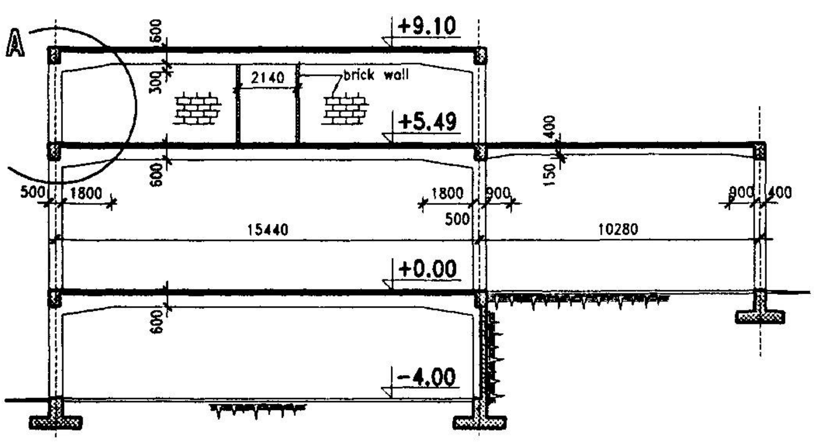

Fig. 1 Disposition of a transverse structure frame

The building is a three-story reinforced concrete frame structure with an annex, with dimensions 73,440 x 26,160 mm in plan, and clear story heights 3,400 + 4,890 + 3,061 mm. The span of the transverse frames, mutually spaced at 5,200 mm, is 15,440 mm in the main bay and 10,280 mm in the annex, Fig. 1.

Frame beams are haunched, of rectangular cross-section, dimensions 400 x 600 mm in the main bay and 400 x 400 mm in the annex. Frame columns, dimensions 400 x 500 mm are founded on strip foundations. Floor structure consists of semi-prefabricated solid reinforced concrete one-way slab, total thickness 120 mm.

Basement of the building is used for workshops, a restaurant is on the ground floor level, while the first floor is occupied by offices.

3. DESCRIPTION OF DAMAGES

The most outstanding cracks, up to 0.5 mm wide, appeared in the structure beams. In the span cracks spread in vertical direction along the entire depth of the beam, up to the roof slab. Towards the ends of the beams, the cracks gradually incline up to an angle of approximately 60 degrees regarding the horizontal plane, Fig. 2.

Appearance of inclined cracks at the top of the short columns in the top story, up to 12 mm wide, indicates failure of the column, Fig. 2. Due to increased deflections of frame beams, up to 60 mm in case of roof beam (span-deflection ratio 255), the functionality of the structure was violated, specially at the top story: appearance of doors and windows distortion and jamming, discomfort while walking, separation of partition brick walls from frame beams and columns, in some places more than 1 cm which violated their lateral stability as well as functionality of the installations. Excessive deflections of the roof beams caused loss of the draining layer slope and therefore retaining of water on the roof.

4. INVESTIGATION OF DAMAGE CAUSES

Adopted small cross-section dimensions of transverse frame elements, especially of beams (span-depth ratio 25.7), even visually give an impression of very slender, deformable frame. Emphasized deflections of slender beams could be accepted at the roof level, with a choice of initially larger slope of the roof. However, at the level of restaurant and offices, provision of functionality of the structure required a more careful choice of girder dimensions and check of deformations.

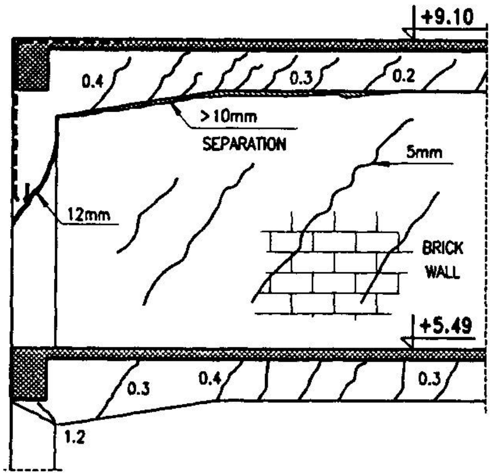

Fig. 2 Characteristic detail 'A' of damages

Subsequent structural analysis showed that, due to omission in load estimation and pattern, safety coefficients regarding the ultimate limit state in the large part of the structure are considerably smaller than required, and that the cross-sections are under-reinforced. For example, original structural analysis was carried out with assumption of uniformly distributed load, which underestimated "concentrated" effect of load from partition brick walls 120 mm thick, which are perpendicular to transverse frames in the corridor zone, Fig. 1. During service of the building, permissible tensile stresses in reinforcement in a large number of beam and column sections were exceeded, and notable cracks and deformations appeared.

The intensity of columns loading was additionally underestimated, since the stiffness of the beam haunches was not taken into account. As it is common in design of buildings, control of shear strength of columns was not carried out. In case of the third floor 'shallow frame' columns, that was an unacceptable oversight. Inspection of performed column reinforcement details showed that the tensioned reinforcement at the column-roof beam joint was not anchored deep enough into the column, which created conditions of "pulling out" the beam from the column, Fig. 2. At the same time, the lateral reinforcement of the columns is not sufficient to resist developed large shear forces, therefore, failure of column, decrease of rigidity of this story and increase of deformations of roof frame beam occurred.

Testing concrete quality by core drilling showed that, in some regions of the structure, concrete of lower quality than required by original design had been obtained.

Conclusion was that the damages of the structure as well as functional problems occurred due to uncarefull selection of cross-section dimensions, inadequate structural analysis and reinforcement detailing, as well as pure quality of workmanship.

5. THE STRUCTURE STRENGTHENING

While selecting the method of structure strengthening, three basic concepts were considered:

- Structure strengthening in existing stress and deformation condition. In that case strengthening elements are activated for additional loads only;

- simultaneously structure strengthening with partial correction of existing stress and deformation condition - strengthening elements are activated for a part of existing loads as well, by temporary raising of primary structure by hydraulic jacks, for example;

- correction of existing condition of stresses and deformations, applying additional permanent external forces, by tensioning the existing structure by prestressing tendons - “active strengthening”.

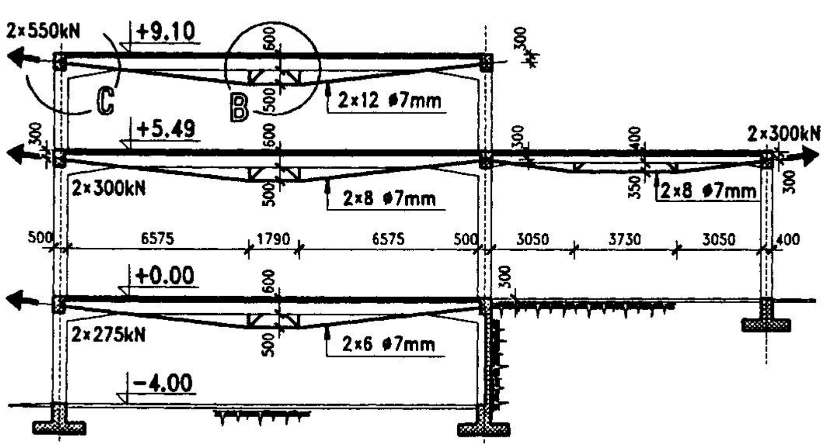

Fig. 3 Tendon layout

Two latter solutions were presented to the owner as technically acceptable. Taking into account the Owner's particular requirement of the least possible disturbance of the building service during strengthening works, the third concept was accepted as the most rational solution.

Correction of stresses and deformations of the structure as a whole was performed by shoring each frame beam with two prestressing tendons (6-12)#7, Yugoslav IMS system, conducted below the beams, Fig. 3. Required sag of 500 mm between the tendons and the beam soffit in the main span, and 350 mm in the annex was provided by inserting steel deviators - "chairs", constructed of steel tubes 63.5 mm in diameter, Fig. 4. Deviator verticals are placed in such a way that tendons equivalent forces

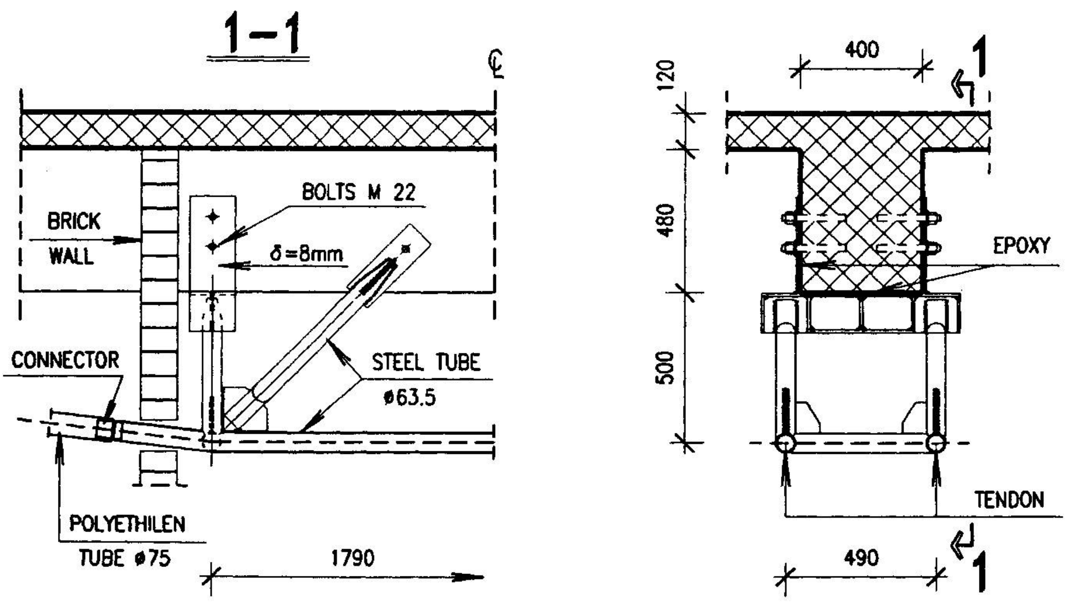

Fig. 4 Deviator detail 'B'

counteract the weight of two heavy partition walls in the top floor corridor. Tendon deviation over the central column at the connection with the annex was achieved by performing the cables through circular segment of steel tube. Tendon anchoring was carried out by a steel anchor box at the external side of the columns, Fig. 5. At straight segments the tendons are performed through polyethilen tubes, which are connected to steel elements after tensioning the tendons. Tubes and anchor boxes are injected with cement emulsion.

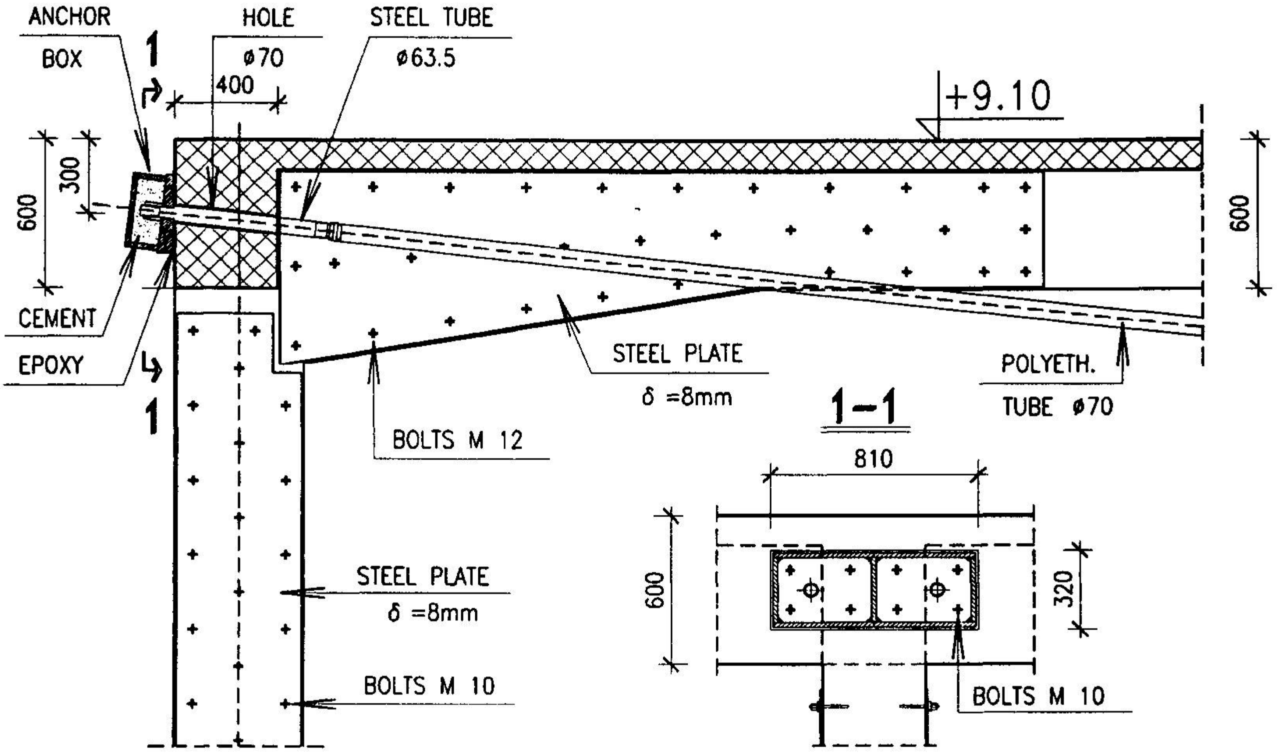

Fig. 5 Detail 'C' of tendon anchoring and lateral strengthening with steel sheets

Before tensioning of the tendons, injecting of cracks and local strengthening of beam and column ultimate shear capacity was carried out by well experienced method of glueing steel sheets onto lateral sides, by glue on the basis of low viscosity epoxy raisins with addition of filler [1]. Firm contact and glue spreading was achieved by bolts, Fig. 5.

The level of tensioning was so selected to fulfill the strengthening criterion that in all frame cross-sections required safety coefficient regarding ultimate limit state is provided. Order of tendon tensioning was from top to bottom story. Tendon tensioning was carried out by Institute for Testing of Materials of the Republic of Serbia - the IMS Institute.

6. RESULTS OF STRENGTHENING

During tendons' tensioning stage, IMS Institute carried out measurements and observations intended to verify accuracy of tendon forces as well as the structure response to actually realized forces, comprising, among other:

- measurement of change of the vertical displacement of the beams with mechanical deflection gauge during few days, up to stabilization (control of global tensioning effects and inertia of system response)

- measurement of strain change in steel elements - "chairs" and steel sheets (control of the friction effects and control of bond efficiency between steel sheets and concrete).

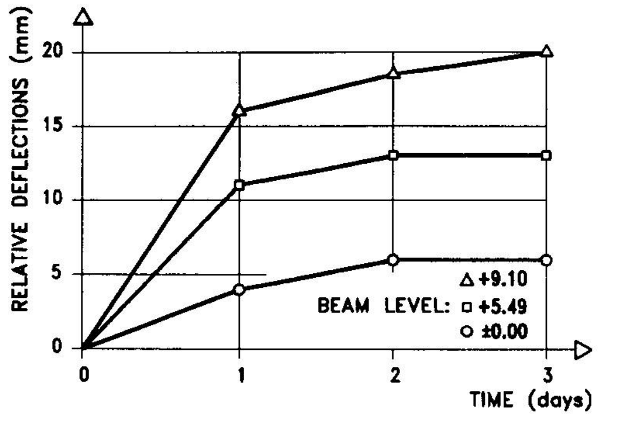

Fig. 6 - Stabilization of deflection in course of time

Results of performed measurements show good correspondence between measured and calculated relative deflections, with average stabilization time of three days following tensioning, Fig. 6. Measured values of change in strains of steel elements indicate well spreading of prestressing force, with expected level of friction, as well as that glued lateral steel sheets become activated.

On the basis of all measurements and observations, a conclusion was drawn that tensioning was successfully performed and that the structure response is within expected limits.

7. APPENDIX

Adopted tendon layout with steel chairs basically change the structure stiffness for additional loads, transforming the frame beams into the "two-chord system". But, utilization of the tendons with relatively small steel area tensioned to high stress level, as in presented case, the structure stiffness is only slightly increased, the basic static system practically does not alter. Control of ultimate bearing capacity of the strengthened structure shows that the effect of the tendons at the most appears as a particular case of external load. Rupture safety coefficient of the tendons actually depends only on the initial stress level during tensioning, since the stress variations in the tendons due to structure deformations under additional loads are negligible.

Concept of "active strengthening" by prestressing tendons was also used up to now by the authors of this paper in cases of bridge [2] and silo strengthening, when insufficient bearing capacity of the structure is prevailing. However, when simultaneous increase of the structural stiffness is also required, the two-chord concept can be also applied providing the greater stiffness of the lower chord by combining the structural - "passive" steel and prestressing tendons - "active" steel of the cross-section.

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Strengthening of Existing Precast Waffle Slabs Using Unbonded Tendons

Renforcement des planchers en dalle nervurée précontraints par post-tension sans adhérence

Verstärkung der vorgespannten Kassettendecken mit nachträglicher Vorspannung

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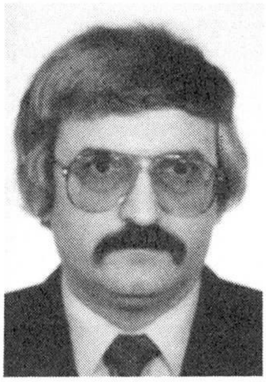


Antal Lovas, born in 1946, received his civil engineer degree at the Technical University Budapest, Hungary. His Ph.D. dissertation: "Static and Dynamic Analysis of Elasto-Plastic Plates". Research field: Modelling of Complex Structures.

SUMMARY

Inspecting a particular structural system tensioned together from precast reinforced concrete elements, considerable corrosion of the reinforcing tendons was found. The paper outlines some questions of dimensioning strengthening by supplementary post-tensioning.

RÉSUMÉ

L'expertise de bâtiments construits par éléments préfabriqués et solidarisés par post-tension a montré la corrosion considérable des câbles de précontrainte. L'article traite de certains problèmes du dimensionnement lié au renforcement des planchers endommagés en utilisant la précontrainte additionnelle.

ZUSAMMENFASSUNG

Es wurden bei den aus Fertigteilelementen, durch nachträgliche Vorspannung zusammengesetzten Gebäuden bei den Spannkabeln der Decken bedeutende Korrosionsschäden entdeckt. Der Artikel wirft einige Fragen zur Dimensionierung der Verstärkung durch zusätzliches Vorspannen auf.

1.INTRODUCTION

The IMS structural system tensioned together from precast reinforced concrete units is primarily fit for construction of dwellings and public buildings. The system was developed in Yugoslavia the late 1950s, however, it has been used in several countries. Since the middle of the 70s, about 400,000 m^2 floor has been built using this method. In the original version, in every field, one precast "waffle slab" unit is joined to the columns through the cut-outs at the edges by tensioning the columns together in both directions after filling the joint gaps with a material rapidly hardening (Fig. 1).

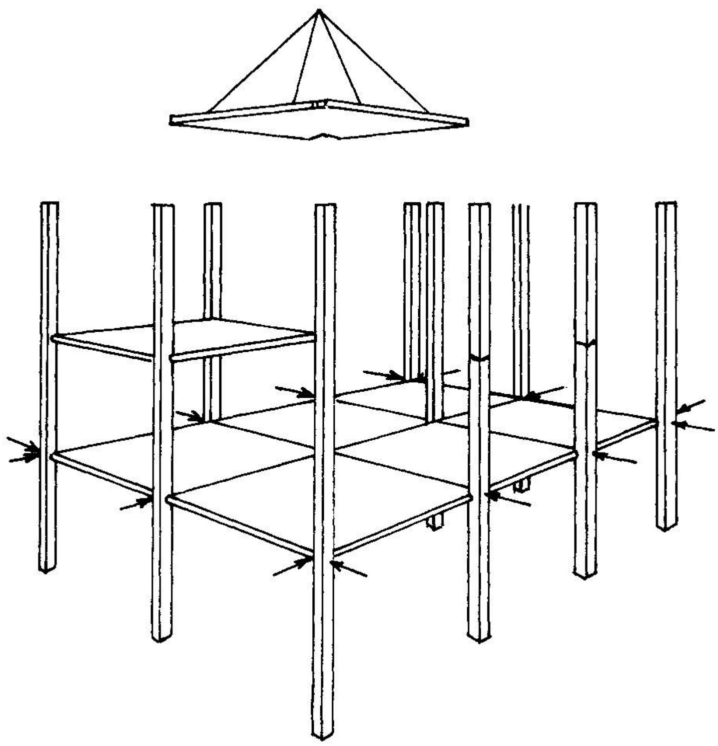


Fig.1 The IMS construction system

Later the system has been developed by tensioning the floor panels together from several (two or four) precast floor units. Thus, larger column distances became possible. Interaction of the multielement waffle slab structure is provided by primary tensioning in the column lines and by secondary tensioning applied in both directions in the span-thirds.

In the system, the horizontal and vertical connections of the structural units are created by post-tensioning, thus, in the multielement system, also the bending capacity is guaranteed by the tensioning alone.

Stresses of the waffle slab floor were computed by a lattice model, effects of the supplementary tensioning were considered as external forces concentrated in anchoring and inversion points of the tensioning tendons [2] (Fig. 2).

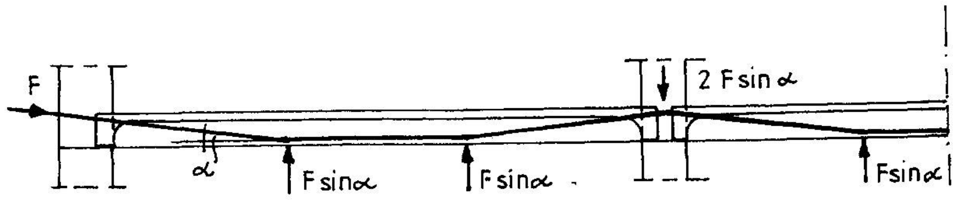


Fig.2 Concentrated external forces equal to tension force

The grid system will be statically solved by a spatial bar system program, thus, components of the stressing forces in the floor plane can be considered, consequently, also the normal forces can be computed in addition to bending moments and shear forces.

Efficiency of the strengthening by post-tensioning fundamentally depends on the distribution of the normal force originating from the stressing force applied concentratedly in the waffle slab ribs as well as on moments and reactions due to replacing forces arising in consequence of zig-zag cable arrangement and acting perpendicularly to the floor plane.

Parameter tests were carried out on a quarter of a fictitous building of 18,0*42,0m. Effect of the subsequent steps of the transverse post-tensioning, that of the column stiffness, role of the location of force application, effect of the possible different lattice models as well as the role of the 3,5 cm thick floor slab in stiffening have been investigated. Test results are summarized below.

4. COMPARISON OF THE POSSIBLE LATTICE MODELS

Distribution of post-tension forces introduced at the edges of the floor slab primarily affects the size of reaction force to be taken by friction in the floor-to-column joints. Determination of the normal force to be considered in the joint is a rather uncertain job, and it considerably depends on the computation model applied. Therefore, computations were carried out with different models for consideration of the effects of tensioning.

In the case of the spatial lattice model, degree of freedom in joints is 6, thus, for a large floor, accurate consideration of the floor structure may be difficult, and data preparation for a complicated model is rather time consuming.

The most simple model only consists of ribs (Model No.1), the next one - containing again only vertical bars - takes the influence of the slab between the ribs into account by computing the cover plate ribs (Model No.2). In-plane stiffening role of the 3.5 cm thick slab can be modelled by transverse grids either according to the Hungarian Standard with a width six times the slab thickness (6*v) (Model No.3-4.

2. LOAD BEARING CAPACITY LOSS

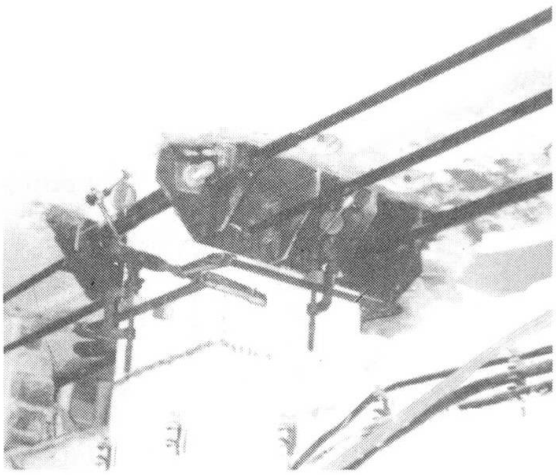
Since 1991, considerable corrosion of the reinforcing tendons in cable channels concreted subsequently has been found in IMS buildings in Hungary, mainly in floor-to-column joints.

Due to corrosion, efficiency of tensioning, thus, load bearing capacity of the floor decrease. Although probability of load bearing capacity loss in the most stressed cross-section increases considerably due to the gradual decrease of the tension force of about 30% according to computations [1], and surpasses the value of 10^{-4} allowed by the Hungarian Standard, this does not mean automatically the collapse of the floor.

However, the experience shows that corrosion failures do not occur gradually. In several cases, rupture of reinforcing tendons was found even in some cross-sections of only several year old buildings. Rearrangement of internal forces due to local rupture of the reinforcing tendons was investigated by the authors of the paper [1] by the spatial lattice model. It was stated that in the case of rupture of reinforcing tendons of one of the directions in the column-floor joint, the perpendicular moment of the same joint increased due to the rearrangement of the stresses so that probability of the progressive collapse of the floor considerably rises, i. e. strengthening of the structure becomes necessary.

3. FLOOR STRENGTHENING BY POST-TENSIONING

Load bearing capacity of the floor structures failed - especially in the case of multielement systems - can be most favourably guaranteed by supplementary post-tensioning. In order to avoid corrosion failures, tensioning strands in greased polyethilene tube should be used. Supplementary tensioning can be arranged - in function of the building's destination - either in the blocks of the waffle slab or under the floor plane usually by zig-zag strands (Photos 1 and 2).



Photos 1. and 2. Structural details

with/without eccentricity), or with the equivalent cross-section parameters suggested by Szilárd [3] (Model No.5-6. with/without eccentricity). Diagonal stiffening bars can be either hinged or clamped to ribs.

In any model, decrease of stresses computed from normal forces arising in ribs can be observed moving from the application spot of the tension force. Hinged or clamped joint of diagonal stiffening bars in model has no considerable influence, but eccentricity of the 3.5 cm floor slab cannot be neglected. At the column lying farther from the application of the tension force, the different models show an essential difference in the stresses that can be computed from the normal force. The most simple model fully neglecting the influence of the 3,5 cm slab is unfit for modelling, but even the model containing only cover plate ribs overestimates the stresses arising from normal forces (»35-40%) and bending moments that can be computed from tensioning (»15-20%) (Fig. 3).

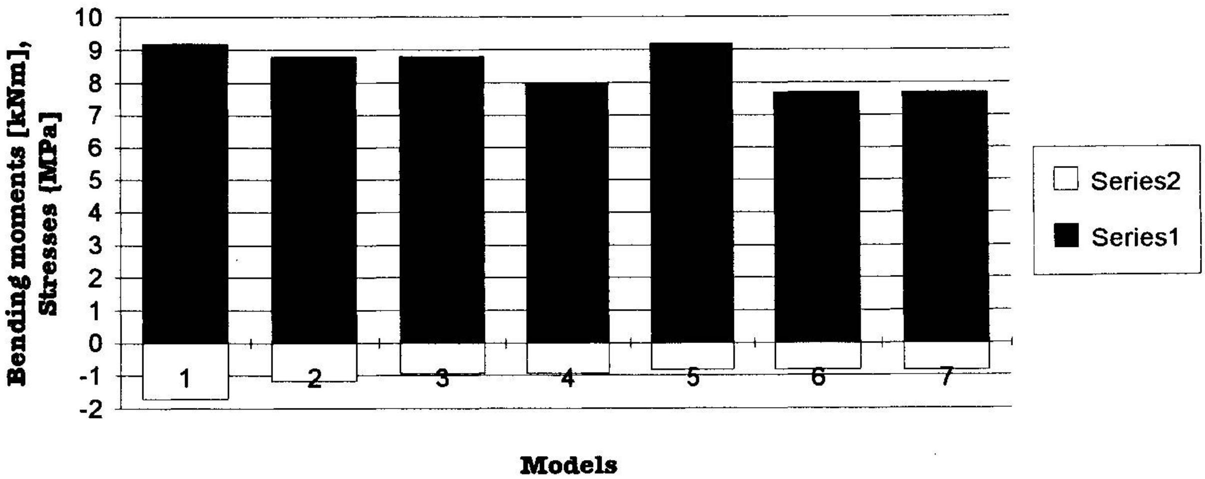


Fig.3 Effect of the different models. Series 1: Bending moment near the column, Series 2: Stresses computable from the normal force

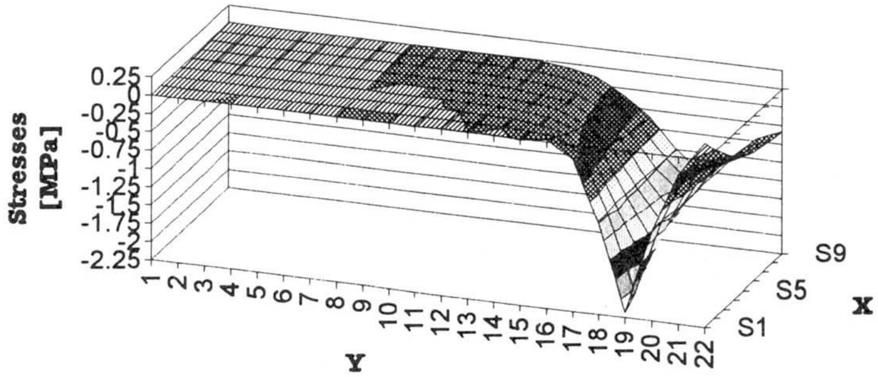
Summarizing, we can state that distribution of the tension force and the effect of the 3.5 cm thick floor slab can be only considered by a sophisticated model. The proposed model is: perpendicular cover plate ribs according to the Hungarian Standard + hinged bars after Szilárd + eccentricity; but of course, it can be a mixed model, too, consisting of bars and flat shells.

5. SUPPORTING EFFECT OF COLUMNS

Colums can be regarded as clamped between floor planes. From their displacement stiffness, a horizontal spring constant can be determined, and this was added to the vertical point-like support as boundary condition. The examination aims at determination of the degree of decrease of the efficiency of tensioning by horizontal displacement stiffness of the columns coming away from the introduction spot of the tension force. Comparison of the stresses computed from normal forces of the ribs in cables' direction after the complete stressing of the building show that this is a ≈3% decrease so it is not essential.

6. SUBSEQUENT STEPS OF POST-TENSIONING

Distribution of the normal force due to the tension force was investigated with the assumption that tensioning occurs symmetrically right and left from the axis of symmetry of the building. Formation of stresses computable from normal forces arising in ribs in direction of the cables is shown in two different steps in Figs. 4. It reveals that the influence of the particular steps of tensioning is considerable only in a zone of about 2 m of the columns - because of the essential in-plane stiffness of the grid. Consequently, the sequence of the tensioning has no particular influence on the final stresses arising from tensioning - in the case of the specific tension force of 1-2 MPa sufficient in practice.



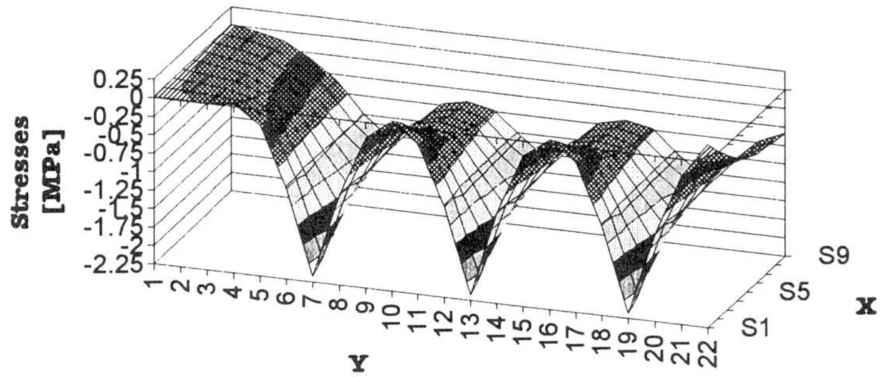


Fig.4 Distribution of stresses computed from normal forces arising in ribs in the cables' direction after the first and third steps of tensioning

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Repair of Precast, Prestressed Bridge Beams Using External Prestressing

Poutres de ponts préfabriquées et précontraintes réhabilitées par précontrainte externe

Reparatur von vorfabrizierten, vorgespannten Brückenbalken durch externe Vorspannung

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SUMMARY

Precast, prestressed beam and slab bridges are common worldwide for short to medium span lengths, as used for overpass bridges. Major lateral impacts of the beams may cause severe damage impairing the function of such bridges. The paper presents experience gained in the repair and strengthening of severely damaged precast, prestressed concrete beams of two bridges. The beams were repaired in situ using proprietary high-strength, shrinkage compensating micro-concrete and external prestress techniques which fully re-established the functional life of the bridges.

RÉSUMÉ

Les poutres et dalles de ponts, préfabriquées et précontraintes, sont réalisées dans le monde entier, pour enjamber les portées de faible et moyenne importance. Les chocs transversaux importants agissant sur les poutres peuvent entraîner de graves dommages risquant ainsi d'altérer la fonction de ces ponts. Cette communication présente les expériences réalisées par la réhabilitation et le renforcement des poutres sérieusement endommagées sur deux ponts. Ces éléments porteurs ont été réparés sur le site, par mise en place de micro-béton à haute résistance et sans retrait, avec application d'une précontrainte externe. Il a été ainsi possible de rétablir complètement la fonction de ces ponts.

ZUSAMMENFASSUNG

Vorfabrizierte, vorgespannte Balkenbrücken werden weltweit für Überführungen eingesetzt in der kurzen bis mittleren Spannlänge. Grössere Querstösse am Balken können ernsthafte Schädigung zur Folge haben, wodurch die Funktion der Brücke beeinträchtigt wird. Dieses Referat beschreibt die Erfahrungen der Reparatur und Verstärkung ernsthaft geschädigter Betonbalken von zwei Brücken. Die Balken wurden am Ort mit hochfestem, schwindfreiem Mikrobeton und externer Vorspannung repariert, wodurch die Funktion der Brücke vollständig wiederhergestellt wurde.

1. INTRODUCTION

Unforseen circumstances and forces of nature often change or cripple the service life of well planned and constructed structures. Such problems, accidental damage and other causes may require the repair and/or strengthening of a structure to re-instate and extend the functional life, as in the case of two precast beam and slab bridges which were severely damaged by vehicular impact.

Most rehabilitation and repair problems fall into the surface repair category and are mainly concerned with aesthetic, durability and serviceability aspects which involve solutions based on materials technology. More serious problems of structural distress fall into the strength and stability category which involve solutions based on design and construction technology. Such structural distress may affect the strength, stability and function of the structure.

The assessment of, and solution to the problem is complicated by the fact that a structure exists with defined member types and dimensions, with material properties that cannot readily be determined accurately and experiencing a stress state that may differ substantially from the original design values.

The required functional life of the structure plays a major role in the decision whether to repair the existing structure or to demolish and reconstruct with a new structure. Such a decision is largely influenced by site conditions, operational requirements, possible technical solutions, and the cost vs benefit relationship. The structural system and member cross-section may influence the choice and detail of the repair procedure. Certain structural arrangements and types of distress may favour plate or reinforcement bonding solutions while others may be better suited to external prestress solutions.

2. STRENGTHENING WITH EXTERNAL PRESTRESS TENDONS

The FIP guide [1], provides some general guidelines to the strengthening of structures with additional post-tensioning. Further information is found in [3] [4] & [5]

The repair and strengthening of structures with external prestress tendons requires three main elements in the structural system, viz, the tendons, the anchorages and for deflected tendons, deviators. These three elements induce a self equilibrating force system into the structure which is resisted by internal member stresses.

The following aspects need to be taken into account when considering the use of external prestress tendons on an existing structure:

- access to the structural member to indtall the external tendon system,

- the feasibility and suitability of providing the necessary anchorages and, if necessary, deviators on the structure to be repaired or strengthened,

- the tendon layout and profile which determines the total effect of the additional prestress on the structural member. A straight tendon profile is preferred in most cases due to a simpler and more economic construction procedure.

3. BRIDGE DAMAGE AND REPAIR

3.1 General

Probably the most common bridge deck system used world wide is a composite precast, prestressed concrete beam with in situ slab type deck. The prestressed beams may be pre- or post-tensioned and are generally simply supported. Freeway overpass bridges commonly span 20m or greater.

Overpass bridges are planned for a 5,100 m vertical clearance. However, road rehabilitation may result in overlays to the underpass route reducing the effective clearance to between 5,000 and 5,100 m. Inspection of freeway overpass bridge soffits indicates that minor impact from vehicle payloads occurs frequently causing minor edge spalling and consequent loss of concrete cover to reinforcement bars.

Occasionally major impact accidents occur with beam and slab decks particularly at risk suffering sever damage.

3.2 Impact Damage, Inspection & Condition Survey

Recently two precast beam and slab bridges were damaged, one being a 2-beam pedestrian bridge, and the other a 4-beam overpass for single lane traffic. Although the beam damage was of a similar nature, the effect on the bridge decks was very different.

3.2.1 Seventh Avenue Pedestrian Bridge
The bridge consists of simply supported decks supported on single column piers with stairs at either end. Each deck comprises two precast, prestressed concrete I-beams with a composite in situ reinforced concrete top slab spanning 20,73 m over a carriageway. The precast beams are 1160 mm deep spaced at 2,35 m and the slab is up to 180 mm thick. The deck cross section is illustrated in Figure 1 which also shows the repair detail.

The beams of the western span were severely damaged between the ¼ and ⅛ span position approximately halfway between transverse diaphragms by a transported excavator whose boom was too high to pass under the bridge. The boom struck the bottom flanges of both beams and the impact shattered the concrete exposing the reinforcement and the prestressing ducts and strands, and sections of the webs. The reinforcement was completely sheared and some strands of the lower prestress tendons of both beams were severed or damaged at the impact area, (Photo 1). The prestress effect on the concrete in the damaged area was lost, but the span was in no immediate danger of collapse. Since both beams were damaged to a similar extent, a simple longitudinal strut and tie system prevailed with no transverse redistribution of loads.

3.2.2 Candy Cornelia Vehicular Overpass
The bridge consists of simply supported decks supported on wall type piers. Each deck comprises four precast, prestressed concrete T-beams with a composite in situ reinforced concrete top link slab spanning 21,2 m over a carriageway. The precast beams are 1143 mm deep spaced at 1,524 m, and the slab is up to 165 mm thick. The deck cross-section is illustrated in Figure 2 which also shows the repair detail.

The western outer edge beam of the southern span was severely damaged between the ¼ and ½ span transverse diaphragms by a tipper truck whose tipper pan was raised during travel. The tipper pan struck the bottom flange of the edge beam and the lateral impact caused a block type shearing failure exposing the reinforcement and the prestress ducts at discrete locations and displacing sections of the bottom flange and the web, (Photo 2). The prestress effect on the concrete in the damaged area was lost, however the span was in no danger of collapse as redistribution of load could take place via the transverse diaphragms to the adjacent undamaged beams.

3.2.3 Beam Damage : General Observation
The nominal reinforcement in the beams for the two bridges was very different and the pattern of damage to the beams was different as can be seen on Photos 1 & 2. It is difficult to ascribe the form of the damage to the impact object or to the reinforcement containment. It is felt that the amount of reinforcement present and the bar spacing play a significant role in containing the damage. Nevertheless, the nature of the damage was very similar. The bottom flange was shattered between the transverse diaphragm beams. An elliptical shaped zone of the web was either shattered or displaced with heavy shear and torsion cracking extending to the underside of the top flange. Reflective impact spalling on the far face of the beam resulted in more extensive damage than on the impact face. The lateral resistance of the bottom compression flange is very low and it is clearly demonstrated that the transverse diaphragms play a very positive role of limiting the damage to the area between adjacent diaphragms.

3.4 Bridge Repair : Solution Development

The repair solution had to reinstate the function and operational life of the damaged bridge and had to incorporate the following operational requirements and procedures:

- The freeway traffic had to continue to flow without permanent lane restriction on each carriageway and a maximum speed restriction of 60 km/hr. Traffic deviation was allowed.

- The bridge had to remain open to overpass pedestrian or livestock traffic at all times to avoid the potentially dangerous situation of movements through the freeway traffic with the resultant risk of collisions and loss of life.

- Provision of an infill soffit slab between the soffit flanges of the beams to improve the lateral resistance against possible future impact damage. A minimum vertical clearance of 4,9 m had to be maintained at all times.

The rehabilitation of various bridge ancillary items and improvement to the general appearance of the bridge.

Various alternative solutions were investigated, ranging from repair of the existing beams to the demolition and reconstruction of the entire beam. The stringent operational requirements, the feasibility of repairing the beams by incorporating external prestress tendons and the lower construction period, cost and lesser traffic accommodation favoured the repair of the beams. The major work items are summarized in Table 1.

BEAM REPAIR		BEAM REPLACEMENT	
1.	Prop beam and remove damaged concrete between adjacent diaphragms up to underside of top flange only. (± 6 m length of beam)	1. 2.	Remove infill top slab and diaphragms to expose splicing reinforcement. Remove beam, kerb and tailing in sections and dispose rubble. (Demolishing a prestressed beam requires care, and fixity with diaphragms complicates the removal process)
	Concrete Removal: 2,0 m^3 vs 101,6 m^3		
2.	Shutter beam repair section and cast through access openings in top slab of deck.	3.	Shutter and cast complete beam on grade (cannot cast in situ due to required access to post-tension beam).
3.	Install external tendons and stress from top of deck @ ± 10 days age.	4.	Install bonded tendons and stress in casting bed prior to handling @ ± 28 days age.
4.	Epoxy injection of cracks in adjacent beam sections if necessary.	5.	Transport and lift beam into position on piers.
5.	Close access openings and remove props.	6.	Cast infill top slab and diaphragm splices.
6.	Clean outer beam face and surface treatment of exposed area of beam.	7.	Cast kerb & install balustrade/railing
	Estimated Cost R27 000 vs R92 000		

TABLE 1 : COMPARATIVE WORK REQUIRED FOR BEAM REPAIR AND BEAM REPLACEMENT (Candy Cornelia Overpass)

Each phase of the beam reconstruction procedure was modelled and analyzed using a grillage type finite element analysis. A normal design approach was adopted with the entire finite element modelling being based on linear elastic theory with the assumption that the principle of superposition is valid.

It is important to evaluate the probable construction and load history of the beams and composite deck as close as possible to that prior to the damage to determine the original design stress state. This is a matter of engineering judgement and an understanding of the construction sequence, the structural behaviour of the deck, the effect of the beam damage, and the repair procedure. The major uncertainties rest with concrete material properties and the shrinkage and creep effects that have taken place.

3.5 Repair of the Bridge Beams (Refer to Figures 1 & 2)

The repair of the beams to their original load carrying capacities centred around the use of high strength, shrinkage compensating micro-concrete and external unbonded prestress tendons. The accommodation of the anchors and the deviators, and the force transfer into the structure required careful evaluation of the load paths and details to ensure that the structural action could be accommodated.

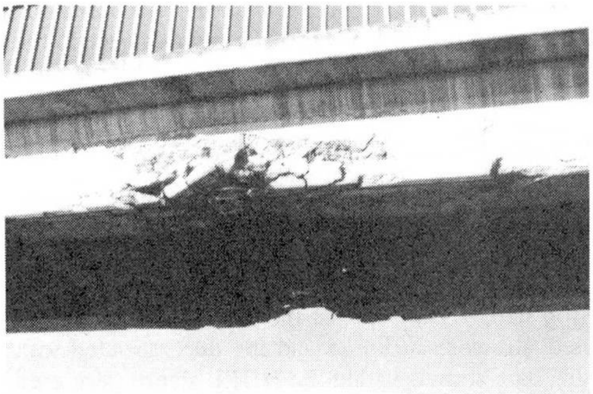

Photo 1 : Damage to Seventh Ave

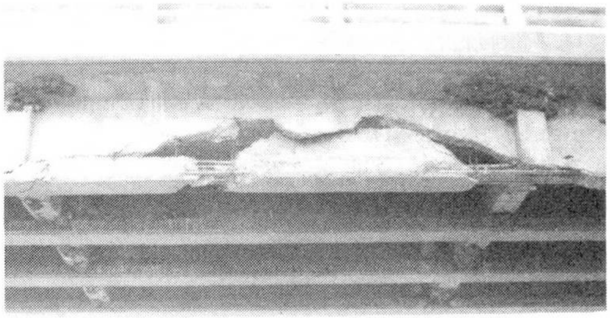

Photo 2 : Damage to Candy Cornelia

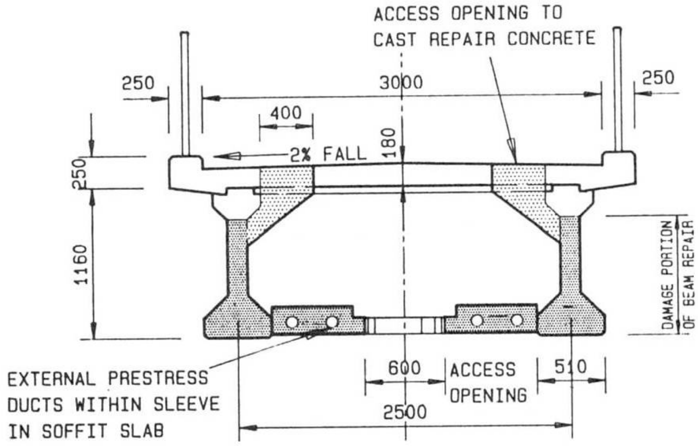

Figure 1 : Seventh Ave deck section indicating beam repair

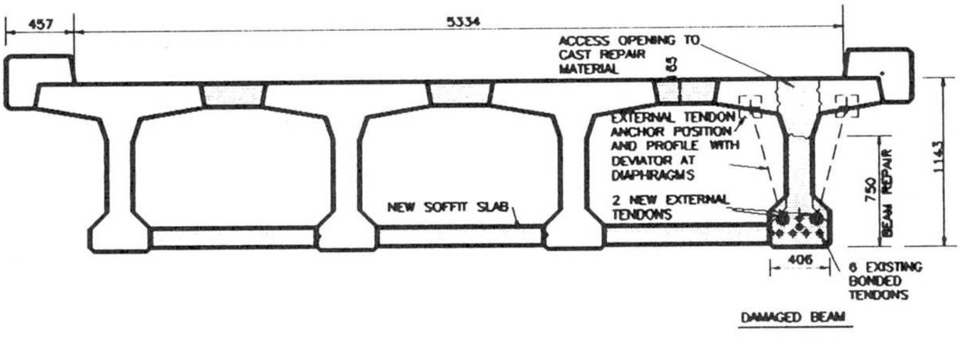

Figure 2 : Candy Cornelia deck section indicating beam repair

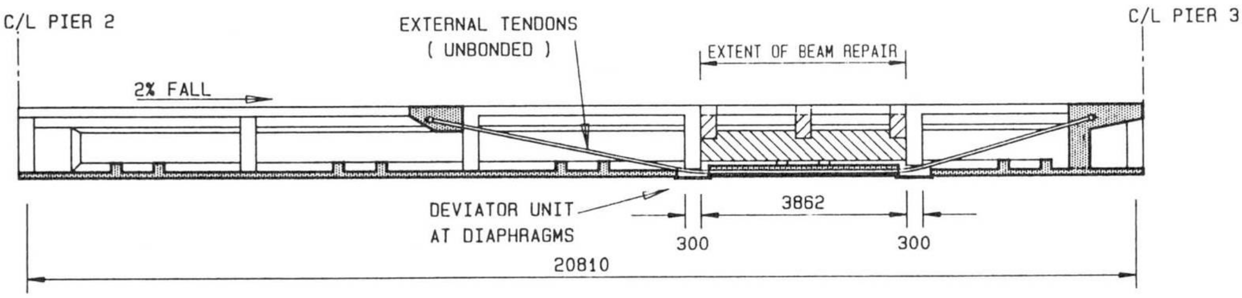

Figure 3 : Typical elevation of beam resonstruction (Seventh Ave)

The damaged beam sections between the diaphragms were removed to vertical faces at the ends and to a horizontal line following the web/top flange bulb junction. The concrete was carefully chipped away to prevent damage to the embedded reinforcement and prestressing ducts. New reinforcement was fixed and the beam formwork placed.

The flowable micro-concrete was batched and mixed on top of the deck and placed through carefully located access openings in the top slab to ensure complete filling of the voids. Light surface vibration on the outside of the formwork was applied to assist the flow and self-compaction of the micro-concrete, especially at the contact surfaces of existing concrete. The access openings were sealed with the last batch. The formwork was kept in place for 7 days for the initial curing of the micro-concrete.

The anchors were cast into a new anchor block just below the top slab, bearing against the exposed slab edge and the transverse diaphragm. The prestress tendons were installed, stressed, anchored and the ducts grouted with cementitious grout. External tendons comprising 7/12,9 mm dia. bare strands within the HDPE sleeve were used and no special provisions were made for monitoring or adjustments. The deviators were located at the diaphragm beams adjacent to the repaired beam sections. The tendon profile followed straightline segments between anchors and deviators.

Certain of the cracks in the bottom flange and webs of the beams extending past the area of reconstruction were were carefully injected with low viscosity epoxy resin to reinstate the material integrity.

Following the necessary surface finishing, the exposed surface of the beams were treated with a penetrating hydrophobic silane-siloxane system and then coated with a cementitious acrylic decorative coating to achieve a uniform texture and colour over the repaired and adjacent older concrete surfaces.

4. CONCLUSION

Where appropriate, the repair of structures with external unbonded prestress tendons provides a cost effective and technically efficient means to reinstate the function, structural integrity and durability of a structure following structural damage or increased load capacity and serviceability requirements. The severely damaged bridge beams described in this paper were repaired and the prestress effect reinstated albeit with a different tendon profile. The bridges can continue to fulfil their function with a new lease of service life.

5. ACKNOWLEDGEMENT

The permission granted by the Director General : Transport, Chief Directorate : Roads to publish the information contained in this paper is gratefully acknowledged.

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Post-Tensioning Level Criterion for Bridge Design and Rehabilitation

Critère du niveau de la précontrainte
pour le projet et le renforcement de ponts

Kriterien für den Grad der Vorspannung bei der Bemessung und beim Unterhalt von Brücken

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SUMMARY

The choice of post-tensioning level has important consequences on the serviceability and durability of bridges. Because of competition, designers often choose the most economical solution corresponding to the code requirements. While such a choice guarantees the resistance of the structure, the serviceability and the durability are not always satisfactory. From 20 years of experience it was observed that bridges with a low level of post-tensioning have often exhibited unsatisfactory behaviour characterised by cracking and instability of deflections. Through the analysis of the behaviour of more than 200 bridges, an appropriate level of post-tensioning is proposed to ensure the satisfactory behaviour of a bridge. A case study is presented.

RÉSUMÉ

Le choix du niveau de la précontrainte dans un ouvrage a des conséquences déterminantes sur son aptitude au service et sa durabilité. A cause de la compétition, les concepteurs de ponts ont tendance a choisir les solutions de précontrainte les plus économiques qui satisferont les critères des normes de construction. Ce choix assure évidemment la sécurité structurale de l'ouvrage; par contre l'aptitude au service et la durabilité ne sont pas toujours satisfaisantes. A partir de 20 ans d'expérience il a été observé que les ponts ayant un faible niveau de précontrainte ont souvent montré un comportement en service non satisfaisant, caractérisé par la fissuration et la non-stabilisation des déformations. A travers l'analyse du comportement de plus de 200 ponts, un niveau de précontrainte approprié est proposé afin d'assurer un comportement satisfaisant d'un pont. Un cas d'étude est présenté.

ZUSAMMENFASSUNG

Die Wahl des Vorspanngrades in einem Bauwerk spielt eine massgebende Rolle für seine Gebrauchstauglichkeit und Dauerhaftigkeit. Infolge des harten Wettbewerbes um preisgünstige Lösungen wird die Intensität der Vorspannung meistens nur so hoch gewählt, als dass sie gerade die Vorschriften der Normen erfüllen, unter Missachtung der eindeutigen Qualitätssteigerung durch eine höhere Vorspannung. Beobachtungen an bestehenden Brücken wahrend 20 Jahren sowie über 200 Belastungsproben haben einen eindeutigen Zusammenhang zwischen Vorspanngrad und Dauerhaftigkeit (Risse und Verformungen) gezeigt. Es wird ein konkreter Vorschlag für die Wahl der Vorspannung abgeleitet. Aus einer Fallstudie wird berichtet.

1. Load balancing method

The degree of load balancing β is the ratio of the equivalent load due to the curvature of the cable u to the permanent load of the structure g (Eq. 1) :

$$\beta = u / g \quad (1)$$

where; β : degree of load balancing, g : permanent load, u : equivalent load given by Eq. 2 :

$$u = 8 \cdot f \cdot P / \ell^2 \quad (2)$$

where; P : prestressing force, f : sag of the parabolic cable, ℓ : span.

2. Influence of load balancing degree and loading level on bridge stiffness

Figure 1 shows the correlation between the degree of load balancing and the ratio of measured deflections to those calculated in state I for 20 post-tensioned bridges. The ratio of measured to calculated instantaneous deflections increases when the load balancing degree decreases. This means that a low prestressing level allows cracking in the superstructure which results in deflections that are higher than those calculated in state I without any cracking.

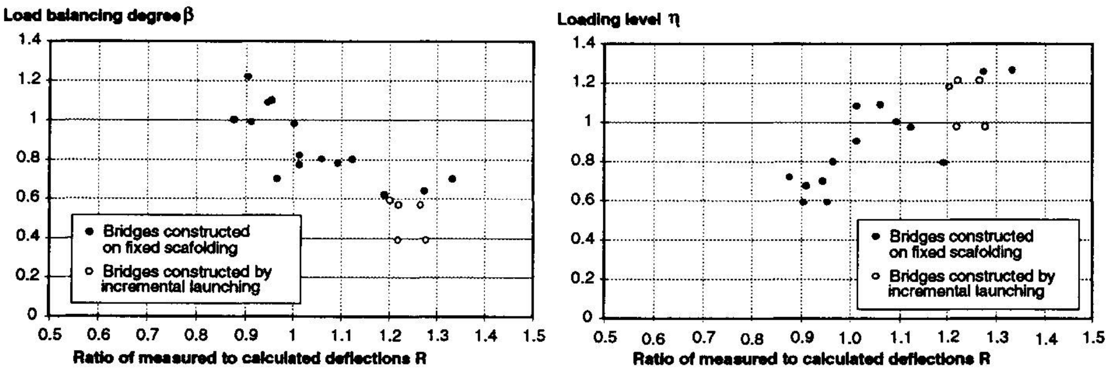

Figure 1: Load balancing degree β and loading level η versus the ratio of measured to calculated instantaneous deflections in state I

The loading level is defined as the ratio between the moment caused by the testing load and the moment of Code design load without multiplication by the load factors. This ratio is calculated at the middle of the loaded span considered as a simple beam. Figure 1 shows the correlation between the loading level and the ratio of measured to calculated deflections. A high loading level results in a high measured to calculated deflections ratio which means a decrease in the bridge stiffness caused by cracking.

3. Concept of compensation of deformations

It is not always easy to apply the load balancing method. An example is provided by the case of concrete structures with straight cables and with variable inertia. Thus, the concept of compensation of deformations can be used as an extension of the load balancing method. In this concept, the deformation of the structure due to any cable geometry is calculated and compared to the one due to permanent load. The ratio between these two deformations is defined as the degree of compensation of deformations and is also denoted β.

4. Recommended load balancing or compensation of deformations degree

The detailed analysis of 20 bridges together with a parametric study enabled us to establish a correlation between the load balancing degree, the loading level and the ratio of measured to calculated deflections. For a loading level equal to one (the code's design load), the load balancing degree is a function of R (the ratio of measured to calculated deflections) according to Eq.3.

$$\beta = 0.83 - 0.9 \ln R \qquad (3)$$

where : β : load balancing degree;
R : ratio of measured to calculated deflections or ratio of stiffness in state I to the mean stiffness with cracking.

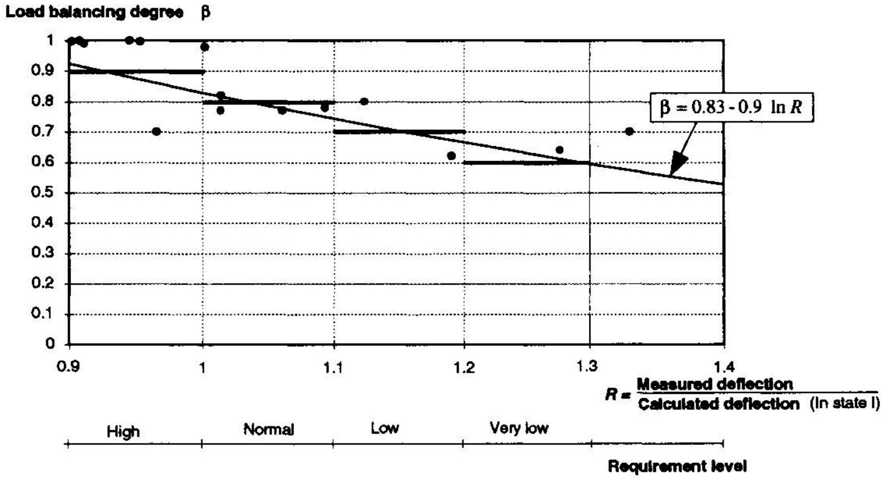

Figure 2: Recommended load balancing degree as a function of requirement level shown with the results of load tests carried out on 15 bridges constructed on fixed scaffolding

Figure 2 shows the load balancing degree described in Eq. 3 as a function of (R). The load balancing degree of 15 bridges constructed on fixed scaffolding are also represented as a function of (R) measured during a load test. Requirement levels are defined as a function of the importance of cracking (expressed by R) and a load balancing or compensation of deformations degree is recommended as a function of the importance of the bridge, the service load and the bridge environment.

The **high requirement level** is recommended when no cracking is allowed under the code representative loads. This requirement is necessary for bridges that have heavy loads or in unfavourable conditions. In these cases a β of 0.9 of permanent loads is necessary to fulfil a satisfactory behaviour. The **normal requirement level** means that the bridge could have a limited cracking but without risk with respect to the bridge durability. In this case a β of 0.8 is recommended. For little importance bridges with low loads and in favourable conditions a **low requirement level** can be accepted. In such bridges a β of 0.7 is sufficient.

5. Case of strengthening : Lutrive bridges

Lutrive bridges (North and South) are two parallel twin bridges. Each one supports one side of the Swiss national motorway RN9 between Lausanne and Vevey. Built in 1971/72 by the corbelling method with central articulations, the two bridges are gently curved (r = 1000 m) and each bridge is approximately 395 m long with four spans of: 57.95 - 129.50 - 143.50 and 64.00 m (Figure 3).

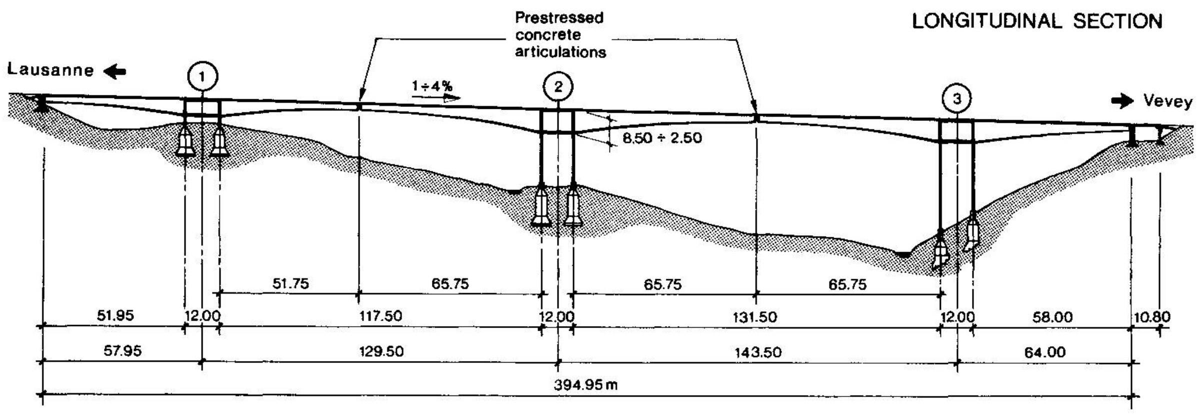

Figure 3: Longitudinal section of Lutrive South bridge

The two bridges have the same cross-section. It consists of a box girder of variable height and two slightly dissymmetric cantilevers, meant to reduce the effect of torsion in the curved bridges (Figure 4).

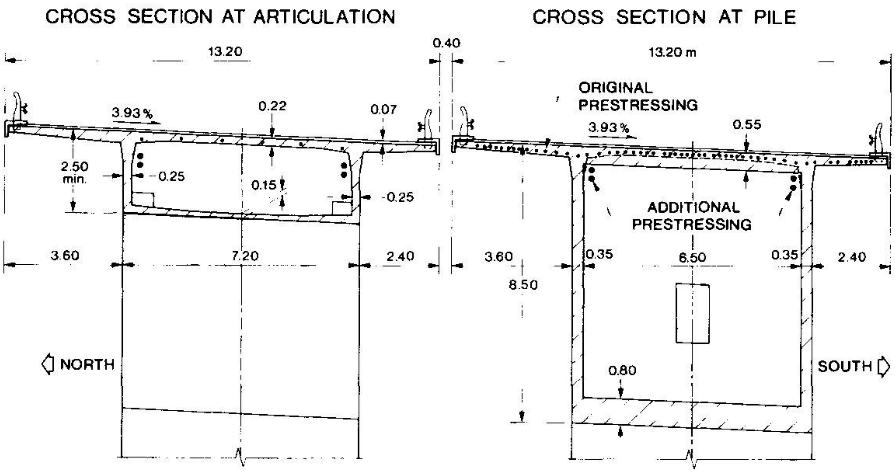

Figure 4: Cross-section at the central articulation and at the pile

The main span, 143.40 m long, underwent an approximate deflection of 16 cm. However this did not include the initial pre-camber of the span which was not measured but estimated at 11 cm. From 1973 to 1987 the deflection was a continuous downward movement which showed no sign of stabilisation (Figure 5).

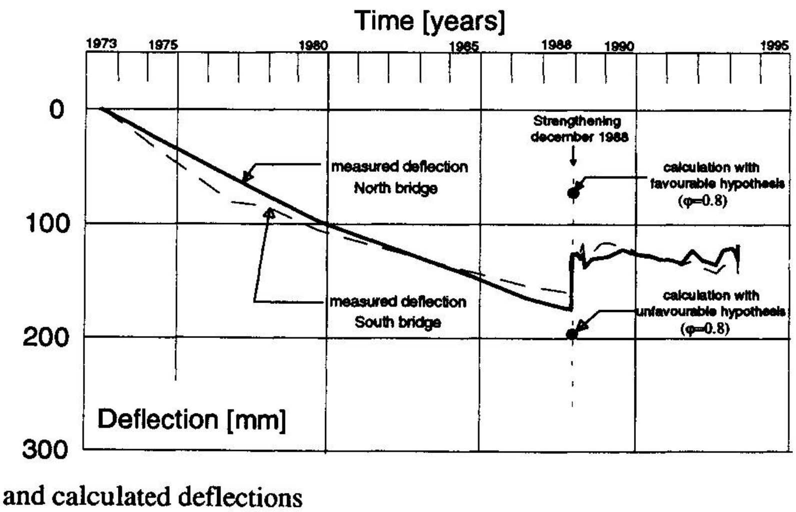

Figure 5: Measured and calculated deflections

Thus, in 1988 it was decided to « repair » the bridges with an additional external prestressing force of $P_0 = 4 \times 3345$ kN = 13380 kN for each bridge (4 Freyssinet cables with 18 strands pretensioned to 0.7 f_{tk}, where f_{tk} = 1770 N/mm²). Figure 6 shows the strengthening project.

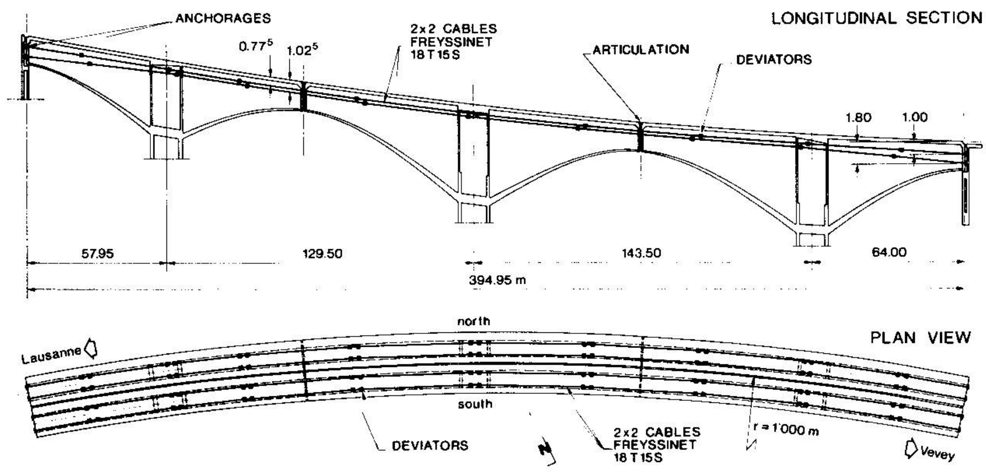

Figure 6: Scheme of the strengthening additional prestressing

The long-term deflections under permanent load of Lutrive bridges were calculated with the non-linear finite element software MAPSDIFF [6] which can take into account the time-dependent effect (creep of concrete), the cracking and the redistribution of solicitations. The calculation was carried out with two assumptions concerning the mechanical and the rheological properties of concrete and concerning the prestressing forces. Table 1 shows the values of the parameters considered in the two cases.

	Case	
	"favourable"	"unfavourable"
Elastic modulus E_{co} [kN/mm²]	35	35
Tensile strength f_{ct} [N/mm²]	2,5	0,0
Creep coefficient φ (∞, July 73)	0,8	0,8
Concrete self weight g [kN/m³]	25,0	26,0
Watertightness+Surface g' [kN/m']	28,0	34,0
Traffic quasi-permanent q [kN/m²]	0,0	2 kN/m² or 24 kN/m'
Average prestressing P_m	0,925 P_0	0,75 P_0
Final prestressing P_∞	0,85 P_0	0,68 P_0

Table 1: Considered values for the « favourable » and « unfavourable » assumptions

Figure 5 also shows the calculated deflections. It was noticed that the measured deflection (160 mm) is much closer to the one calculated with the « unfavourable » assumption (196 mm) than the one calculated with the « favourable » assumption (75 mm).

The probable degree of compensation of deformations β for Lutrive bridges can be estimated as follows:

- the degree of compensation of deformations calculated with an average prestressing force $P_m = 0.925\ P_0$ is equal to 0.79.
- according to our calculations, it seems that the real prestressing force was overestimated for these bridges and in reality we have to consider an average prestressing force equal to $P_m = 0.75\ P_0$.
- so the actual degree of compensation of deformations is equal to:

$$\beta_{origin} = (0.75 / 0.925) \times 0.79 = 0.64$$

After strengthening the value of β can be increased by:

$$\beta_{strengthening} = 4.43 / (20.5 + 3.5) = 0.18$$

with

4.43 [cm]: the calculated value of the elastic deflection due to the additional prestressing.

20.5 [cm]: the calculated value of the elastic deflection under self weight $\gamma = 25\ kN/m^3$.

3.5 [cm]: the calculated value of the elastic deflection under watertightness + surface of 28 kN/m'.

Thus, the final value of β is equal to:

$$\beta_{total} = 0.64 + 0.18 = 0.82$$

This new degree of compensation of deformations improves considerably the situation and it approaches the recommended value of $\beta = 0.90$ for high quality bridges.

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Post-Tensioning of the Limfjord Tunnel, Denmark

Précontrainte du tunnel de Limfjord, Danemark

Vorspannen des Limfjord Tunnels, Dänemark

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SUMMARY

By the early l990s, after 25 years in service, leakages and associated deterioration in the Limfjord Tunnel had reached a stage where an overall repair of the structure was needed to ensure and extend its lifespan. After studies of a wide range of possible repair methods, a post-tensioning repair strategy was chosen. The paper describes the post-tensioning project, focusing on the character and causes of the problems, the design basis and the design and construction works and the expected and the observed effect of the post-tensioning.

RÉSUMÉ

Au début des années 1990, et après 25 ans de service, les fuites et la dégradation du tunnel de Limfjord avaient pris de sérieuses proportions nécessitant une réparation majeure des structures afin de sauvegarder le tunnel et prolonger sa durée de vie. Après l'étude de nombreuses méthodes de réparation possible une stratégie de réparation comportant la précontrainte de la structure a été retenue. L'article expose le projet de précontrainte, le caractère et les causes des problèmes, les données de base des études et la conception, les travaux de construction et, finalement, les effets prévus et observés.

ZUSAMMENFASSUNG

In den frühen 90er Jahren, nach einer Dauer von 25 Jahren, hatten die entstandenen Undichtigkeiten und der damit verbundene Verfall im Limfjord Tunnel ein Ausmass erreicht, wo umfassende Reparaturarbeiten notwendig waren, um dessen Lebensdauer zu sichern und zu verlängern. Nach durchgeführten Untersuchungen von möglichen Reparaturmethoden wählte man ein Vorspannen. Der Artikel beschreibt das Vorspannprojekt mit Schwergewicht auf den Charakter und die Ursache der Probleme, die Bemessungsvoraussetzungen und Bemessung, Konstruktionsarbeiten und schliesslich den vorausgesehenen und den wahrgenommenen Effekt des Vorspannens.

1. INTRODUCTION

The Limfjord Tunnel in northern Jutland, opened in 1969, connects the cities of Aalborg and Nørresundby on each side of the Limfjord. At the same time the tunnel forms part of the continental highway, E45, connecting Scandinavia with the rest of Europe.

At present a daily average of nearly 40.000 vehicles pass through the tunnel; however, prognoses indicate that the traffic intensity will reach the capacity of the tunnel within the next 15-20 years.

Throughout its twenty-five years in service the Limfjord Tunnel has suffered from ingress of salt-laden water causing premature deterioration of the concrete and contamination with chlorides, which has led to extensive corrosion of the embedded reinforcement.

By the early 1990's the leakages and the associated deterioration had reached a stage where an overall repair of the structure was needed to ensure and extend its lifespan.

The operation and the maintenance of the tunnel is handled by the Bridge Department of the Danish Road Directorate with the assistance of consultants, contractors and service companies, some of which perform a 24-hour service.

2. THE TUNNEL STRUCTURE

The tunnel is a reinforced concrete structure with a total length of 945 m, of which 510 m is immersed precast tunnel units and 43 m is in-situ cast tunnel.

The immersed part consists of five 102 m long precast reinforced concrete units joined together to form a monolithic structure. The units were cast in a dry dock in 12,8 m long sections, separated by 1,8 m wide gaps into which reinforcement bars protruded. The concrete for these gaps was poured after the sections had shrunk.

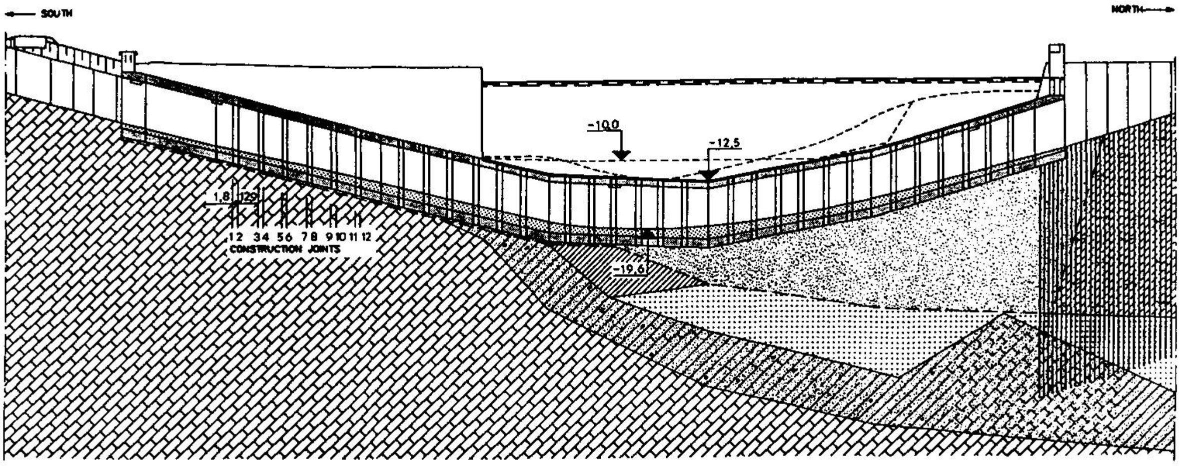


Fig. 1 Longitudinal section

The units were waterproofed with a 2 mm butyl membrane which, as described in the introduction, has never been fully effective.

The tunnel has a typical box type cross-section with two separate tubes each carrying 3 traffic lanes of 3.5 m and 2 pavements of 0.75 m.

The tunnel is founded on an approximately 1 m thick sand bedding jetted in place after positioning of the tunnel units, and is supported by the portal buildings at the north and south ends, the northern portal building being founded on piles.

Longitudinal movements are taken up by expansion joints at the portal buildings.

3. HISTORICAL REVIEW

As described in the introduction, leakages were observed in tunnel walls and ceilings shortly after the tunnel was put into service.

The leakages are primarily located in the immersed part of the tunnel at construction joints between the 12,8 m sections and the 1,8 m gaps where cracks extend in full depth through walls, top and bottom.

The repair of the leakages and the deteriorating concrete is complicated because of the natural tendency of the structure to expand and contract in warm and cold weather. When the tunnel contracts in cold weather, longitudinal tensile stresses are introduced in the structure as friction between the surrounding sand filling and the structure hinders it from moving freely. Therefore, as the tunnel has intentionally been under-reinforced in the longitudinal direction, the cracks at construction joints are wider in winter than they are in summer.

Furthermore, repair is complicated by settlement of the northern part of the tunnel which has caused additional tensile stresses in the bottom tunnel slab, due to the deformation of the structure.

The post-tensioning repair strategy was chosen after detailed studies and analyses of a wide range of possible repair methods with respect to technical, economic and traffic aspects as well as elements of risk involved.

4. THE POST-TENSIONING REPAIR STRATEGY

The basic idea of the longitudinal post-tensioning of the tunnel is to impose a permanent compressive force sufficient to ensure that the entire tunnel cross-section is always under compression.

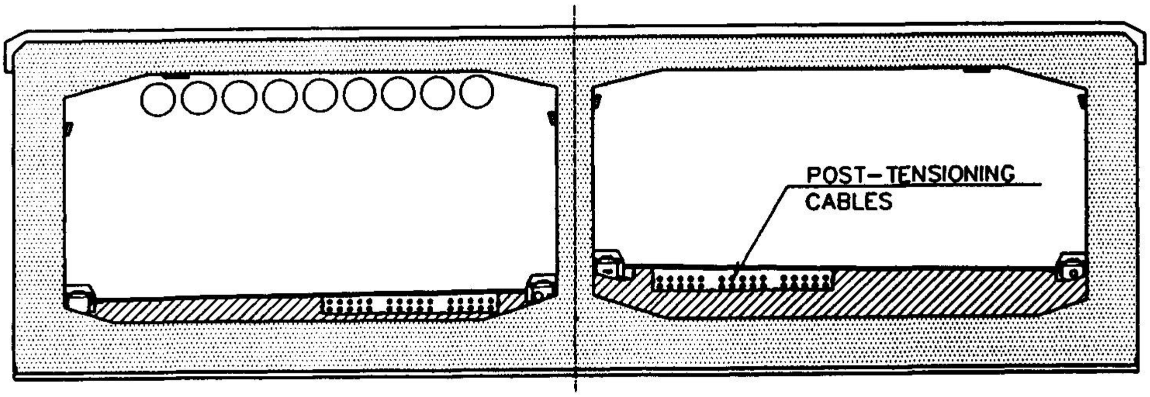
Consequently and most importantly, the post-tensioning is expected to eliminate tensile stresses and crack movements caused by the thermal contraction of the tunnel structure in the cold season. This is considered to be a fundamental condition of a future successful, durable and effective overall repair of the structure.

The necessary post-tensioning force was determined from detailed analyses of the varying longitudinal forces acting on the tunnel structure when the structure expands or contracts. The assessment of all the parameters influencing the longitudinal forces and their ranges of variation is based on comprehensive registrations, laboratory analyses and expert's opinions.

A fundamental part of the design basis was established from registrations and analyses of the longitudinal movements of the tunnel at the expansion joints at the north and south ends of the tunnel, recorded since the tunnel was put into service in 1969. Furthermore, from registrations and analyses of the corresponding temperature in and around the tunnel, and from laboratory determinations of the thermal expansion coefficient and the modulus of elasticity of the concrete, it was possible to determine the probable magnitude and distribution of the longitudinal force, the fixation force, caused by friction between the tunnel structure and the surrounding sand filling.

Having established the design basis, the necessary post-tensioning force was found to be approximately 120 MN.

The post-tensioning was established by installation of a total of 60 post-tensioning cables in the bottom slab of the tunnel, extending from the expansion joint at the north end to the expansion joint at the south end.



5. CONSTRUCTION WORKS

Construction works in the tunnel, including application of the post-tensioning, commenced in late October 1993 and were completed in June 1994. The works included the following main activities

- Removal of existing asphalt covering
- Removal of mass concrete
- Construction of anchorage blocks for post-tensioning cables
- Installation and tensioning of cables
- Casting of new mass concrete
- New asphalt covering

The construction works were completed in the north-bound tunnel tube before starting in the south-bound tube, i.e. post-tensioning was first applied in the north-bound tube, and completed 3 months later in the south-bound tube.

Traffic through the tunnel was maintained throughout the construction period in the tube free of construction works, constantly adjusting the direction of the traffic in the center lane to correspond to the dominating traffic direction.

Removal of both asphalt covering and mass concrete was done by large scale milling. Milled materials were immediately loaded on waiting trucks and removed from the construction site.

After removal of the mass concrete the reinforced concrete anchorage blocks for the post-tensioning cables were constructed. To ensure that the post-tensioning force is effectively transferred from the anchorage blocks to the existing tunnel structure approximately 35,000 anchors of reinforced steel and threaded rods were fastened in drilled holes (0.5-1.0 m depth) in the bottom slab of the tunnel before casting the anchorage blocks.

After completion of the anchorage blocks and installation of cable ducts the post-tensioning cables were installed. The cables were tensioned in a predetermined order using conventional technique and equipment.

6. EFFECT OF POST-TENSIONING

The primary goal of the longitudinal post-tensioning is to ensure that any cross section of the tunnel is permanently subjected to a compressive force, thereby eliminating the seasonal movements of the cracks occurring primarily at casting joints.

The post-tensioning is expected to result in an overall compression of the tunnel structure which will widen the gaps at the expansion joints at the north and south ends of the tunnel.

Furthermore, the post-tensioning is expected to reduce future settlements of the northern part of the tunnel.

In order to establish whether expectations have been fulfilled, relevant data have been collected on the tunnel movements and temperature during and after the tensioning process.

For reasons of continuity and comparability with past registrations, collection of data is primarily based on existing points of measurement.

6.1 Longitudinal Movements

Measurements of the tunnel movements at the expansion joints show that the tunnel, six months after completion of the post-tensioning, has shortened by approx. 15 mm at the north expansion joint and 5-6 mm at the south expansion joint.

The theoretical values, computed on the basis of the design model used for determining the post-tensioning force, are a 16 mm shortening at the north expansion joint and an 8 mm shortening at the south expansion joint.

The previous discussion concerns the recorded movements and the corresponding theoretical values before and after the post-tensioning only. Similar registrations and analyses of the tunnel movements during the two post-tensioning phases have shown equally good agreement between the theoretical values and the corresponding registrations.

The results are summarized in the table below.

	Shortening Post-Tensioning Cables East Tube [mm]		Shortening Post-Tensioning Cables West Tube [mm]		Total Shortening [mm]	
	Measured	Calculated	Measured	Calculated	Measured	Calculated
North End Tunnel	9-10	app. 11	app. 6	app. 5	15-16	app. 16
South End Tunnel	3-4	app. 3	2-3	app. 5	app. 6	app. 8

Table 1 Summarized results

It is worth noting that additional permanent compression is expected, especially in the northern part of the tunnel, as the tunnel temperature reaches its minimum. The magnitude of the additional shortening is estimated at approximately 10 mm.

6.2 Settlements

The most recent levellings in the tunnel prior to the post-tensioning indicated that settlement of the northern part of the tunnel had resumed after years of stagnation.

Levellings in the tunnel after the post-tensioning indicate as expected that the settlement has again stagnated. However, given the short period of time since the completion of the post-tensioning, it is still too early to draw a conclusion on this issue.

6.3 Local Measurements

Strain-gauge measurements at cracks and casting-joints in representative areas confirm that the tunnel has been compressed at all locations, as a result of the post-tensioning. However, the magnitude of the compression varies, which is ascribed to the varying character of the cracks and their location in the tunnel.

6.4 Summary

To summarize, all measurements on the tunnel during and after the post-tensioning have confirmed the expected effect of the post-tensioning on the tunnel.

Finally, although the post-tensioning has resulted in an overall compression of the structure as well as local compression of cracks, and although leakages have been reduced after the post-tensioning, the authors wish to stress that:

- The post-tensioning is not expected to stop leakages

- The post-tensioning is expected to stop the seasonal movements of cracks and make it possible to stop leakages by injection

7. CONCLUDING REMARKS

The post-tensioning of the tunnel was the first and most important step in an overall repair of the structure. Future repair phases include the following activities:

- Watertightening, i.e. injection of cracks and leakages
- Repair of concrete and reinforcement in deteriorating areas
- Preventive measures

Furthermore, future activities also include fire protection of tunnel ceilings, new covering on tunnel walls and other activities aimed at generally enhancing the standard of both the structural components and the technical installations.

When these repairs have been completed, no further major repairs are expected within the following approximately 30-40 years.

The total cost of the repairs, including the post-tensioning, is expected to amount to approximately Dkr 100 mio., or one tenth the cost of a new tunnel.

Behaviour of Beams Prestressed or Strengthened with Fiber Reinforced Plastic Composites

Comportement de poutres précontraintes ou renforcées avec des composites à fibres de carbone

Biegetragverhalten von Balken mit faserverstärkten Kunststoffen

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SUMMARY

Flexural behaviour of rectangular concrete beams prestressed with Carbon Fiber Reinforced Plastic (CFRP) tendons and beams externally strengthened by epoxy bonded CFRP laminates are investigated. Comprehensive test data is presented on the effect of carbon composites on the cracking behaviour, deflections, ultimate strength and failure modes. Theoretical analysis using a computer software is presented to predict the ultimate strength and moment-deflection behaviour of the beams.

RÉSUMÉ

La recherche présentée traite du comportement en flexion de poutres en béton à section rectangulaire précontraintes et renforcés par des fibres de carbone (CFRP) ou consolidées par des plaques collées à l'extérieur de la poutre. L'influence des composites à fibre de carbone sur le développement des fissures, les déformations, la résistance à la rupture et les modes de rupture a été étudiée au cours d'essais systématiques. Une méthode d'analyse numérique destinée à prédire la résistance à la rupture et la relation moment-flèche des poutres est présentée.

ZUSAMMENFASSUNG

Die vorliegende Untersuchung behandelt das Biegetragverhalten von Betonrechteckbalken, die mit Spanngliedern aus kohlenstoffaserhaltigem Kunststoff (CFK) vorgespannt oder mittels extern angeklebten CFK-Lamellen verstärkt wurden. Es werden umfangreiche Versuchsergebnisse zum Einfluss der Kohlenstoffverbundwerkstoffe auf das Rissverhalten, Durchbiegungen, Bruchfestigkeit und Versagensarten präsentiert. Zur Vorhersage der Tragfähigkeit und der Momenten-Durchbiegungsbeziehung wird ein Computerberechnungsprogramm eingesetzt.

1. INTRODUCTION

Corrosion of prestressing steel tendon is one of the major problems which affect the lifespan of bridges and other prestressed concrete structures, especially in coastal areas. Composite materials offer unique advantages, apart from high fatigue strength, in solving many practical problems in areas where conventional materials fail to provide satisfactory service life. Unlike steel, composites are unaffected by aggressive environmental conditions and have good corrosion-resistant property.

Carbon Fiber Reinforced Plastics (CFRP) represent an ideal alternative for prestressing steel and can also be used for repair and rehabilitation of damaged structural members. The main attributes of CFRP composites are high strength, lightweight, resistance to chemicals, good fatigue strength and non-magnetic and non-conductive properties. The limited information available on design, standards and evaluation criteria on CFRP prestressed / strengthened structures necessitates the need for further research in this field. This paper presents the experimental studies on flexural behavior of concrete beams prestressed / strengthened with CFRP tendons / laminates. The effectiveness of CFRP composites was assessed in terms of deflection, ultimate load and cracking, which were evaluated for a series of test beams. Theoretical analysis was also performed for prediction of load-deflection behavior of the beams.

2. RECTANGULAR BEAMS PRESTRESSED WITH CFRP TENDONS

2.1 Experimental Program

The CFRP prestressing tendon used in the study was carbon fiber composite cable (CFCC 1x7, 12.5Φ) composed of PAN type carbon fiber and resin [2]. The basic properties of CFCC tendons (CFCC 1x7, 12.5Φ) and shear stirrups (CFCC 1x7, 7.5Φ) are presented in Table 1. Four rectangular beams of dimensions 254 mm x 254 mm x 2490 mm were cast with minimum reinforcement based on the ACI code requirements. Two CFRP prestressing tendons were used with an eccentricity of 83 mm and pretensioned to a force of 185 kN. CFCC rectangular helicoidal hoops were used as the shear reinforcement. A prestressing bed facility was made use of for fabricating the specimens at Florida Atlantic University. Strain gages were installed at midspan on prestressing tendons to measure the deformation in cables during pretensioning. Load cells were placed at the dead ends of prestressing tendons and connected to a data acquisition system. The CFRP tendons were pretensioned to about100 kN (60% of breaking strength) and the anchorage loss was about 5% to 7%. The prestressing forces and concrete compressive strengths for the beams are presented in Table 2. The beams were tested after 28 days of casting over a simply supported span of 2440 mm (96 in).

Table 1 Basic characteristics of CFCC

Form, Size	Diameter	Effective section area	Breaking strength	Tensile strength	Tensile modulus	Elongation
CFCC1x7 12.5Φ	12.7 mm	76.0 mm^2	167 kN	2.20 GPa	141 GPa	1.6%
CFCC1x7 7.5Φ	7.4 mm	30.4 mm^2	60 kN	1.96 GPa	130 GPa	1.6%

The load points were 533 mm (21 in) apart from the center of the span for Beam-1 and 280 mm (11 in) for the other beams (Fig. 1). Four vibrating wire strain gages were mounted on the concrete beams along the depth at midspan. Dial gages were set up to measure deflections at midspan, support and tendon slip. Static load was applied in 5 to 10 kN increments and monitored by a load cell connected to System 4000 data acquisition system. Concrete strains, midspan and support deflections, and tendon slip were recorded at regular load intervals together with crack pattern.

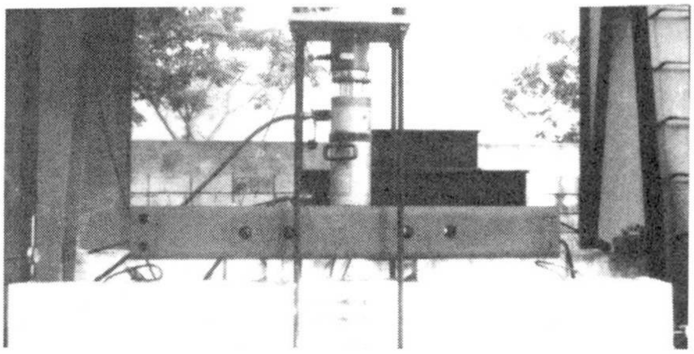

Fig. 1 Typical beam test setup

Table 2 Specimen characteristics

Beam no.	Prestressing force kN	Ratio f_i/f_u percent	Anchorage loss percent	Concrete strength MPa
Beam-1	183.7	55.0	6.1	60.6
Beam-2	185.6	55.6	6.6	75.2
Beam-3	188.8	56.5	5.0	53.3
Beam-4	187.1	56.0	4.8	47.9

2.2 Results and Discussions

The ultimate flexural strengths of beams compare well with the predicted values based on the conventional theory for PC beams with prestressing steel tendons (Table 3). Beam-1, with a larger pure bending region, failed by diagonal tension at a lower moment value. The first crack moments for all the beams were lower than the predicted values. The moment vs. deflection of all the beams show the typical bilinear behavior (Fig. 2). Crack propagation in all the beams was similar. The first initial crack was observed under the load points at a load of about 57 kN. Then new cracks formed between the load points symmetrically. When the applied load reached 70% of the ultimate value, more cracks developed beyond the load points perpendicular to the line of principal stress. Some cracks appeared at the tendon level when the applied load reached 80% of the ultimate value. Beam-1 failed by diagonal tension all of a sudden and the other beams failed by tendon rupture with sufficient warning.

Table 3 Test results

Beam number	First crack moment (kN-m)		Ultimate moment (kN-m)		Maximum deflection (mm)	Failure mode
	Experimental	Theoretical	Experimental	Theoretical		
Beam-1	27.4	36.2	50.0	65.9	23.1	DT
Beam-2	26.3	36.8	68.1	66.8	33.0	F
Beam-3	27.2	34..8	66.5	65.3	27.2	F
Beam-4	28.2	37.1	66.6	64.8	23.6	F

No tendon slips were observed in all the beams. The strain distribution across the depth was essentially linear. The upward neutral axis shift was observed with increasing load. When the load reached its ultimate value, the compression zone was about 50 mm from the top.

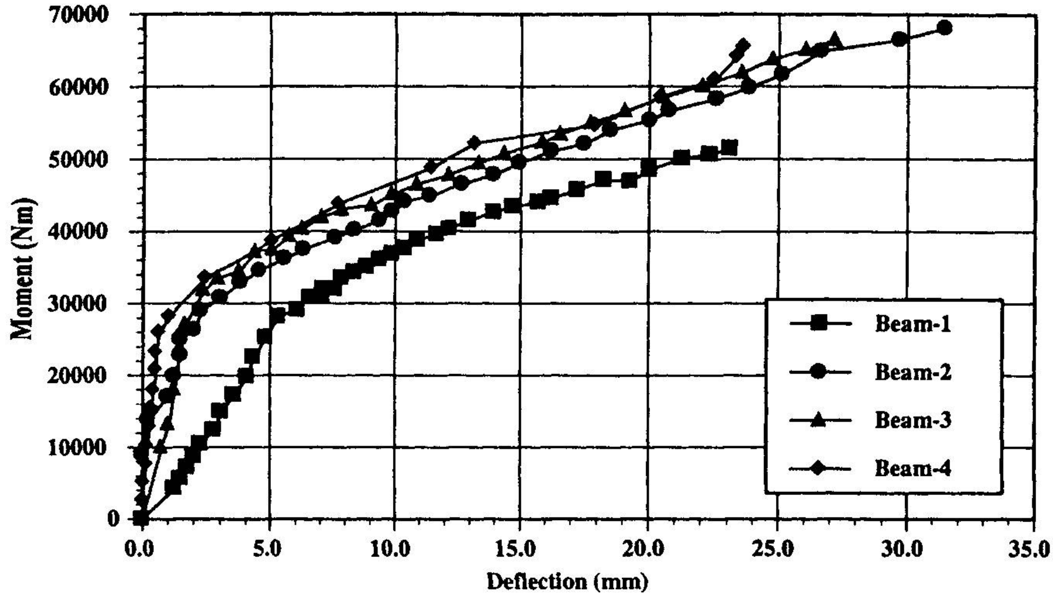

Fig. 2 Moment vs. deflection

3. RECTANGULAR BEAMS STRENGTHENED BY CFRP LAMINATES

3.1 Experimental Program

Six reinforced concrete rectangular beams were cast with minimum steel reinforcement according to ACI 318-89 code provisions (Fig. 3). One layer of CFRP laminate was bonded to one reinforced concrete beam. Two reinforced concrete beams bonded with two layers of CFRP laminates and another two with three layers of CFRP laminates were prepared for testing and the remaining beam was used as the control beam. Carbon fiber prepreg laminate with a tensile strength of 2,758 MPa and tensile modulus of 141 MPa was used in the study. The two-part epoxy resin consisted of a main resin and a curing agent with a mixing ratio of 2:1. The cured adhesive had a tensile strength of 60 MPa. The CFRP laminates were bonded to the tension face of the beams following the procedure detailed in ref. 3. The beams were tested under two-point static loading over a simple span of 2438.4 mm (8 ft.). Deflections were measured at midspan, support and section at 609.6 mm (2 ft.) from midspan. Strains and deflections at various locations, crack patterns and the applied load were recorded for every load increment till the ultimate load capacity of the beam.

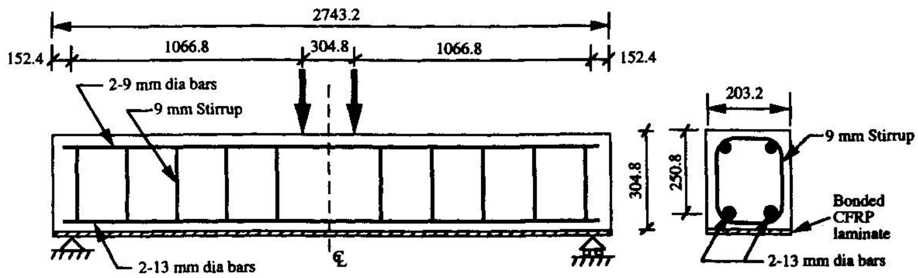

Fig. 3 Reinforcement details of rectangular beam

Note: All dimensions are in mm

3.2 Theoretical Analysis

The determination of ultimate strength of the beams based on ACI method needs modification since the fiber reinforced plastics have no definite yield plateau. Hence, a computer program has been developed to perform flexural analysis of rectangular sections with single or multiple FRP reinforcement layers. The program is based on the stress-block factor approach [4] considering the linear-elastic stress-strain properties of FRPs and parabolic stress-strain relationship of concrete.

3.3 Results And Discussions

A significant increase in the moment capacity can be observed for the beams bonded with laminates over and above the control beam (Fig. 4). All the beams show a significant decrease in the ultimate deflection, indicating that the stiffening effect of the CFRP laminates reduces the structural deformation until failure. A considerable increase can be observed in load carrying capacity of the laminated beams over the control beam at ultimate load stage (column [2] of Table 4). The ratio of experimental to theoretical ultimate load values (column [4]) can be seen to decrease with the number of laminates, which indicates the prediction of higher stiffness for the laminated beams by the program. The load factors, which are the ratios of experimental ultimate loads to the theoretical service loads (column [6]), decrease with increasing number of laminates; however, the values are significantly greater than unity which indicates the CFRP laminate restraining effect.

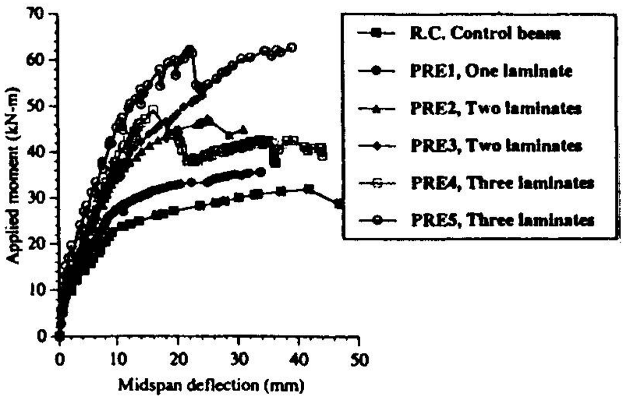

Fig. 4 Effect of CFRP laminates on deflection

The prediction of higher ultimate capacity by the program can be attributed to the assumption of perfect bond between the concrete and CFRP laminates. The CFRP laminate strains at ultimate load predicted by the program were significantly higher than the experimental values [5]; this indicates the higher tensile force developed by the laminates and higher ultimate capacity and the corresponding service load than those observed from the experiments. The control beam exhibited widely spaced cracks, whereas the beam with CFRP laminates showed cracks at relatively close spacings. All the beams with CFRP laminates failed by crushing of concrete with a significant increase in the flexural capacity. Longitudinal splitting of the CFRP laminate preceded debonding before complete failure of the beam. For a beam with two CFRP laminates (Fig. 5), although the theoretical curve exhibits a higher ultimate moment and higher stiffness, it compares well with the experimental curve. The strain-compatibility method seems to give accurate results, although more precise predictions could be made with better idealization of the bond between laminate and concrete and the failure criterion.

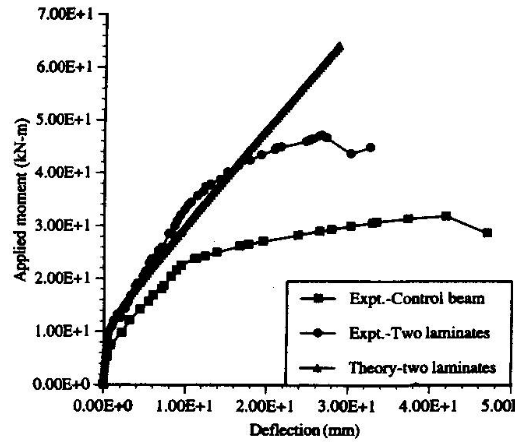

Fig. 5 Moment deflection behavior of control beam and beam with two CFRP laminates

Table 4 Experimental and theoretical loads

Beam number	Number of CFRP laminates	Exptl. ultimate load UL (kN) [1]	Factor of increase F_{UL} [2]	Theor. ultimate load (kN) [3]	[1] / [3] [4]	ACI 318 service load (kips) [5]	[1] / [5] [6]
S5-STL	-	59.793	1.000	53.632	1.115	26.721	2.238
S5-PRE1	1	66.603	1.114	97.225	0.685	49.798	1.338
S5-PRE2	2	88.422	1.479	120.356	0.735	62.044	1.425
S6-PRE3	2	97.923	1.638	140.817	0.695	72.875	1.344
S6-PRE4	3	91.967	1.538	163.059	0.564	84.650	1.086
S6-PRE5	3	116.214	1.944	163.059	0.713	84.650	1.373

4. CONCLUSIONS

The flexural strength of concrete beams prestressed with CFRP tendons could be predicted using conventional theory. The moment-deflection characteristics of the CFRP pretensioned beams show typical bilinear behavior. The failure mode indicates brittle behavior, which needs further study. The addition of bonded CFRP laminates to reinforced concrete beams results in a significant reduction in deflections and crack width with a substantial increase in the ultimate flexural capacity. The computer program based on ACI strain compatibility method gives a good estimate of the nominal moment strength and corresponding curvatures and deflections.

5. ACKNOWLEDGMENTS

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Reinforcement of Bridges Piers Foundations

Réparation des fondations de piles de ponts

Verstärkung von Brückenfundamenten unter Stützen

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SUMMARY

The authors propose a method to repair the disorganised masonry of a foundation pier by means of prestressing with a corrosion-proof steel lining in a closed circuit. The service life of the lining is expected to be more than fifty years.

RÉSUMÉ

Les auteurs proposent une méthode de réparation de la maçonnerie désorganisée d'une fondation de piles de pont, au moyen de précontrainte réalisée avec des barres d'acier inoxydable. La durée de service de la fondation réparée est estimée à plus de cinquante ans.

ZUSAMMENFASSUNG

Die Autoren schlagen eine Reparaturmethode für ein Stützenfundament mit Hilfe eines vorgespannten rostfreien Stahls vor. Die Lebensdauer nach der Reparatur soll mehr als 50 Jahre betragen.

1. INTRODUCTION

A latitudinal corridor of the Transsiberian main line (Glavsib) containing a railway and highway polygon of the Siberian region is examined. Here are included meridional branches also. A corridor length of main axe is about 4 500 km. A meridional branches length is about 2 000 km. Engineering and geographical conditions are unfavourable. The polygon is consisted of a north building-climatic area that includes about 30 % of Baikal high-seismic zone.
A polygon net crosses the Ob, Enisei, Lena and Amur rivers of a first and a second orders and passages also large and out-of-class bridges that erected from 1950 to 1990.
For the most part, a stable work of the road polygon depends on the bridges. Bridgeworks are classified as barriers of the transport network. The highways insure the Glavsib in difficulties.
A normalized service life of the bridges is limited by viability of the piers, as more durable structures. That is way, restoring and maintaining the total service life of the river piers it is an important problem of reliability and stability of the big bridges.
In this paper, term "pier" without a foundation is presented.

2. THE ENVIRONMENT OF WORK OF PIERS

It is distinguished an influence of environmental conditions on the foundations and the piers. Environment exerts influence on the foundations one or two time order more slowly, which is almost equally a bridge service life, as compared with one on the piers. Exceptions are river bed deformations. Therefore, a priori, the foundations are considered as with quasi-steady states, apart from pier states and then may be excluded from our discussion. It is an actual problem to retrieve a technology and materials to restore and support operation resources of the river bed piers of the big bridges.

3. TECHNICAL DECISIONS

The Department of Bridges and Constructions offers, as radical decision, a method to restore operation resources of the river bed piers on the basis of a lining of a corrosion-proof steel sheet (steel grade 15ХСНД) and filling in by a concrete with a crushed stone of fine-grained fraction or a cement mortar a clearance between the lining and a support body. However, technical and economical evaluations of the decision making are subject of another work.
As necessary and sufficient conditions of technical and economical evaluations of usage of the steel lining it is recommended to consider a operation reliability of one in during of half a normalized bridge service life. It is limited by resistance of the lining to a chemical corrosion.
There are no analogous samples in the world construction practice.
In The Department was designed and carried out the multiyear and multicriterion (modeling theory) experiments. It generalized the fullscale experiments of two river bridge piers of Yaya river by an inclement iced climate and winter air temperature under - 40 C:
- in the middle part of river (t. Anjero-Sudjensk, Kemerovo region)
- in the lower part of river (t. Bolshoye Dorokhovo, Tomsk region)

The experiment was carried out in natural conditions; an experimental object corresponds to an application; the time consists of 40 % and 16 % of the normalized service life without an extrapolation; a purity of the experiment it is undouptedly.
As a result of the experiment it is determined that the steel lining, in running water, may be in long-term usage without a corrosion-resisting coating or other applying a dressing. A corrosion-

resistance of the experimental lining is more stable than prognosed one by Norms [3]. An ice abrasion resistance of the experimental lining is more als ten times as large one of a prefabricated reinforced concrete and it is compared with a stone facing of strength volcanic rocks (basalt, granite). The service life of the steel lining (steel grade 15ХСНД) may be continued to 55-60 year.
Usually, to repair and reinforce the piers with disturbanced massif continuousness and external surface integrity it is necessary to disassemble a bad masonry, to restore it and inject a cement mortar in the massif in order to fill in cracks and voidnesses.
It is disadvantage to disassemble 40-50 % massif volume with breaking off a bridge transport traffic and removing a transport load from the piers to execute a construction works and, moreover, to expect possibilities of repeated deformations of a repaired masonry and abrasion working of external surfaces by drifting an ice and drawing river deposits (river drifts). An efficiency and the service life of aforesaid technical measures are not predicted and a cost it is compared with one of new piers.
It is a subject of investigation to restore pier maintenance resources by means of erecting the high-resistance to mechanical action and stands up to attack by chemical corrosion sheet steel lining on a pier perimeter for anyone massif forms and external surface dislocations.
Our investigation is realized by forming the steel lining, as a renewal construction (masonry into collar), by means of filling in a concrete in a tunneled lining clearance from wall to wall with the massif. The high-strength lining steel takes up all internal loadings; a steel resistance to an abrasive wear is more high als one of anyone concrete; an anticorrosive strength permits to predict in practice the service life with a sufficient authenticity The collar permits 100% usage of the pier masonry without to disassemble it for the expensive repair and construction works.
A quality and state-of-the-art of the collar-lining correspond to known technologies to erect metall welded tank-reservoirs of cylinder forms to store oil products. It is borrowed erecting a vertical walls of large-size prefabricated half-finished products without a bottom and roof, its assembling and welding on a construction site.
The collar-lining of closed contour squeezes the pier massif by means of the concrete of tunneled lining filling in, limits a crack-formation, stabilizes a massif internal stress, protects safety external surfaces from shocks and ice abrasive wears and drawing river deposits. A collar function is based on principle of the tunnel lining.
A design position of the collar-lining on the pier has been fixed by insert parts of a merchant-mill product (rolled steel) or a reinforcement. Filling in a tunneled lining clearance has been realized by a concrete pump and compacting a concrete by adjustable external and internal vibrators.
The bridge piers contain a masonry of prefabricated and monolithic concretes dislocating by crack-formations and also a protection coating of 50 % damage surface processed by an ice drifting action and drawing river deposits.
The lining of prefabricated panels had been assembled and welded by field erection welds.
Design position and tunneled lining clearance, filled in by a concrete with crushed stone small fractions, have been supplied by clamps.
Steel tunnel lining for a general overhaul of the river piers allows to restore completely design technical characteristics bringing the service life to normalized one and to exclude routine repairs.

4. CHARACTERISTIC PROPERTIES OF WORK OF THE COLLAR-LINING

It is important characteristic property of the collar-lining construction that it provides 100 % usage of prefabricated masonry, gives an opportunity of a bridge transport traffic without limitations and requires small quantities of building works on a construction site, material consumption capacities and costs.
A function of the steel lining of a support body depends on a complex stressed-deformation state. Three composite components interact in the collar-lining construction, Fig. 1:
1 - available pier massif;
2 - filling in tunneled lining;
3 - steel tunnel lining.
There are contact surfaces (1-2) and (2-3) between adjacent bodies. Interaction character of the adjacent bodies by means of contacts is ambiguous and depends on an internal stress state and a lining construction technology. Body masses of 1, 2 and 3 match up one for other as 90:8:2.
As a main factor, environmental temperature variations (air, water) determine a stress-deformed state of the piers and the lining. When no exothermic forces exist in the massif then it is recommended to limit by a temperature action only.
Two nature factors that positioning (adapted) a design technology of the river piers lining are following:
- ice regime and hydrological regime of river;
- outside-air temperature [4].
The level regime is characterized by year graph of construction levels of probability of difference of height from 10 % to 50 %. It is statistical variable behaviour and it may be different from real one. In Summer-time, the lining is submerged and therefore it is in stable and softening temperature regimes (as compared with air environment). In Winter time, the lining is exposed to subzero outside-air temperature that is considered as unfavourable regime.
Tangential stresses of the lining squeeze the pier massif and form radial stresses of the masonry; it both are interlocated with known formulae of momentless theory of thin-walled shells. The steel lining is not to expose to the force ice actions before it filling in the tunneled lining in order to save a design geometrical state, form and strength of one.

5. CONCLUSION

A technology and work scheduled network are to limit by environmental conditions. That is to be on schedule at four winter months. Co-working 1, 2, 3 components begins from a moment of interlocking under where that is supposed to fill in the tunneled lining clearance by the concrete to a design level with taking a strength. This concluding operation is to realize with positive temperature of the massif [1].
Presented method for Siberian region may be recommended as a reliable modeling the problems of the bridges of Russian road network.

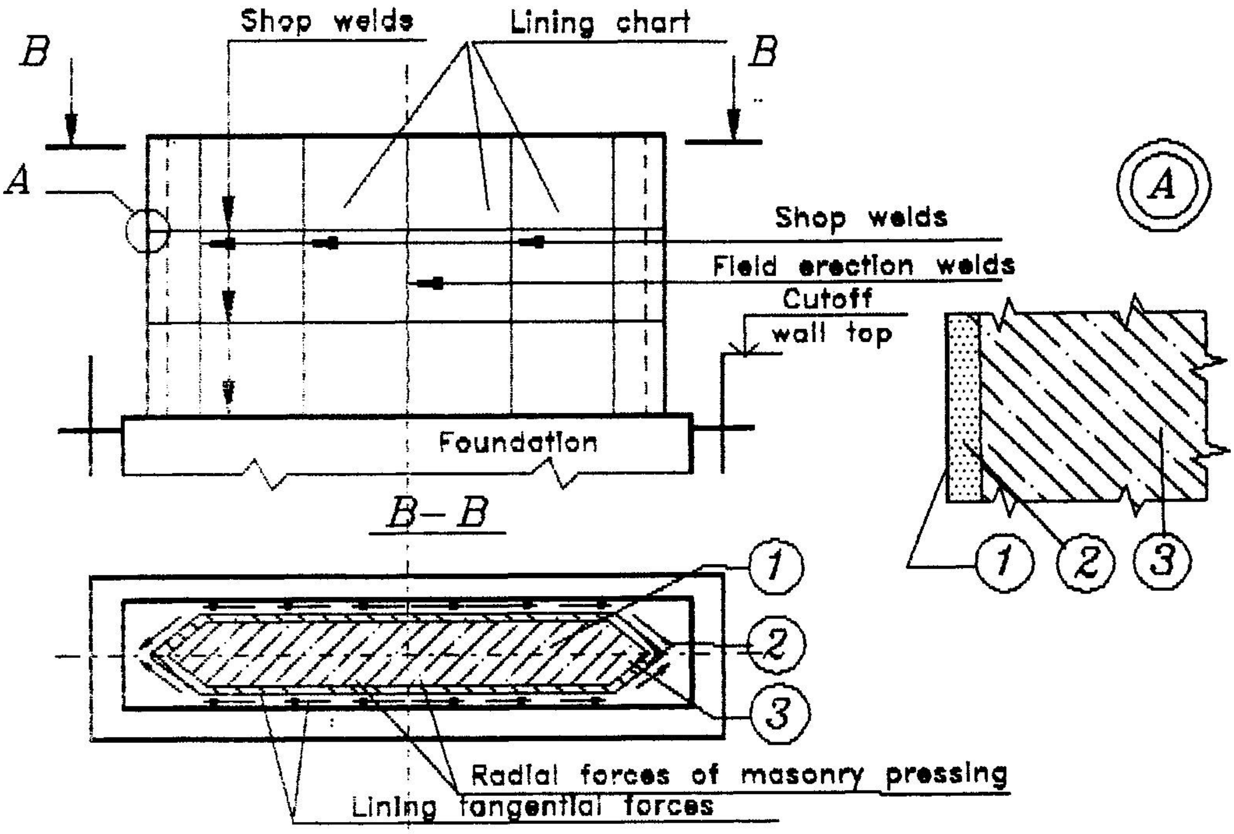

Fig.1 Basic circuit arrangement of steel lining piers

6. ACKNOWLEDGEMENT

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Strengthening of Gravity Dams Using Post-Tensioned Anchors

Renforcement des barrages poids à l'aide d'ancrages précontraints

Verstärkung der Schwergewichts-Dämme mit Hilfe von nachgespannten Ankern

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SUMMARY

Many old gravity dams are in need of rehabilitation as a result of ageing, deterioration, deficiencies in design and construction, and more stringent safety standards. Post-tensioned anchors are the most practical and cost-effective method for strengthening concrete gravity dams subjected to direct tension, sliding, overturning, and seismic loading. This paper reviews 27 case histories of concrete gravity dams strengthened by cement grouted multi-strand post-tensioned anchors. The general design of post-tensioned anchors for strengthening gravity dams is outlined.

RÉSUMÉ

De nombreux barrages poids doivent être consolidés suite au vieillissement, à la détérioration, aux déficiences dans la conception et la construction, et aux normes de sécurité plus sévères. La post-tension par ancrages est la méthode de renforcement la plus pratique et la moins onéreuse pour les barrages poids en béton soumis à une tension directe, glissement, renversement ou à un séisme. Cet article décrit 27 cas de barrages poids en béton renforcés à l'aide d'ancrages précontraints composés de câbles injectés au coulis de ciment. La conception générale des ancrages précontraints pour le renforcement des barrages est présentée.

ZUSAMMENFASSUNG

Viele der alten Schwergewichtsdämme sind renovationsbedürftig aufgrund der Alterung, Abnutzung, Mangel in Plan und Konstruktion, sowie strenger werdender Sicherheitsnormen. Benutzung von nachgespannten Ankern ist die praktischste und kostengünstigste Methode für die Verstärkung von Beton-Schwergewichts-Dämmen, welche der direkten Spannung, Rutsch- und Umkippgefahr, und Erdbebenlasten ausgesetzt sind. In diesem Artikel werden 27 Fälle dargelegt, in denen Beton-Schwergewichts-Dämme mit vorgespannten Ankern verstärkt worden sind. Allgemeine Entwürfe und Bauarten vorgespannter Anker für die Verstärkung von Schwergewichtsdämmen werden beschrieben.

1. INTRODUCTION

Many old gravity dams are in need of rehabilitation as a result of aging, deterioration, deficiencies in design and construction, and more stringent safety standards. Post-tensioned anchors are the most practical and cost effective method for strengthening gravity dams. The application of the post-tensioning force increases the frictional resistance at the concrete/rock interface and the moment against overturning, and reduces uplift pressures underneath the dam and tensile stresses on the upstream face. Post-tensioning a gravity dam can be an especially suitable remedial under the following conditions: 1) where the lift joins were inadequately treated during construction and so were weak in tension and shear; 2) when drawing down reservoir level is restricted due to the intolerable economic loss; 3) preservation of historical structures is required; 4) the space on the downstream face is limited for adding concrete mass; and 5) adding concrete is objectionable as in seismically active areas. The post-tensioning technique requires minimum demolition, has only minor impact on the structure, and is relatively inexpensive by using a small number of anchors.

Table 1 shows some gravity dams strengthened using cement grouted post-tensioned anchors in the last 20 years for the various reasons: 1) upgrading to the Probable Maximum Flood (PMF) (11 dams); 2) raising dam height (5 dams); 3) upgrading to the Maximum Credit Earthquake (MCE) (4 dams); 4) deficiencies in design (2 dams); 5) stabilizing concrete cracks (2 dams); and 6) others (4 dams). A dam may have more than one reason for rehabilitation. Post-tensioned anchors are most commonly used to meet the new design criteria relating to the updated PMF and MCE that expect much greater loads acting on a dam.

Steel strands are widely used for post-tensioned anchors due to their high strength, ease of tendon transport and storage, and considerable length. Recently, a comprehensive research program is being conducted on the strengthening and rehabilitation of gravity dams using post-tensioning techniques at the Université de Sherbrooke. This includes: 1) using various types of tendon strands such as conventional strands, epoxy-coated strands, and fibre reinforced plastic (FRP) strands; 2) developing instrumentation systems such as using vibrating wire gauges and optical fibre sensors; 3) investigating the performance of post-tensioned anchors in dam rehabilitation. On reviewing 27 case histories, anchor dimensions used in dam rehabilitation are summarized. The general design of cement grouted multi-strand post-tensioned anchors for strengthening gravity dams is highlighted.

2. DESIGN METHODS

Each anchor installed in a dam should be regarded as permanent. Therefore, corrosion protection is a vital and integrated part of anchor design and construction. The anchor design consists of mainly the overall stability of the strengthened dam, fixed anchor dimensions, and anchor embedment depth.

2.1 Overall Stability of a Strengthened Gravity Dam

The stability analysis of an old gravity dam strengthened with post-tensioned anchors can be accessed in the manner as for a normal gravity dam, with the addition of an extra stabilizing force from the anchors. Post-tensioned anchors for dam rehabilitation should be installed as near as practicable to the upstream face of the dam to provide the greatest stabilizing moment. Basically, a gravity dam should be safe against overturning at its toe and against sliding along the concrete-rock interface or at any weak plane within the foundation. The loads acting on a dam consist of the hydrostatic loads (H_1, H_5), uplift load (V_2), inertia loads of water (H_6) and concrete (H_7, V_3) due to seismic activity, and excess loads due to silt (H_2), ice (H_3), and impact of waves (H_4), as shown in Fig. 1. The post-tensioned anchor (F) will counteract, together with the dead concrete weight (V_1), against overturning and sliding of the gravity dam. Stability against sliding along the concrete-rock interface and overturning about its toe is expressed as:

Sliding: $$SF = \frac{Ac + (V_t + F)f}{H_t} \quad [1]$$

Overturning: $$\frac{\Sigma MW + MF}{\Sigma MV} \geq F_s \quad [2]$$

where SF = shear friction factor; A = contact area of the base; c = unit shear resistance at the base; H_t = total horizontal load; V_t = total vertical load; f = coefficient of internal friction at the base; F_s = designated factor of safety; ΣMW = summation of anti-clockwise moments of forces V_1, H_5 about point O; ΣMV = summation of clockwise moments of forces H_1, H_2, H_3, H_4, H_6, H_7, V_2, V_3, about point O; MF = moment of force F about point O.

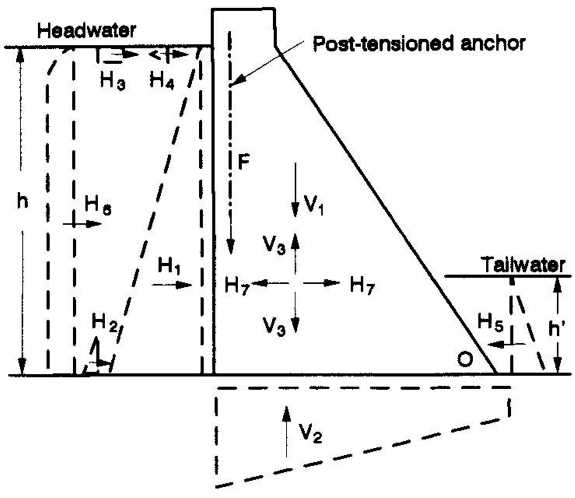

Fig. 1 Loads acting on the section of a gravity dam

The loads acting on a dam are generally categorized as usual, unusual and extreme. Post-tensioned anchors are usually used for the unusual and extreme loading situations that are based on the usual load combination with the PMF level, instead of the normal design reservoir level, and the usual load combination acting together with the MCE, respectively. The recommended factor of safety should be 2.0 and 1.3 for the unusual and extreme load combinations [18]. The force for individual anchor can be obtained from the above equations and is based on the dimension of dam crest, anchor spacing requirement and availability of anchor capacity. Most codes require a design working load of $0.5f_{pu}$ (ultimate load of anchor tendon) with a factor of safety of 2 for permanent anchors. However, an analysis of anchors in Table 1 shows that a higher design working load of $0.6f_{pu}$ is commonly used for permanent anchors. An even higher working load of $0.65f_{pu}$ has been used for several dams in Australia where the anchors are monitorable at any time. Currently, the anchors installed at Burrinjuck Dam in Australia are the largest ones in the world, which consist of 63 No. 15.2 mm strands with a maximum capacity of 16500 kN and an overall length of 128 m [9].

2.2 Fixed Anchor Dimensions

The design approach to calculate the fixed anchor dimensions has been based on an assumed uniform bond stress between the grout and the rock [19]. Generally, the ultimate bond stress depends on the shear strength of the rock, the grout strength, and the method of construction. In UK practice [21], it is required that the ultimate bond strength used in the design should not exceed the minimum shear strength for rock with uniaxial compressive strength (UCS) less than 7 N/mm^2, and can be taken as 10% of the UCS of the rock for strong rock but should not exceed 4.0 N/mm^2 for UCS value exceeding 40 N/mm^2. Cement grouts with water-cement rations of 0.40 to 0.45 are commonly used in practice and the rock/grout bond values recommended for design can be found elsewhere [18, 19, 21].

The fixed anchor length should be neither less than 3 m nor greater than 10 m [20]. For a short bond length (less than 3 m), sudden changes in rock quality along the fixed anchor zone as well as any constructional errors may induce a significant decrease in pull-out capacity. On the other hand, fixed anchor lengths of 10 to 12 m are considered useful only in particularly weak rocks such as mudstones and shales [21]. An analysis of anchor bond length in Table 1 shows that the bond lengths are generally in the range of 5.5 to 10 m. Anchors with a longer bond length up to 16 m have also been seen in practice. One of the extreme example is at Lake Lynn Dam, where a bond length of 18 to 40 m was used in clayey siltstone [6].

Fixed anchor diameters depend mainly on the size of tendon, corrosion protection requirements, size of drilling bit, and ground conditions. The practical hole sizes adopted in dam rehabilitation generally range from 114 to 330 mm (4.5 to 13 inch). Anchor spacing affects the behaviour of individual anchors. In dam rehabilitation, the minimum spacing is governed by practical requirements such as hole tolerances to avoid deep anchor crossing in the drilling line, anchor capacity, applied load per meter length of the dam, and rock mass discontinuity spacing. An analysis of anchor spacing in Table 1 shows that anchor spacing is generally in the range of 0.9 to 5.5 m with a more common range of 1.5 to 2.5 m.

2.3 Anchor Embedment Depth

The load of an anchor is resisted by the weight of a mobilized rock mass beneath the dam. Two alternative methods are used to determine the size of the mobilized rock mass. The pull-out cone method assumes that an inverted cone of rock is pulled out of the rock mass for a single anchor at failure. The weight of rock mass mobilized is calculated using the volume of the cone and the submerged weight of rock situated below water table. The wedge method assumes that the foundation rock is tied to the dam to against overturning with the dam. Case study has shown that the assumed cone method is more common than the wedge method in practice.

Current design approaches ensure that anchors are embedded deep enough so that failure of the tendon or at the grout-rock interface occurs before the rock mass being pulled-out. Over the years, the choice of apex cone position has been varied from the base of the anchor, the middle of the fixed anchor to the top of the fixed anchor. In dam rehabilitation, apex cone positions are usually chosen at the top or at the middle of the fixed anchor, and are often dictated by the surrounding ground conditions. It suggests that the apex cone position be taken at the middle of the fixed anchor when considering load transfer by bond [19, 20].

Generally, the included angle of inverted cone is assumed to be 90° in sound homogeneous rock with the apex at the bottom of the fixed anchor, and 60° in weak or highly jointed rock masses with the apex at the middle or at the top of the fixed anchor. When groups of closely spaced anchors have their fixed anchor zones located in the same rock horizon and the rock mass is horizontally bedded, a laminated failure could occur. As a result, it is necessary to incline the fixed anchor further apart or stagger the fixed anchors at different depths in order to reduce the intensity of stress on any plane.

3. SUMMARY

Based on reviewing 27 case histories, anchor dimensions used in dam rehabilitation are summarized. The general design of post-tensioned anchors for strengthening gravity dams is briefly described.

Country-Dame	Dam height (m)	Anchor no.	Strand tendon (mm)	Design capacity T_c (kN)	Design load (%T_c)	Bond length (m)	Anchor spacing (m)	Hole size (mm)	Anchor length (m)	Ref.
USA-Minidoka	*	7	44/15.2	11458	60	7.6	5.5	254	35	[1]
USA-Morgan Falls	16.8	100	31/12.7	6461	55	9.0	*	165	*	[2]
USA-Rainbow	8	6	Strand	4447	60	*	4.5	152	22	[3]
USA-Ryan	27	31	45/12.7	9576	60	16.2	0.9	203	49	[3]
USA-Crescent	12.2	151	12/15.2	3600	52	*	*	*	31	[4]
USA-Conowingo	30.4	537	20/15.2	5300	60	8.4	2.8	190	61	[5]
USA-Lake Lynn	38.1	75	58/15.2	15118	60	39.9	2.0	280	92	[6]
USA-Shepaug	40	97	53/15.2	13789	59	14.9	1.8	254	59	[7]
USA-Libby	128	100	16/12.7	2936	60	6.1	*	127	46	[8]
Australia-Cataract	54	*	55/15.2	13750	60	*	2.5	310	84	[9]
Australia-Burrinjuck	79	161	63/15.2	16500	65	*	1.5	310	128	[9]
Australia-Nepean	76	*	63/15.2	16500	57	*	2.0	314	119	[9]
Australia-Goulburnweir	15	*	27/15.2	6750	*	*	1.8	200	*	[9]
Australia-Hume	51	*	55/15.2	13750	65	*	*	312	85	[9]
Australia-Warragamba	137	*	63/15.2	16500	60	*	2.5	310	112	[9]
Australia-Maroondah	47	*	52/15.2	13670	*	*	3.6	305	*	[9]
Australia-Bickley	13	*	12/15.2	3150	*	*	1.2	165	*	[9]
Australia-Manly	19	46	24/15.2	6000	60	*	5.0	215	43	[9]
Iran-Sefid Rud	106	*	54/15.2	14000	54	*	*	*	40	[10]
Morroco-L. Taberkoust	57	54	49/15.2	12625	53	*	*	*	115	[10]
Australia-Tenterfield	15	>44	12/12.5	2508	62	5.5	1.8	114	24	[11]
Germany-Eder	47	104	34/15.2	8800	52	10.0	2.3	273	75	[12]
UK-Mullardoch	50	26	37/15.2	11100	42	5.4	1.5	330	55	[13]
Brazil-Rasgao	22	80	12/12.7	2232	45	*	1.0	*	*	[14]
Parkistan-Tarbela	148	576	16/15.2	4170	60	6.0	*	165	38	[15]
Uganda-Owen Falls	*	115	12/15.2	3180	62	8.0	*	131	36	[16]
Zimbabwe-Sebakwe	38.8	80	38/15.2	9905	60	8.0	*	175	70	[17]

Note: * = data not available

Table 1 Concrete gravity dams strengthened using multi-strand post-tensioned anchors

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