French Experience in Prestressed Structures Repair

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1. Introduction

1.1 History of prestressing in France

On France's national roads, only 19% of bridges are made of prestressed concrete, but they account for 48% of the deck surfaces! In fact although Eugène Freyssinet filed his patent on prestressed concrete back in 1928, it was only after the second world war that this technique really began to develop and particularly since 1970.

1.2 History of prestressed bridge repairs

Like every other type of bridge, prestressed concrete bridges have shown specific defects. There are many reasons for these defects: inadequacies in calculations of some structures built before 1975, corrosion of tendons with insufficient protection, and so on... Furthermore, there are more and more socio-economic demands such as widening or live load increases.
In France, the first significant operations on prestressed concrete bridges took place in the early seventies. A recent inventory shows that about fifty large prestressed concrete bridges have been reinforced or repaired, in most cases with additional external prestressing.
Most of the defects encountered in France, principally tendon corrosion or structural defects, concern deck girder bridges and box girders using staged construction.

2. Repair techniques

2.1 The different techniques

Depending on the origins of the defects, various repairing or strengthening techniques can be used. We will limit the scope of this paper to the repair technique by the addition of force, currently widely used in France [1].

2.2 Additional prestressing

Additional prestressing, generally external to the concrete, is the best known repair and/or strengthening method for bridges and other prestressed concrete structures. It can be used on bridges cracked through errors in design or construction, on bridges with highly-corroded prestressing reinforcements and even on bridges in good condition but whose load-carrying capacity must be increased.
In France, many structures have been strengthened by this technique, in which structural detailing and efficiency checks have made great progress since the first repairs in the seventies.
The additional prestressing may be longitudinal, transversal or vertical.

3. The lessons drawn

France has 25 years experience of repairing prestressed structures in order to extend their life span or to update their performance for contemporary needs.

3.1 Lessons drawn for repairs

3.1.1 Methodology

It is essential to use a strictness methodology for expert appraisals and repairs [2], which must respect the following three stages:

- preliminary diagnosis ;
- performance of tests and studies specified in the investigation programme;
- synthesis and finalisation of a diagnosis to explain the causes of observed defects and quantify the resulting weak points in the load-carrying capacity. Experience has shown that repairs made without this prior analysis have more often than not been failures.

3.1.2 Repairs by additional prestressing

For repair work by additional prestressing, specific requirements must be met, such as stop trafic during the hardening phase of the additional concrete or do radiographies before drilling, etc.

3.2 Lessons drawn for new structures

We can also draw lessons from this experience to design new structures.

3.2.1. Basic rules to limit the ageing of structures

For example :

- Any change in usage requirements of a bridge (future widening, increase in load-carrying capacity) must have been already taken into account in the design.
- All monitoring and maintenance operations on all parts of the structure must be simple, easy to perform and must not adversely affect durability.
- Systematically carrying design regulations to their limits results in a build-up of reinforcements and tendons incompatible with good installation of the concrete, proper concrete cover and positioning of the reinforcements.
- etc.

3.2.2. Technological changes and durability

The durability problems that sometimes occur with bonded post-tensioning (grouting with cement) may induce engineers to consider new methods of protecting prestressing.
But experience has shown that opting for such solutions is sometime like jumping out of the frying pan into the fire, due for example to the incompatibility that may exist between some cements and some additives when preparing a grout mix with a controlled grouting time, that make cause settlement phenomena and grout setting defects in the prestressing ducts.

4. Conclusion

In order to improve the Quality of structures, it is important to draw lessons from experience and build it up. This analysis made in France has resulted in a considerable reduction in defects, even though further progress remains to be made.
Requirements in terms of durability must remain the absolute priority and it is particularly important for future European technical approvals for prestressing systems to take them into account.

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New Life by Post-Tensioning: Rehabilitation of Two Box Girder Bridges

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Summary

In this paper, a rehabilitation project of two box girder bridges is described. The aim of the project was to increase the load capacity of the bridges and, simultaneously, to ensure their durability by filling the cracks and repairing defects. The load capacity was increased by strengthening the bridges by external tendons, the criteria for choosing the tendon forces being to prevent further cracking and growth of the midspan deflection. The construction is shortly described, and the costs of the rehabilitation is compared to the costs of replacing the old bridges by new ones. The chosen solution is found more economical than building new bridges.

Keywords: post-tensioning; external tendons; strengthening; rehabilitation; concrete; bridges

1. Introduction

On the Finnish highway road net, there are about 150 reinforced concrete box girder bridges, which have similar features. These bridges were constructed between 1950 and 1970, at the time when prestressing technology was not widely used in Finland. Murhasaari and Puodinkoski Bridges, which are both of this type, are now chosen for rehabilitation. They are located on the highways 11 and 4, respectively, on parts which belong to the route net for heavy transport in Finland. The both bridges did not meet the requirements set on highway bridges [2,3]. The rehabilitation and improvement of the bridges was accomplished by strengthening them by external post-tensioning and repairing other defects. Post-tensioning was chosen as the strengthening method because of its positive influence to the cracking and deflection behaviour of the bridge superstructure.

2. The bridges before rehabilitation and improvement

The both bridges are three-span continuous reinforced concrete box girder bridges, the height of the girder varying along the span. Originally, a 140 kN axle together with a 30 m long distributed load of 4 kN/m^2 was applied as the design load. On this rehabilitation project, as design load was applied the actual bridge design load [4], which consists of an three-axle vehicle of 630 kN on max. two lanes and a distributed load $q = 3$ kN/m^2. Applying these design load results to a bearing capacity, which covers also the over-heavy loadings. The condition of both bridges was rather similar. Both had visually disturbing deflections and numerous cracks. The conrete surfaces were in relatively good condition. The carbonation depth was found to be only some millimeters in both cases.

3. Rehabilitation and improvement

By choosing the amount of prestress, two different criteria were applied. For Murhasaari Bridge, the amount of prestress and the tendon layout were chosen such way that the bending moment M_P due to prestress force P_∞ equals as accurately as possible the bending moment M_g due to permanent loads. For Puodinkoski Bridge, a criterion for concrete tensile stresses σ_c, which may not exceed the cracking stress σ_r, is applied. Under a quasi-permanent load combination M_g+M_P+0,3M_q, the whole cross-section is in both cases under compression. At ultimate, the following assumptions were made: 1) the rebars yield; 2) no additional strain of tendon is considered; 3) the compressive stress P_∞/A_c is considered. The permanent midspan deflection of the both bridges was not possible to be eliminated by the chosen prestressing.

Tendons consisting of 12 strands with 0,6" diameter (140 mm^2) was used in both cases. The strands are placed in a plastic duct, which will be injected by cement grout after tensioning. The anchors located at both ends, are of the types normally used for internal, bonded tendons. The works are planned so that at least one lane is on traffic all the time. The works were relatively simple and they were possibly to realise under ongoing traffic.

4. Discussion

The carbonation will probably not reach the level of the reinforcement during the following 70 years. The tendon protection is designed according the same criteria as in new constructions. According to the authors' knowledge, the 70 years service life should be possible.

The estimated costs of the rehabilitation of the bridges were 500 USD/m^2, which also could be realised at Puodinkoski Bridge. With the assumed service life of 70 years, the annual costs are 19,3 USD/m^2. Replacing the old bridges by new ones would cause costs of 1100 USD/m^2, including demolishing works of the old constructions but excluding the costs of a temporary road connection. Using a service life of 100 years for the new construction, the annual costs will be 37,4 USD/m^2.

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Modern Technology to Extend the Life Span of Bridge Structures

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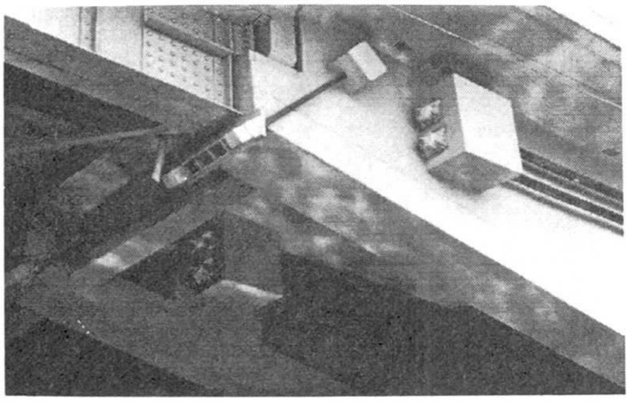
Summary

This paper will present through few selected examples in Vietnam, Mexico and Romania how this upgrading and strengthening is achieved. Some specific methods and technologies have been developed : sometimes we replace poor quality or defective material by better quality material, or we attach an additional load-bearing material, or we may redistribute the loading actions through imposed deformation of the structural system. In any case, each project is unique and it appears that strengthening and repairing a structure is really an art.

1. The Tan Thuan bridge repair and strengthening in Vietnam

The Tan Thuan bridge is located north-east of Ho Chi Minh city. The total bridge length is 239 m and its total width is 12.50 m. It carries a 8 m roadway and two 1.26 m sidewalks.

The aim of the strengthening work was to enable heavy goods vehicles (container trucks) to cross both ways. The reinforcement techniques mainly involved the use of external prestressing both for concrete and steel spans.



External prestressing strengthening

2. Cerna bridge rehabilitation in Romania

This 6-span bridge, 10.60 m wide, was built in 1968 using balanced cantilever construction. The spans are distributed as follows :

26.60 + 4 x 54 + 26.60.

Very soon, large deflections appeared at mid-span. In 1976 an additional surfacing was done to compensate the deflections. In 1979 hinges were replaced.

The statical scheme has been transformed : no more hinges but a continuous deck. This requested, of course, to create a physical separation between piers and deck. Consequently the piers were cut, the continuous bridge deck lifted and sliding bearings placed in between. Diaphragms were prestressed with external cables and bars.

Rehabilitation of Ancient Steel Structures

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Abstract

The first iron bridge in Europe was constructed in Coalbroockdale (England) in 1776. The span of its structural arches was 32 m. Twenty years after the first bridge construction, another one of similar structure was built in the town Lazany (Laasan) that now belongs to Poland and those days was in Germany. Lazany is not far from Wroclaw. Arches of that bridge was of 11.2 m span. Structural iron members have begun to be used in industrial buildings as early as in the first half of 19^{th} century. It was common to construct cast iron columns supporting suspended slabs. Sometimes, also cast iron beams have been used into the slab structure. Many of bridges, buildings and tanks constructed in that time, have been using, till now.

Beginning from the half of the 19^{th} century, iron materials like puddled steel, cast steel and cast iron have been used more and more in industrial structures. Due to the simple production technology of iron materials, their chemical heterogeneity was high. Also, the dispersion of their mechanical properties was big. Puddled steel and cast steel have low impact resistance and welding of those materials was impossible. Bolted and riveted connections were commonly used.

On the turn of the 19^{th} century puddled steel was replaced by cast steel, manufactured in Bessemer converters (1855), Thomas converters (1880) and Siemens-Martin furnaces (1864). The new technologies secured high quality of cast steel and its high homogeneity. Technical properties of the historical iron/steel materials are presented in the paper.

The paper presents case studies of several iron/steel structures, constructed in the 19^{th} century and beginning of the 20^{th} century, that are still in use. All of the presented structures were renovated, strengthened or rebuild. The examples of interesting rehabilitation processes of old iron structures are presented in the paper. The rehabilitation processes are unique for each structure. They were designed with a big precise. Some interesting case studies are presented in the paper.

All cases show mainly strengthening of floor slabs in old buildings. For example, there is a steel structure of the building constructed in 1834. The main floor of the beams were strengthened by supporting them at mid span by means of the steel tension members 2ϕ30, running longitudinally. Partial interaction between the reinforced concrete slab and the floor beams (composite structure)

has been taken into account. Composite action together with strengthening by tension members allowed for the service load p = 2500 N/m^2. Load capacity of cast iron columns of tube section was suitable for the new level of loads and was not increased.

In other, cast steel building constructed in 1897, increase of the load capacity of floor beams was needed. Overloaded I 130 beams have been unweighted by additional system of steel beams supporting of I 130 beams at their mid spans. Additional I 240 beams were supported on new transverse beams fixed to existing main floor beams (2 I 280) near the columns. In this way, the bending moment in main beams (2 I 280) has been increased of small value, while the beams have been not overloaded. New beams were connected with the old ones by bolts. There was no need to strengthen of the cast steel columns, supporting the floor and the roof.

Unique, multistory shopping center steel building was constructed in Wroclaw, in 1928. Ten years ago, service loads had to have been increased up to the level 5000–6000 N/m^2. In such a case, it was necessary to strengthen of the main beams (I 600) and secondary beams (I 220, I 360, I 380). Only the steel built-up columns with brick walling, had sufficient margin of capacity. In order to strengthen of the floors, the additional RC slab has been constructed. The slab has been connected with floor beams I220, I 360 and I 380 in such a way that composite action was developed. The increased weight of the floor has been compensated by considerable increase of load capacity of floor beams. Main floor beams were supported as simple beams. Increase of load capacity of that beams have been developed by making spring connections between beams and columns. Angles between I 600 beam flanges and columns gave the spring action. Moreover, special reinforcing of the additional RC slab, near the supports, gave also the effect of spring action.

Because of both, historical value and good serviceability, the steel structures constructed in the 19th and beginning of the 20th century should be strengthened and renovated. Steel products manufactured in that time have low strength, heterogeneous chemical composition and usually they are not suitable for welding. Strengthening and renovating of these structures require unconventional technologies. The technologies should take into account properties of historical steel and also historical value of the structure. Renovation done in a proper way can considerably elongate technical live of old structures. What's more, properly strengthened structure can be useful for loads of modern production processes.

Learning from Canadian Experience with Concrete Building Structures

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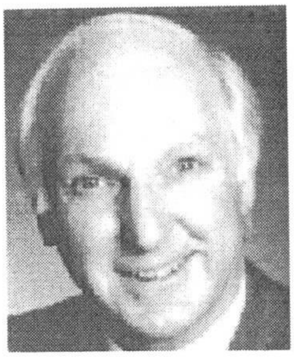

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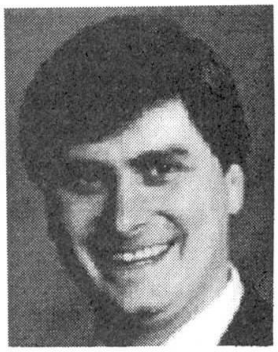

Summary

The paper is a sequel to a presentation to IABSE by Halsall in 1987 in Lisbon. The paper discusses types and extents of problems found in exposed parking structures and in enclosed buildings which are not exposed to de-icing salts. Structures investigated include those reinforced with plain and epoxy-coated bars and with various types of unbonded post-tensioning systems. There is discussion of methods of investigation, analysis and repair. The design of a new 4,000 car parking structure incorporating lessons learned is described.

Keywords: corrosion, epoxy-coating, unbonded tendons, investigations, repair, parking structure design.

1. Introduction

This paper reviews some of our experience in the investigation and repair of concrete structures since 1987. It will discuss the types and frequencies of the most common problems, and some recently developed techniques of investigation. In the last 10 years, we have been engaged in the investigation of over 500 buildings. About 300 involved damage due to corrosion of bar reinforcement, and 21 have involved stressed tendons. Due to the severe winters, and widespread use of de-icing salt on Canadian roads, parking structures often experience extreme temperature ranges and exposure to chloride contaminated water. Consequently, parking structures are found to develop problems more widely than any other category of building.

2. Parking Structures

Since 1987 Halsall has investigated, and usually repaired , over 3 million m5 of parking structures. The paper will discuss some of our experiences concerning:

- methods of diagnosis
- methods of repair
- some unsatisfactory performance of previous attempts at repair

- design and detailing of parking structures in the light of these experiences, illustrated particularly in the example of a 4000 car parking structure, partly below ground and partly above ground, using beams and slabs post-tensioned with unbounded tendons.

3. Other Post-Tensioned Buildings

Several failures have been investigated of P/T tendons in enclosed buildings not exposed to de-icing salts.

The paper will discuss some diagnostic methods, some remediation methods, including tendon replacement and an acoustic monitoring procedure to identify wire failures in real time.

4. General

The importance will be stressed of understanding the whole structure, from both structural analysis and materials point of view. There will be some discussion of the problem of how to ensure that stress distribution in a repaired structure does not deviate unsafely, from that intended in the original design.

Enhancing the Fatigue Life of Rio-Niterói Bridge's Orthotropic Steel Deck

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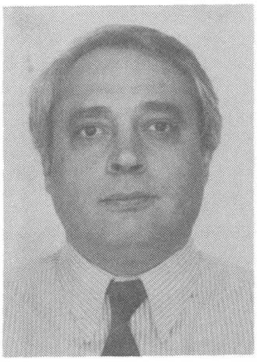

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Abstract

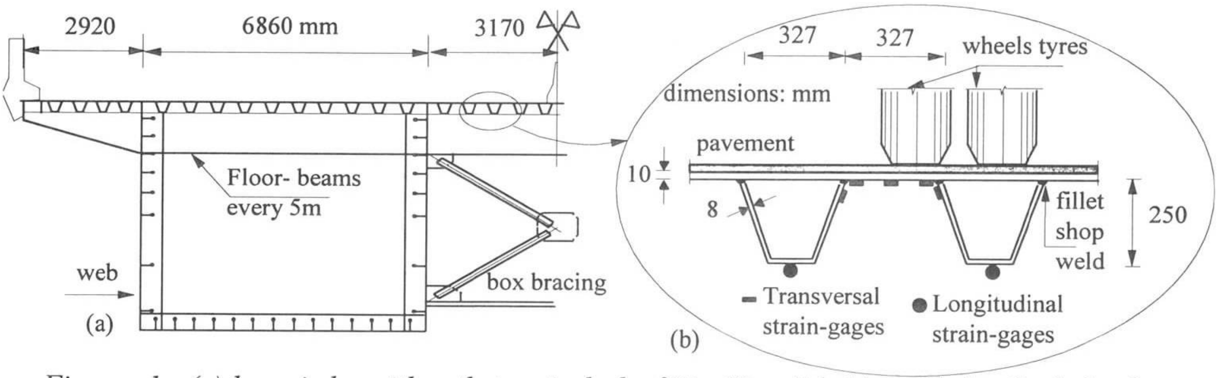
The thin-walled orthotropic deck of Rio-Niterói bridge, shown in Fig.1, has been under stochastic traffic loading frequently damaged by cracks in the fatigue-prone welded joints. Traffic on this bridge, brought into service in 1974, has raised to figures well beyond the initial estimates reaching now over one hundred thousand vehicles per day; almost 20% of these being heavy vehicles among which an increasing number of trucks with 3 and more axles.

To unveil the main causes of occurrence of observed cracks, it was carried out by a team of researchers and engineers with the COPPE's laboratory of structures a series experimental strain measurement campaigns: *in situ* as well as on a prototype scale model of the steel deck [1,2], i.e. a model in 1:1 geometric scale factor, shown in Photo 1. Refined numerical results from parametric studies, obtained with experimentally calibrated finite element models, were then used to better understanding the static and dynamic behaviours and sensitivity of this structure under heavy vehicle loading.

One important advantage of this slender structure is its low weight combined with great longitudinal bending stiffness. On the other hand, low transversal stiffness (see Fig.2) also brings higher sensitivity of local stresses to a series of relevant random and interdependent factors which appear in both static and dynamic problems of contact pressures resulting from the interaction between the vehicles' pneumatic tyres, the flexible asphaltic pavement and the slender steel orthotropic deck. Random stresses variations are then sensitive to all these concurrent factors and may cause precocious fatigue cracks in welded details, due to a complex and evolutive cycle of causes and effects. An attenuating effect is brought in by comparisons between today's and past measurements taken around 20 years ago which show that traffic loading is less adverse today than it was in the past, as far as localized stresses ranges and consequent fatigue life are concerned. First reported cracks occurred for a service time interval which could be closely predicted by taking the stresses histograms from past measurements and applying the Miner-Palgren rule and appropriate S-N curves for calculating cumulative damage.

With a better insight into the focused problems new solutions could be envisaged to attenuate the degree of sensitivity to so many random factors as well as to enhance the ulterior fatigue life of the slender steel deck and the service life of the pavement. From the obtained results it becomes clear that the very slender orthotropic steel deck lacks transversal bending stiffness and proper damping in all multiple and clustered frequencies vibration modes that leads to precocious fatigue of welded joints and of the adhesive layer of the asphaltic pavement on the steel deck.

A rational solution which fulfills lacking properties has been found by transforming the deck into a sandwich structure composed by a RC slab on top of a layer of visco-elastic material adhered to the steel plate. The performance of this composite structure, in which the RC slab may become itself the proper pavement was assessed with experimental tests performed on the prototype scale model. Redistribution of stresses resulting from the composite stiffness properties led to a substantial reduction of values in both longitudinal and transverse bending stresses. Visco-damping properties of the sandwich structure, with modal damping factors ranging from 3% to 7%, resulted similar to that displayed by the steel structure with a flexible asphaltic pavement having weight and thickness equivalent to the RC slab.



Figures 1 - (a) box girder with orthotropic deck of Rio-Niterói bridge. (b) detail of slenders trapezoidal ribs and deck plate, also showing typical instrumentation with strain-gages.

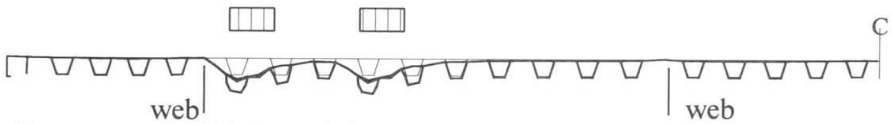


Figure 2 - Cross section of deformed deck under wheels loading (3D FEM model).

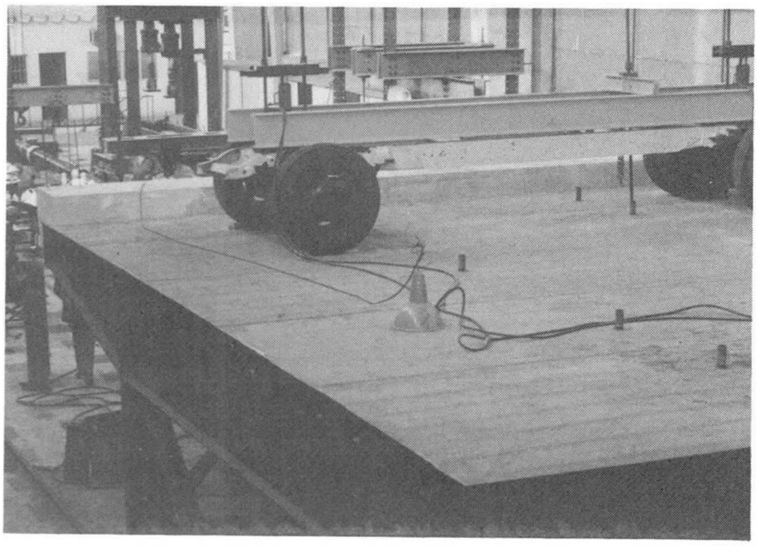


Photo 1 – View of the prototype scale model of the deck at COPPE's Laboratory of Strucutures.

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Bridge Inspection and Assessment

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Summary

The methodology and the main aspects of bridge inspection and assessment for concrete and steel bridges are presented in a synthetic form illustrated with the author's experience. Visual inspection and interpretation of test results are discussed. Two examples are presented: a reinforced concrete 35 years old arch bridge and a steel cable stayed 20 years old bridge.

Keywords: Bridge; Inspection; Assessment; Marine Environment

1 Introduction. Planning a Bridge Inspection

The paper covers both aspects of deterioration and structure safety assessment. A preliminary visit is essential, often in the stage of preparing the proposal or contract. This first visit will identify visible signs of deterioration or structure distress and will reveal the required means of access to inspect the bridge.

In what concerns the visual inspection it is very important to get near the structure surfaces what in general requires special means of access. In what concerns testing it is essential to define beforehand the objective of the inspection and to identify the visible problems so that suitable tests are chosen. The number and location of tests is then planned.

2 Inspection

The basic inspection includes the visual inspection and the fundamental tests strictly required to access the degree and extent of the deterioration and structure distress.

The visual inspection has the objective of producing a deterioration mapping and a description of the main visible defects in the superstructure, in foundations, in bearings and expansion joints and in non structural elements. At the same occasion the conformity of the geometry of the bridge with the design drawings shall be checked and hammering to detect delamination executed (very important do be done on this occasion if access is difficult).

Testing is needed to access the deterioration process which is in general a two phase process with an initiation stage where no defects are visible and a propagation phase, usually much shorter than the initiation one, which leads to visible defects.

For concrete bridges basic testing includes the hammering for delamination detection, the bar location and cover measurement, the measurement of the depth of carbonation and chloride penetration (this last one of special importance in bridges located near the sea).

For steel bridges, basic testing includes measuring the thickness and bond of the painting, the loss of section due to corrosion, the use of penetration liquids to identify cracks in welding, the measurement of distortions or deviations from flatness, the checking for loose bolts, ...

In some cases special testing may be required. For concrete bridges cores are a very important mean to see the concrete surface quality and to obtain samples to perform various tests. When depassivation of reinforcement occurs the rate of corrosion may be of special interest and then electrical potential, concrete resistivity and polarisation resistance are needed to be performed in selected areas.

For steel old (more than 50 years) bridges steel samples shall be taken from suitable locations to perform tests on the chemical composition, tensile strength and ductility, energy dissipation (Charpy) and fatigue resistance (of a vital importance for railway bridges).

3 Assessment

The assessment of the structure response and safety may require the following works: -topographic structure geometry definition; dynamic response to traffic excitation; measurement of live loads acting in the bridge; static or dynamic load tests; strain/stress evaluation.

The objective of assessment is to understand the causes and effects of the mechanisms of deterioration or structure distress or to assess the structure safety levels. In the paper the assessment of chloride penetration in a concrete bridge is presented and the structure assessment discussed.

4 Examples

The Arrabida Bridge Over River Douro in Oporto has a total length of 493.2 m and an arch with 278.4 m span, 52 m height. The bridge is located in an atmospheric marine environment and was designed and constructed 35 years ago with great care and quality for its period. A concrete cube strength of 40 MPa was specified (a design mix with w/c = 0.32 was used) and all concrete surfaces were painted. However significant visible deterioration was found, after 30 years, underside the deck and in particular in the lower corner of the beams. The inspection showed that only in areas of defects in concrete quality or very low cover, the deterioration reached the level of delamination or exposed bars. The depassivation of steel was due to the chloride penetration.

The Figueira da Foz Bridge Over River Mondego has a total length of 1 421 m including a cable stayed steel bridge 405 m long with a central span of 225 m. The bridge was constructed 20 years ago. The visual inspection showed that the steel paint is destroyed and that corrosion is progressing, mainly in the areas adjacent to the concrete slab, which cracked and lead to water leakage. The stays and deviators were open for inspection showing that the steel galvanised protection is still active. In the reinforced concrete elements corrosion and exposed bars are already visible in the mats and piers. The structure safety assessment revealed that an inadequate seismic design associated with the fixing conditions of the deck in the towers and piers resulting there in insufficient safety levels.

Failures, Repair and Protection of Cooling Towers

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Summary

The presented paper will attempt to summarise some conclusion from a comprehensive refurbishment program successfully implemented at one coal and one nuclear power plant with totally 12 cooling towers.
In practice we find that cooling towers have frequently experienced accelerated deterioration due to concrete deterioration, reinforcing steel corrosion, the temperature gradient and freeze-thaw cycling. The objective of the initial inspection and testing was to establish the extend of deterioration which will serve as a basis for service life prediction and repair methods of cooling towers.

Keywords: Cooling tower, corrosion, failures, protection, repair methods, strengthening

1 Introduction

Cylindrical-shaped structures occupy a special place in industrial structures. Chimneys, silos, cooling towers and water tanks fall under this category. These structures have in common that large wall surfaces are exposed to environmental factors, which can be quite substantial and have a great impact on the serviceability and durability. In practice, we find that cooling towers have frequently experienced significant deterioration due to reinforcing steel corrosion, the temperature gradient and the freeze-thaw cycling. The varying exposures often act synergistically to impose an increasingly aggressive attack on the concrete and reinforcing steel.

2 Analysis of the critical points

2.1 Thermal stresses

The following two operational cases can lead to vertical cracks which can impair the function of the cooling tower shell:

- Normal operation: Because of the heating up by steam and as a result of external weather influences like frost and solar radiation, temperature differences dT are encountered in the shell.

As a consequence, bending moments are produced which, starting approximately from dT = 15 K, leading to the formation of vertical cracks.

- Set out of operation: Sudden temperature decrease (from about 25°C) in the thinner parts leads to great temperature stresses in the lower part of the shell. Excessive crack formation in the tensile area is the consequence.

2.2 Freeze-thaw bursting

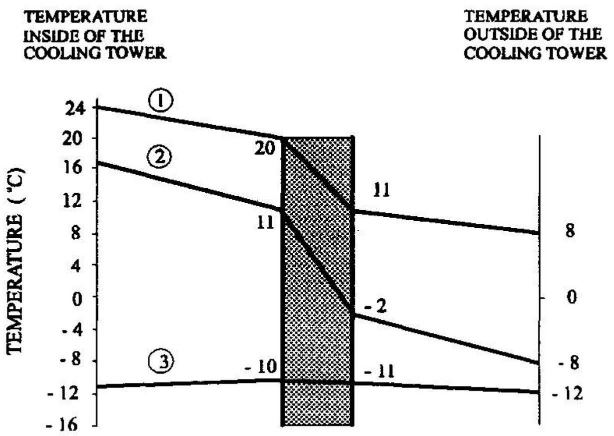

Fig. 1 Temperature profiles through reinforced concrete shell

Transition from water to ice involves an increase in volume by 9%. When cement paste is saturated with water this expansion of water on freezing can lead to disruption of the concrete if its strength is insufficient to resist the forces involved. The saturation need not be over the whole volume of the concrete and surface layers can suffer frost attack even though the main body is unaffected or vice versa. Figure 1 is a set of measurements made on the shell of a 125 m high cooling tower during a winter. Lines 1 and 2 were recorded at times when the tower was in normal operation. Line 3 was recorded when the tower was set out of operation. Note particularly that the freezing temperature of the outer shell surface was reached when the outside temperature was about - 8 °C. When set out of operation after 4 - 5 hours the temperature inside the shell decreased on the level of the ambient air. One episode of freezing is not enough to cause damage and major effects occur after a number of freeze-thaw cycles. The concrete surface in the middle European area is exposed to freeze-thaw cycles more than 100 times in a year. Since the exposed surface of the relatively thin concrete shell will be more susceptible to rapid cooling and thawing and because the surface is more likely to be saturated, the general effect of frost attack is a scaling or delamination of the surface.

3. Conclusions

1. Corrosion of reinforcement and climate induced cyclic thermal loads (temperature gradient or freeze-thaw cycling) are the most common sources of deterioration of cooling towers.
2. The selection of successful repair techniques should consider the causes of cracking and whether the cracks are dormant or active.
3. A long delay in repairs rapidly increase repair costs and ultimately imposes risks of structural failure. As an increasing amount of efforts is being spent on maintenance and repair of cooling towers, the industry has been going through a learning curve to achieve cost - effective repair and protection methods and materials.
4. Surface treatments should give an added protection against carbonation (R > 50 m) and at the same time decrease the water saturation of concrete under 92%.

Preservation Strategy of the Shortest Highway Crossing of the Alps

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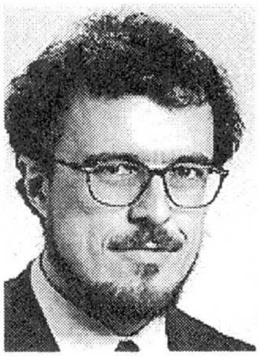

Thomas Vogel, born 1955, graduated from the ETH Zurich in 1980. He gained 11 years of experience in bridge and structural design in consulting firms in Chur and Zurich. In 1992 he joined the ETH as a professor of Structural Engineering.

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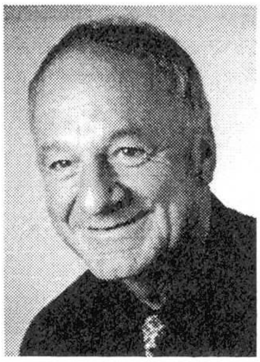

Heribert Huber, born 1940, graduated from the ETH Zurich in 1968. He gained 18 years of experience in bridge and structural design in consulting and management functions and as vice-director of an insurance company in Zurich. Since 1986 he heads the structural department of the Baudirektion Uri.

Summary

Preservation programme and rehabilitation measures for the highway forming the Northern access to the St. Gotthard pass and tunnel are described. After a short historical background the preservation programme is explained with emphasis on systematics concerning grouping of the network, guidance of traffic, planning and execution phases and categories of intervention. Financing and organisation as well as the actual state are briefly mentioned. Finally, some groups of structures like bridges, galleries and retaining walls are mentioned, highlighting problems and solutions common to these groups and some special measures taken or in progress.

Keywords: avalanches, bridges, chlorides, columns, desalination, edge beams, galleries, natural hazards, rehabilitation, retaining walls.

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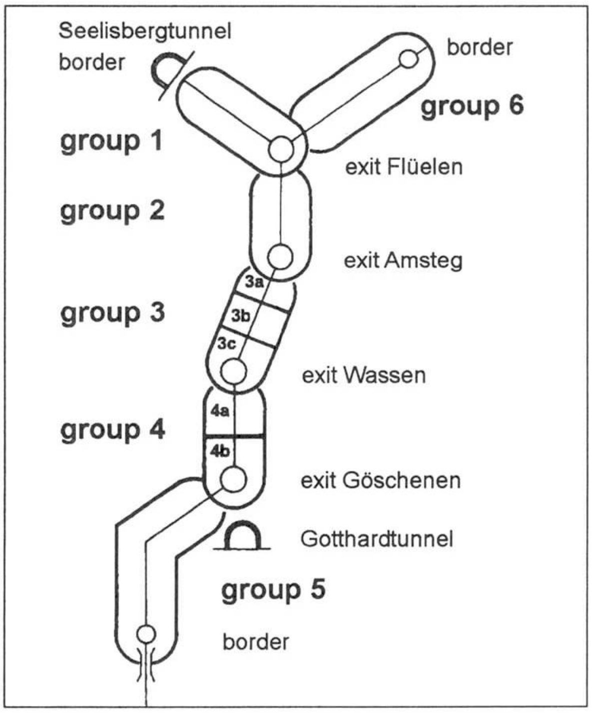

Fig. 1 Grouped net of federal highways of the Canton Uri

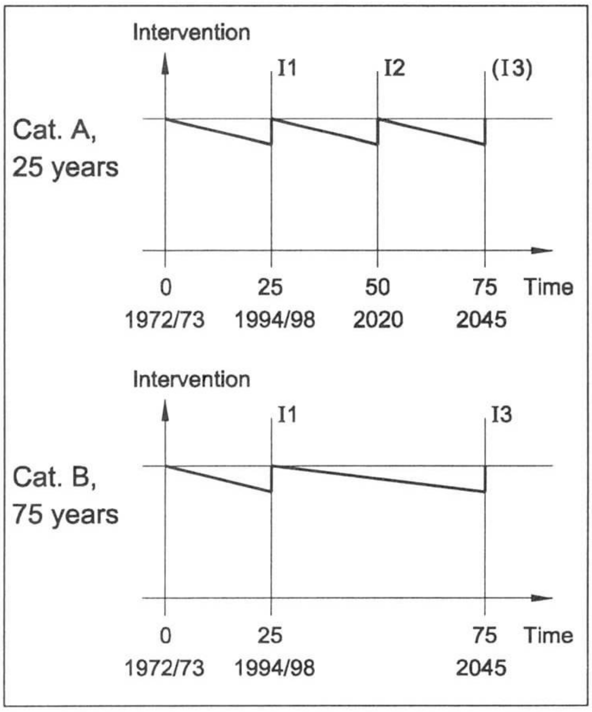

Fig. 7 Categories of interventions

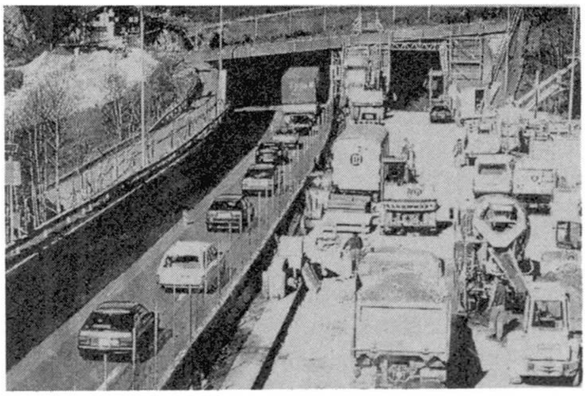

Fig. 5 Highly congested construction site

5. Conclusions

On important roadways with no possibility for detours, preservation programmes and consecutive rehabilitation activities are governed by the needs of traffic. All activities on site have to be planned carefully in order to be able to reduce restrictions and interruptions to a minimum. Rehabilitation measures need a clear goal expressed as the duration of a trouble-free period.

5.2 References

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Strengthening for an Existing RC Gerber Bridge Using External Cables

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Abstract

In Japan, a lot of continuous reinforced concrete (RC) gerber girder bridges which were constructed in 1950~60's, at present play an important role of ground transportation. However, these bridges will be required an urgent rehabilitation work since they have severe damage and shortage of loading capacity due to long-term deterioration as well as increase in design vehicle load.

This study describes a new technique for strengthening an existing continuous reinforced concrete gerber bridge, which benefits from requiring minimum traffic disruption. The proposed strengthening system aims at reducing the excess live shear load at the gerber hinge by lifting cantilever girder up, not to introduce compressive stress into the girder such as ordinary external prestressing methods. In this system as shown in *Fig.1*, the external tendons are arranged along the whole length of bridge, and deflected by the deviator attached the additional lateral beam beside the gerber hinge, and anchored by the concrete anchor block constructed on the extension of the bridge as earth anchored system. In order to prevent the reduction of cable tension by the friction, a steel device is installed in the underside of concrete deviator (*Fig.2*).

The proposed system has been first adopted in the strengthening of Titose Bridge, which is the 177.2 m long seven-span continuous reinforced concrete gerber girder bridge, completed in 1955. On-site construction was conducted with little traffic interruption by the work with hanging scaffold under the bridge. Prestressing was done from both sides of the suspended girder divided 3 blocks, and from one side of anchor girder, respectively. Prestressing for 3 suspended tendons and 2 anchor tendons were simultaneously done by using 10 hydraulic jacks so that the horizontal

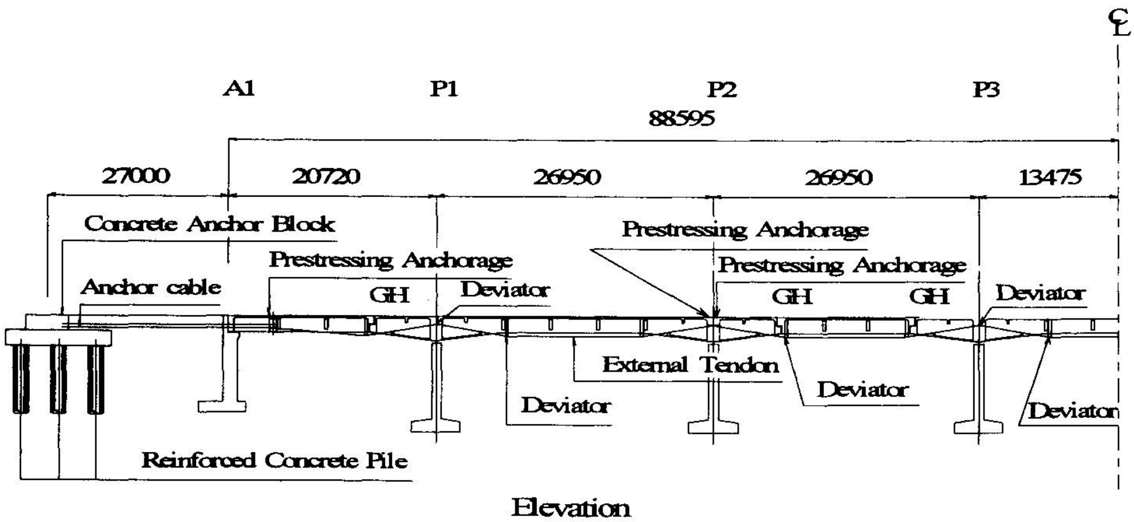

Fig.1 : Cable Layout for Continuous Concrete Gerber Girder Bridge

force may not occur into the bridge piers due to unbalanced cable tension. Assuming the friction coefficient μ=0.3 /rad to be an upper limit for the effect of deflected tendons at the deviator, the prestressing work was conducted in the range of $0<\mu<0.3$ /rad by elongation control of each tendon. This work was executed in two nights from 22 to 5 o'clock with an overall stop of traffic, including the field tests.

The strengthening effect was confirmed by the field test on the bridge which was carried in parallel to the work. As examples of the test results, *Figs. 3* and *4* show the reduced reaction at each gerber hinge, and comparison of girder deflections between before and after strengthening work, respectively. From the results, the following are confirmed: (1) the effectuality of the proposed strengthening system, (2) validity of the adopted design and analytical model, and (3) validity of the construction procedure. In addition, it can be said the application of PC technology to the new field in the point of using PC tendons with the purpose except for stress introduction means.

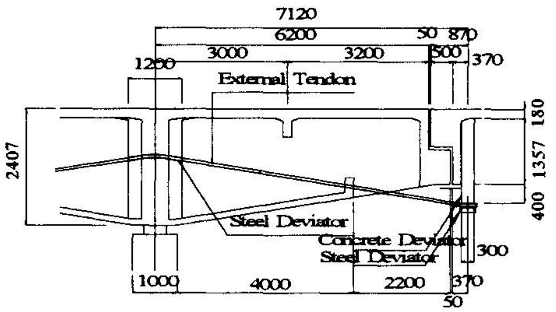

Fig.2 : Cable Arrangement at Gerber Hinge

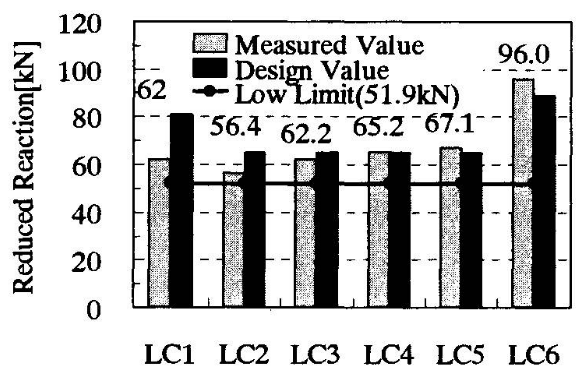

Fig.3 : Reduced Reaction in Gerber Hinge Support

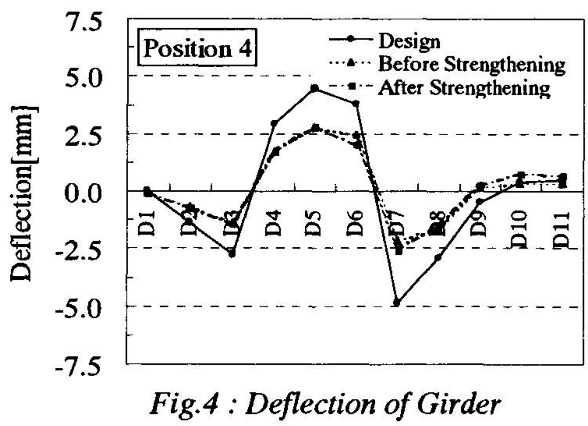

Fig.4 : Deflection of Girder

Bridge Rehabilitation with a Specific Composite Deck Construction

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Summary

This paper reports on experimental and numerical investigations on the local-carrying behaviour of a composite deck consisting of a reinforced concrete slab and an orthotropic steel deck. This deck construction was used for the rehabilitation of a major motorway bridge of a transit route in Austria.

Abstract

The motorway bridge near Salzburg in Austria on the North-South transit route is one of the most heavily loaded road-bridges of this area. Up to the average of 7000 trucks daily and about 200 special vehicles of 150 to 240 tons a year the load impact on the traffic deck has been continuously increasing. The steel bridge in form of a 'middle arch structure' was built as the first example of this type of structural system in 1970. It is a structure with a bow string arch in the centerline with a span of 133 m and the traffic deck projecting to both sides over 15 m each (Fig. 1).
The deck construction is an orthotropic steel deck with an asphalt layer on it and covered by a concrete surface. As result of the highly increased impact of the traffic load compared to the construction time the deck surface has been damaged in form of cracks in the concrete slab, which allowed penetration of water, seriously impairing the consistency of the asphalt layer.
The rehabilitation of the traffic deck was based on the idea of rearranging the two layers on top of the orthotropic deck, the first now being the reinforced concrete slab (t=140 mm) and the second the asphalt surface (t=80 mm). The concrete slab could now be connected with the orthotropic steel deck by stud bolts resulting in a composite action.
In this composite construction the two plates of very different stiffness have to work together; the concrete plate should distribute the local traffic loads and the orthotropic steel plate should transfer them to the main girders.
For a realistic basis of design experimental and numerical investigations were performed. The tests were performed by models which represent a cut out of the traffic deck in the scale 1:1. The boundary conditions were adapted to the global plate behaviour of the deck construction by specific support girders. Fig. 2 illustrates the test model and the constructional details of the composite deck. The test loading was defined by local wheel-loads, which were derived from the load models given by EC1-2. The dominating local load resulted in the wheel-load of 200 kN acting on the base area of 40·40 cm. The test loading was carried out statically as well as dynamically up to $3 \cdot 10^6$ load cycles for ΔP=140 kN. Vertical deformations of the deck surface, differential deformations between the horizontal planes of steel plate and concrete slab as well as strains of the steel ribs were measured.

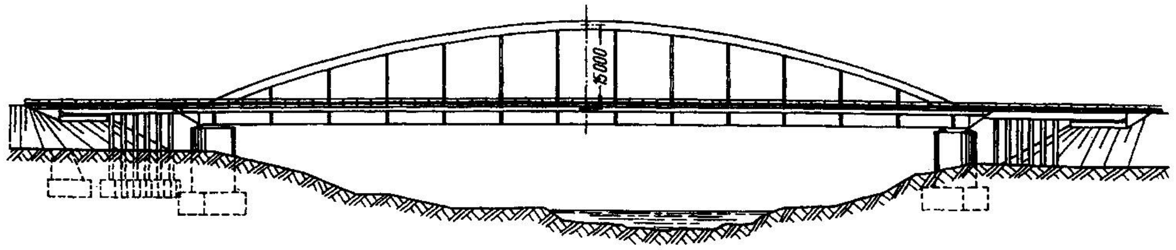

Fig. 1 Structural system of the motorway bridge near Salzburg

For reason of comparison the test-loadcases were also numerically calculated. The structural steel components were modelled by means of shell- and beam finite elements. The concrete slab was represented by shell elements and a material model, which accounts for the specific behaviour of concrete and reinforcement. The composite action between the concrete slab and the steel deck was investigated by various assumptions, i.e. discrete shear studs of different stiffness, continuous rigid composite action or pure contact without any composite action.

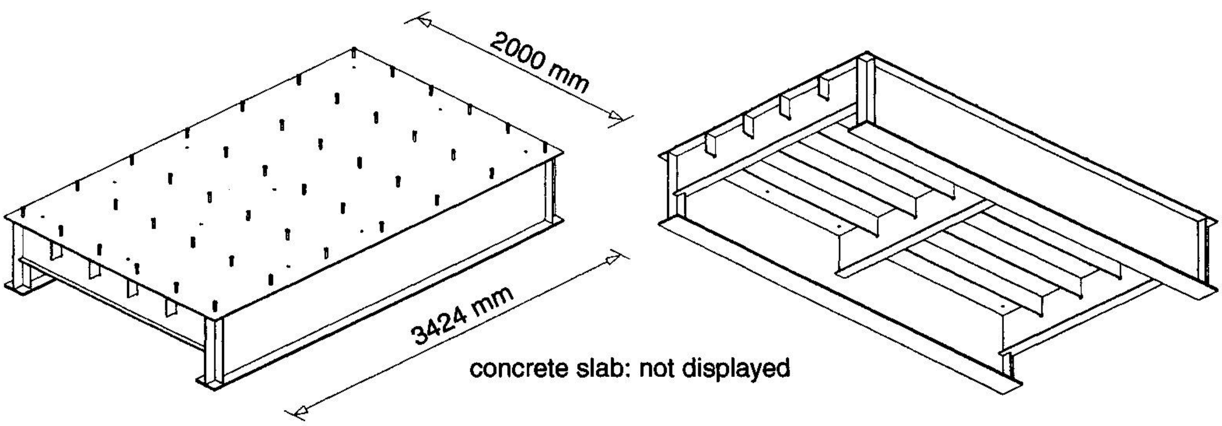

Fig. 2 Local test model

Four test models were investigated. Test model (1) and (2) differ in its strength class of the concrete. The results show that the effect of the two concrete classes is practically insignificant. The tests further verified that the fatigue effect in the composite action due to about $3 \cdot 10^6$ load cycles was very low, resulting in an increase of deformation up to about 10% only. Among the numerically investigated alternatives the closest accordance with the test results was obtained for the assumption of rigid composite action. Further to the tests with the EC-load model 2 increased load steps were investigated up to 980 kN. At the load step 740 kN the test indicated acoustically as well as by a jump of the differential deformation between concrete plate and steel deck that the adhesive strength in the contact plane had been exceeded.
Test model (3) was carried out without a regular composite action, i.e. without the action of shear studs and adhesive strength. The beneficial effect of the composite action on the load carrying behaviour could be evaluated by comparison with test model (2). The maximum deformations and stresses for 200 kN differ by a factor of approximately 2.
Test model (4) was carried out for the pure steel deck without a concrete slab; it indicated that the steel deck without concrete slab would significantly be overstressed.
The investigations resulted in fairly good accordance between test and numerical analysis and indicated that the composite construction presents a robust load carrying behaviour for the high local traffic load of EC1. The rehabilitation of the motorway-bridge accordingly was carried out in this way.

Rehabilitation of the Austrian Mint, a Historic Monument

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Walter J. Paul, born 1931, received his civil engineering degree from Technical University Vienna, formerly professor at the Federal Technical College Wiener Neustadt (A) for 33 years in addition to heading of an own structural design company, he is now still structural engineering consultant.

Franz PAPEZ
Engineer
Executive Vice President
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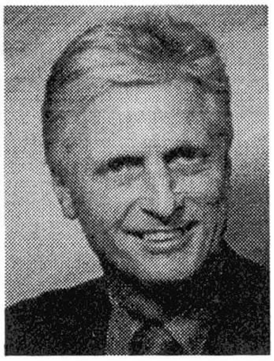
Franz Papez, born 1950, received his construction engineering degree from Federal Technical College Vienna; after having been an employee for building contractors and architect's offices he is now working in executive position for "Projektierungsbüro für Industrie- Hoch- und Tiefbau AG", Vienna.



Summary

The Vienna mint edifice, A-1030 Vienna, Am Heumarkt 1 is the visible expression of the 800 years old Austrian tradition in coin- and medallion minting. The historical continuity of minting had to be carried on into our days, modernizing the mint- and medallion striking by setting-up hitech production methods.

Keywords: Planning hitech parallel out-dated production, strengthening vaults and walls, static safeguarding, historic façades, stone monuments, two-storied basement in inner courtyard.

1. Introduction

After the decision by the Board of the Austrian Mint, the company's edifice in Vienna, a historic monument from Biedermeier period, built 1835 – 1838 by the famous Austrian architect Paul E. SPRENGER, had to be converted into a hitech coin- and medallion production centre and renovated for preservation in many sections of the building and in architectural details of the historic substance. All measures had to be coordinated with the Austrian Federal Monument Administration. The Austrian coinage has continuity since 1194, but from the year 1918 the only Austrian mint remained in Vienna with now remarkable high quality coining - especially known are the gold coins "Wiener Philharmoniker" since 1989, celebrating the worldwide famous Vienna Philharmonic Orchestra and since 1998 the actual new European currency "Euro".

The edifice was built conform to the last century's art and craft of coining: heavy brickwalls and brickbasements, vaults and piers in the lower parts, systems of dowelled waney wooden beams in the upper floors – in dimension and loadbearing capacities dependent on equipment weights and life loads necessary for production and office rooms in those times.

Developments in the minting technology became absolutely imperative to modernize all mint production processes, although the running mint production had not to be interrupted during these activities. That required to reorganize the arrangements of the production areas. The engineering necessities resulted in ingenious ideas for strengthening of vaults and walls in order to bear heavier loads, penetrations of wall-zones and wide tasks in static safeguarding and building stabilizing. All engineering work was carried out successfully, ecologically and economically throughout the period of planning, building and renovating phases since 1989.

2. Planning – Preparing the technology

Description of the planning concept, the ways of its realisation besides the existing production full in function and the measures in restoring the edifice as a historic heritage

3. Strengthening of Vaults

3.1. Main Purposes for the Measures

Report about the necessity of the strengthening measures and the engineering challenges in the course of the works

3.2. Chosen Method of Strengthening

Presenting one typical example out of a large number of treated cases; structural analysis and parameters in the chosen method

4. Preservation of the Edifice

4.1. Main Purposes for the Preservation Work

The importance of the preservation of the historic heritage; pointing out the damages of the façade-crowning group of stone figures

4.2. Methods of Repair

Explanation of the renovating and preservation measures for the above mentioned stone structure

5. Inner Courtyard

Planning and constructing a new part of the building situated in the underground of the inner courtyard

6. Discussion and Conclusions

7. References

Fig. 1: Aspect of the renovated and strengthened edifice from the main road "Am Heumarkt 1"

Damping Wind and Traffic-Induced Oscillations of the Rio-Niterói Bridge

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Michèle Pfeil, born in 1959, received her B.Eng. from Catholic University in Rio and her M.Sc. and Ph.D. from COPPE Federal University. Her current field of research is aeroelastic problems of bridges.

Abstract

Brought into service in March 1974 the 13.3 kilometers long Rio-Niterói bridge spans across the Guanabara bay in Rio de Janeiro. Most of it is a prestressed concrete structure but its three central spans (of 200 - 300 - 200 meters) are bridged by remarkably long and slender continuous steel twin box girders, as illustrated in Figs. 1 and 2. The central navigation span is the largest steel girder span in the world and together with the side and link spans weigh 13,100 tonnes of steel and make a total length of 848 meters[1].

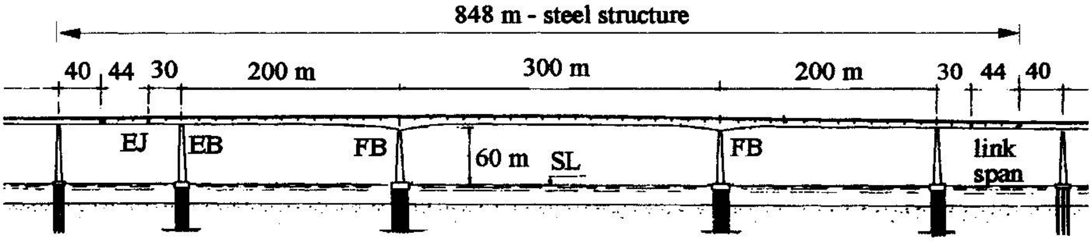
A recurrent aeroelastic aspect of this very slender steel bridge - with average girder height to span length ratio close to 1/45 - is its across-wind vertical bending motion. Cross-winds of relatively low velocities have often set into vortex-induced oscillations the lightly damped twin-box-girders. Because of this behaviour, the bridge has to be closed to traffic of any vehicle, for the sake of user's confort and overall safety, whenever cross winds reach velocities near 50 km/h (~14m/s). For sustained wind velocities close to 60 km/h the bluff box section bridge experiences vortex-induced oscillations in the first vertical bending mode with amplitudes that may well reach values about 250mm at the center of the navigation span. Theoretical-numerical and experimental results from wind-tunnel tests on a sectional model [1-2], plus astonishing images documented by video-cameras installed on the bridge for traffic control, corroborate the previous statements. Large vertical oscillations that left people panic-stricken were first reported on relation to a storm that occurred in August 17, 1980, and more recently in October 16, 1997 and February 17, 1998, when oscillations lasted 5 to 7 minutes. This deterrent aspect of the world largest steel box-girder bridge was explored to develop the conceptual design of feasible passive control devices [1,2] to attenuate the observed oscillation amplitudes - due to the action of winds as well as traffic of heavy vehicles on irregular pavement and joints - and, consequently, to upgrade user's confort and the serviceability of this bridge, that has an average daily traffic of 100,000 vehicles

An appraisal of the actual bridge dynamic behaviour is made by a mathematical 3D FEM model calibrated in terms of experimentally measured frequencies and associated oscillation modes [1]. The derived modal equations, including correlated aeroelastic forces, are further combined with a multi-objective optimization technique [1,2] to assist in designing simple mechanical and robust tuned-mass-dampers. A feasible mechanical arrangement is given by small masses TMD's distributed along the center span. Table 1 presents the structure's modal parameters and the TMD's

characteristics, whose performance is demonstrated through comparison of numerical results obtained for time responses of the original and controlled structure (Fig. 3).

Table 1 - Structure and Passive TMD's characteristics

Characteristics	Structure $( )_B$	16 TMD's $( )_A$	Ratios$( )_A / ( )_B$
Frequency (Hz)	0.32	0.29	0.9
Mass, m (ton)	5×10^3	50.0	0.01
Damping, ξ (%)	1.0	7.5	7.5



Legend: EB = Expansion Bearing; FB = Fixed Bearing; EJ = Expansion Joint; SL = sea level

Figure 1 - Elevation of Central Spans of Rio-Niterói Bridge

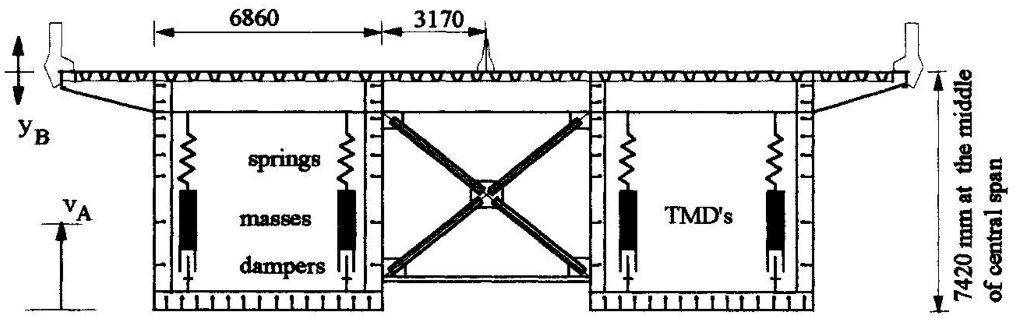


Figure 2 - Typical Cross Section with Tuned-Mass-Dampers located in the middle of central span

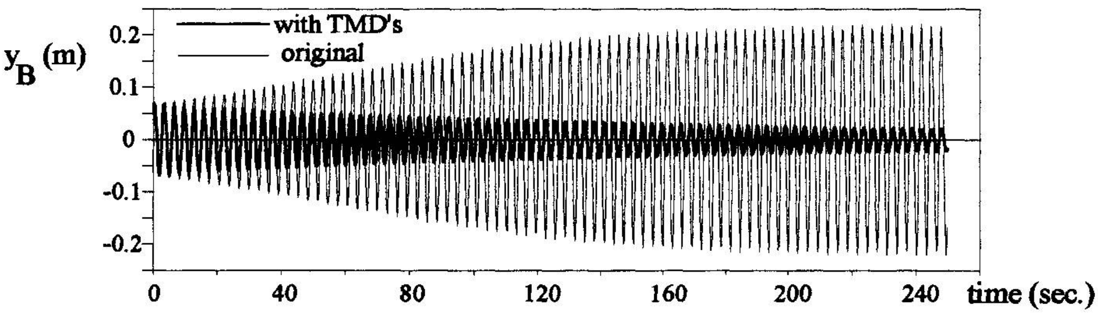


Figure 3 – Amplitude response in the first bending mode (middle of central span)

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Upgrading of the North Viaduct of the Suspension Bridge in Lisbon

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Summary

The purpose of this paper is to describe the works of strengthening, improvement and elimination of expansion joints, in the North Viaduct of the Tagus River Suspension Bridge.

Keywords: External Prestressing ; Vertical Prestressing ; Assesment ; Upgrading ; Strengthening ; Epoxy Resin ; Repair Mortar

1. Introduction

The suspension bridge over the Tagus River in Lisbon – Portugal, was one of the largest in the world, when opened to traffic in 1966,.

The bridge comprise the suspension bridge itself, in steel, with a total length of 2300 m and a central span of 1013 m (the longest in Europe at the time), and the viaduct over the North embankment (North Viaduct), in concrete, with a total length of 950 m .

The bridge, which was initially used for roadway traffic, was submitted to large works for the installation of the railway traffic, in a new deck built under the existing roadway platform.

The North Viaduct is a prestressed concrete bridge with typical spans of 76 m. It was built by the balanced cantilever method with expansion joints at every mid spans.

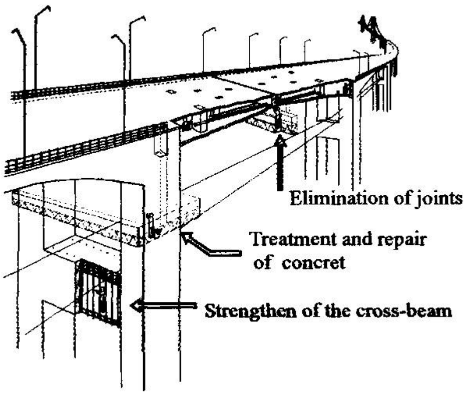
The railway installation in the viaduct, obliged to the strengthening of some structural elements, and the elimination of the most part of the expansion joints existing in the roadway deck. Besides the general good behaviour of the viaduct, a general repair was carried out with the closing of the cracks and the substitution of the degraded concrete.

In this paper is presented a general description of the execution of the works in the viaduct, which were strongly affected by the specific conditions of the works, namely the fact they, were executed in an urban area, the large height of the columns, and the need of non stopping the roadway traffic.

The pathologies observed in the viaduct were due to:
- the deficient concrete cover that justifies the stage of degradation;
- the absence of prestressing, which leads to crack formation.

2 - Injection of Epoxy Resins and Repair of Concrete

The cracked zones of the roadway deck and of the columns, and specially the edges of the cantilevers, were submitted to a treatment by injection of epoxy resin by the BICS method.
In places where the structural concrete was disintegrated, it was removed and repaired with non-retractile high resistance mortars.



The main work carried out were:

- Strengthening of the cross-beams of the columns;
- Elimination of expansion joints;
- Treatment and repair of the degraded concrete.

3 - Strengthening of the Cross-Beams of the Columns

To support the new railway platform the cross-beams between the column shafts had to be reinforced because of their insufficient shear resistance. To strengthen the cross-beams, the following works were carried out:
- Increasing of the cross-beam wall by the execution of filling concrete walls, on the interior side, with 0.30 m thickness and solidarized to the existing structure by connectors.
- Application of vertical prestressing to the cross-beam walls, obtained through the stressing of prestressing steel bars that cross the walls of the filling concrete and the slabs of the existing structure.
- Increasing of the thickness of the cross-beam top slab and execution of plinths to support the steel beams of the railway platform.

4 - Elimination of the Joints on the Roadway Deck

The deficient behaviour of the expansion joints on the roadway deck, led to relative vertical displacements between the edges of the cantilevers, which caused deterioration, as well as cracking of the concrete of the structure.
So, it has been decided to eliminate of eight of the twelve existing joints on the platform.
The structural modification of the spans continuity, was effected through the placing of exterior prestressing cables anchored on columns disposed symmetrically on the balances of the adjacent cantilevers. These columns were connected on the base to horizontal beams, solidarized to the bottom flange of the box girder and on the top to the superior flange, through indentation.
The joints eliminated showed important relative displacements between cantilevers, caused mainly by the traffic. To proceed to the joint elimination it was firstly necessary to nullify these displacements.
This immobilization was effected through the opening of the joint using hydraulic jacks, interposition of metallic plates, placed on both sides and on all extension of the joint, and application of longitudinal prestressing through the joint.

Rehabilitation of Parking Garages

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Summary

Millions of dollars are spent annually to address deterioration and maintain or restore serviceability in reinforced concrete parking structures. Investigation of distressed conditions in hundreds of parking structures and subsequent design and installation of repairs have revealed the most common causes of distress and the most effective ways to repair and extend the life of these structures. This experience has also lead to the development of maintenance priorities that can improve the serviceability of parking structures and reduce the probability that future major repairs will be required.

1. Visual Signs of Problems

The most common signs of potential problems in parking structures are cracking and spalling of concrete and leakage of water through cracks and joints.

Cracking in concrete structures is not unusual and occurs for a variety of reasons. One of the most common is the stresses caused by restraint to ordinary volume changes in the structure. Loadings that exceed the cracking strength of the concrete are another cause. Cracking can also be an indication that corrosion of embedded reinforcement is occurring or that a structural deficiency exists.

While most cracks are not an immediate structural concern, all cracks should be assessed by an engineer or other qualified person to verify their cause. If the cracking is not related to a structural deficiency, the cracks should be evaluated for their impact on serviceability. If exposure to moisture and salts is not severe, no further action may be necessary. On the other hand, in a high moisture or salt environment, the presence of any cracking can accelerate penetration of moisture and salts into the concrete causing reinforcement to corrode. Water penetrating through cracks in overhead slabs can also drip onto vehicles and damage finishes.

Concrete spalling is usually caused by corrosion of steel reinforcing embedded in concrete that is exposed to moisture and chlorides (from de-icing salts). Since the volume of corrosion by-products (rust) is significantly greater than the volume of the original steel, corrosion of embedded bars creates expansive forces which can cause the concrete to crack and spall.

Cracking and debonding of sealants used in the joints of certain types of concrete parking structures are a sign that maintenance or repair is needed. Visible distress in expansion joint seals, which may not have been designed to accommodate structural movements or which have been damaged by snowplows or vehicular traffic, are also a sign of problems.

Visible signs of distress should be evaluated. This is usually done by performing a condition survey which may be supplemented by structural analysis or testing. The specifics of a typical condition survey, which include document study, visual examination, delamination survey, core sampling, and nondestructive testing, are discussed.

2. Repair Techniques

The results of the condition survey are used to develop repair documents. Repair strategy depends on the condition of the structure. In addition to technical issues, selection of the appropriate strategy will be dictated by costs, the owner's future plans, and availability of funds. Scheduling of repairs will depend on the need to maintain partial occupancy during the work.

Proven techniques for patching floor delaminations and overhead and vertical distress, installing full-depth slab repairs, sealing cracks, adding bonded concrete floor overlays, installing membranes and sealers, and addressing expansion joint and drainage problems are covered.

Specifics on each technique and useful tips based on successful experience are provided. A recommended maintenance program for a reinforced concrete structure in reasonably good condition, and located in an environment with moderate to heavy moisture and salt exposure is also provided.

3. Conclusions

The causes of distress and deterioration in reinforced concrete parking structures are common and well known. Experience proven procedures are available to successfully repair the common types of distress and deterioration. Once repairs are completed, establishing a routine and reasonably inexpensive maintenance program can prolong the useful life of the structure and minimize the need for costly additional repairs.

Preservation of Structural Monuments – Charles Bridge

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Summary

Degradation and corrosive processes of structures cannot be generally prevented. They are manifested by the structures' ageing. Experimental monitoring and investigation of Charles Bridge has shown exceptional importance of non-force impacts and effects acting on structures. These impacts and effects due to namely physical, chemical and microbial processes are a major cause of gradual deterioration and ageing of structures. From the point of view of preservation of prominent structural and cultural monuments, it is necessary to study these processes, their causes and mutual connections.

Keywords: physical degradation, recrystallisation cycles, microbial corrosion, biodegradation, non-force impacts, chemical and microbial processes, structural damage, cyclic temperature and moisture impacts, long-term impacts

1. Historical introduction

On 9th July 1357, Charles IV laid the foundation of a new stone bridge with a level line by 4 m higher than that of Judith's Bridge, with 16 compartments with a clear span of 16.62 m to 23.38 m. The overall bridge length is 515.76 m and it is 9.4 m wide. Its plan is slightly elongated, goosenecked, curved in shape. The dimensions of massive stone piers at the vault impost range between 8.5 m to 10.84 m x 24.0 m to 25.0 m. Pier heads have been "sharpened" against the water stream at an angle of 65°. The bridge construction, entrusted to Petr Parléř's stone works, was completed in 1406.

2. Physical, chemical, microbial and mechanical impacts

Among the most frequent causes of primary physical degradation of stone masonry there is migration and crystallisation of salts, combined with moisture penetration inside stone and with frost cycles. The samples taken from sandstone blocks of Charles Bridge were subjected to frost and recrystallisation cycles. The achieved partial results have proved that frost cycles do not represent extreme danger for the stability of sandstone blocks of Charles Bridge. The stone degradation is only of surface character. Deeper changes have not been recorded. The impact of salts contained in considerable amounts in sandstone blocks is much more noticeable. In order to achieve moisture balance between the inside and outside environment during stone moistening, due to their hydroscopic properties salts either absorb or release water. Thus hydration and crystallisation pressures arise which may affect the development of fine expansion joints or microcracks in stone. This was also confirmed by experimental results. As early as after the first cycle, fine cracks (arkose) appeared which gradually grew wider. It is probable that the old stone blocks of Charles Bridge in particular will be very much susceptible to hydration or recrystallisation pressures exerted namely by sulphate, chloride and nitrate salts, which is necessarily manifested by accelerated development of cracks on the stone surface. Due to variable chemism of solutions migrating in the pores of building stone, cyclic growth and, simultaneously, dissolution of salts occur (formation of two generations of plaster stone was proved). The ageing of Charles Bridge building stone is highly affected by deposits and crusts formed by minerals with high contents of molecular water. At the crusts' contact with the building stone surface, the crusts are dissolved and the solutions creep through the rock pores into the centre of the stone; this leads to salt crystallisation in the rock pores. The analysed samples proved that maximum concentration of crystallised salts occurred in the zone approximately 11mm below the stone facing. Integral part of sandstone weathering is microbial corrosion which is manifested by coloured stains and patina, efflorescence, pulverisation or organic material deposits on the stone. Sandstone bridge block biodegradation, however, does not represent just a cosmetic problem. It is a synergistic action combining both physical and climatic effects, as well as biological impacts.

Long-term measurements (1984-1988) show that in the course of an annual cycle vertical movements arise in the vault crown - crown rise and flattening (ranging within approximately 1/7500 - 1/2500 of the vault span) - while the character of the vertical deformations pattern corresponds to the outside temperature pattern in keeping with the season. The value showing the difference of measured vertical deformations given by the difference of the highest positive (rise) and the lowest negative (fall, flattening) value lies in the interval of 2 mm - 10 mm. The value of the permanent component of vertical deformation of crown sections growing in time (approximately 0.4 to 0.7 $mm.year^{-1}$) testifies to a gradual disintegration of the bridge vault masonry. The impacts of temperature and moisture cause horizontal deformations of namely the crown cross sections of the bridge structure whose permanent component grows in time (approximately 1.5 to 12 $mm.year^{-1}$). The deformations caused by non-constant temperature and moisture patterns along the bridge structure section are characterised, above all, by changes in the shape of the crown cross section, by the appearance of tensile transversal normal stresses, shear stresses between individual cross sections of the bridge structure and by the bridge body stratification due to shear forces. The temperature and structural analysis of a section of the Charles Bridge structure has shown that the values of maximum deformations of the bridge structure cross section range within 0.6 mm to 2.4 mm for a temperature load characterised by a 10 °C to 17 °C difference between the surface and core temperature. Maximum values of tensile normal stresses amount up to +770 kPa, while maximum values of compressive normal stresses reach -773 kPa (bottom section) for the given temperature load.

3. Conclusion

Degradation and corrosive processes of structures cannot be generally prevented. They are manifested by the structures' ageing. An integral part of any structural design and its implementation, or reconstruction has to include, apart from meeting the relevant demands, also the necessary reliability that guarantees the function of individual elements and the structure as a whole in time, or during the time of their presumed service life. This requires such solution that limits the intensity and kinetics of degradation and corrosive processes to the lowest practically achievable level.

FRP Composite Strengthening of Concrete Slabs

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Summary

Fiber reinforced polymer matrix composite materials show significant potential for use in the rehabilitation of aging and/or deteriorating concrete civil infrastructure components. This paper describes results of a focussed test program on the use of prefabricated carbon/epoxy strips used for the external strengthening of concrete slabs. Tests are conducted at full scale level and emphasise aspects of strengthening and repair. The tailoring of strip capacity (modulus) is also addressed and it is shown that optimisation of materials form and performance can result in cost-effective structural functionality.

Keywords: Strengthening; Slabs; Fiber-reinforced-composites; Carbon; Debonding; Concrete.

1. Introduction

In the past steel plates have been used for the external strengthening of damaged/deteriorated slabs, or in cases where an increase in load capacity for an existing structures is required. Although clearly feasible the method has its disadvantages in that the steel plates are heavy, require substantial equipment to place and connect (adhesive bonding with end bolting or anchoring), have problems related to length restrictions and field joining, are difficult to erect in cases where clearance is limited, and are susceptible to corrosion. In comparison, fiber reinforced polymer matrix composite plates provide ease of installation, high stiffness-to-weight and strength-to-weight ratios, light weight, and do not corrode. Although these materials have been extensively used in the field through demonstration projects, research on structural response and the development of design guidelines has been almost restricted to their use for the flexural strengthening of beams. However, a review of a large number of field demonstrations shows that a significant number of these have been related to the use of composite material for the rehabilitation of slabs wherein the response and geometrical configuration is substantially different from that of a beam.

Previous studies on the use of externally bonded composite strips for the strengthening of scaled slabs [1,2] have indicated that failure was almost always through debonding at about 50% of the actual material capacity. Debonding was generally initiated in the midspan region as a tension failure of the concrete cover between the steel reinforcing bars and the adhesive with the final failure path being through the composite itself. Materials related reasons for this response were given in [1], and a number of issues were raised related to effects of materials and geometrical conditions on failure initiation and resulting performance of the strengthened slabs. This paper focuses on investigation of some of these issues through a series of full-scale tests in which reinforcement detailing, composite strip geometry and properties, and condition of the slab prior to application of the composite serve as variables.

2. Materials and Test Methods

All tests were conducted on reinforced concrete slabs of dimension 4500 mm x 960 mm x 203 mm, tested in flexural loading with clear spans of 4170 mm. Load was introduced at midspan through two line loads 450 mm apart with the line loads being applied over the central 760 mm of width. A schematic of the test set-up is shown in Figure 1. Load was applied in displacement control at a rate of 0.85 mm/s. A set of 17 slabs was tested with details of configuration as listed in Table 1. It should be noted that these 17 slabs were divided into four groups of which the first set was unstrengthened and served as the control for the investigation. The second set was strengthened externally with Sika Carbodur strips as detailed in Table 1. In the third set which were also strengthened with Sika Carbodur strips the longitudinal steel reinforcement was modified such that four of the seven rebar were cut at midspan prior to pouring of concrete to simulate damage. The fourth set used tailored strips fabricated using the wet layup process. The pultruded strips had nominal tensile modulus and failure strains of 175 GPa and 0.65% respectively, whereas the strips fabricated using the wet layup process had a modulus of 85 GPa. Design of the strip configuration was made following strain limitations.

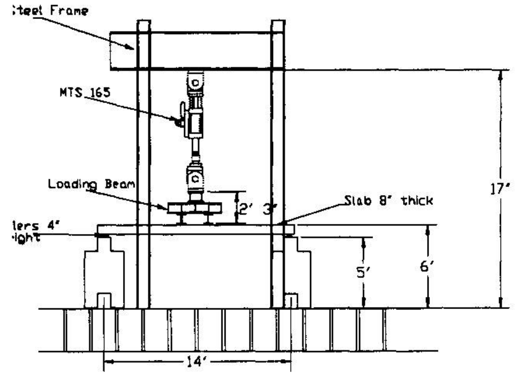

Figure 1: Test Set-Up

Type of Strengthening	Details of Longitudinal Steel Reinforcement	Details of CFRP Strengthening	Expected Moment Capacity for $\varepsilon_{L,u} = 0.65\%$
As-Built			
SF 01	3 # 6 bar (861mm²)		62,0 kNm
SF 03	7 # 4 (896mm²)		65,5 kNm
SF 07	7 # 4, 4 of them cut at midspan		28,7 kNm
Group A (Sika Strips)			
SF 1	3 # 6 (861mm²)	2 * S1012 (240mm²), L = 3250mm	113 kNm
SF 2	2 # 7 (782mm²)	2 * S1012 (240mm²), L = 3250mm	108 kNm
SF 15	7 # 4 (896mm²)	2 * S1012 (240mm²), L= 3250mm	116 kNm
SF 4	3 # 6 (861mm²)	4 * S 512 (240mm²), L = 3250mm	113 kNm
SF 5	3 # 6 (861mm²)	2 * S1012 (240mm²), L = 3610mm	113 kNm
SF 6	2 # 7 (782mm²)	2 * S1012 (240mm²), L = 3610mm	108 kNm
Group B (Repair)			
SF 7	7 # 4	2 * S1012 (240mm²), L = 3250mm	80,4 kNm
SF 8	4 of them cut at	2 * S 512 (120mm²), L = 1680mm	54,7 kNm
SF 9	midspan	4 * S 512 (240mm²), L = 1680mm	80,4 kNm
SF 10	($A_{s,eff}$ = 384mm²)	2 * S 512 (120mm²), L = 1680mm + 2 * S 512 (120mm²), L = 3250mm	80,4 kNm
Group C (VB Strips)			
SF 11	3 # 6 (861mm²)	3 * 3 ply (397mm²), L = 2900mm	103 kNm
SF 12	3 # 6 (861mm²)	3 * 6 ply (650mm²), L = 3250mm	129 kNm
SF 13	3 # 6 (861mm²)	2 * 6 ply (431mm²), L = 3250mm	107 kNm
SF 14	3 # 6 (861mm²)	3 * 3 ply (395mm²), L = 3250mm	103 kNm

Table 1: Overview of Test Specimens

Assessment and Repair of a Concrete Dockyard

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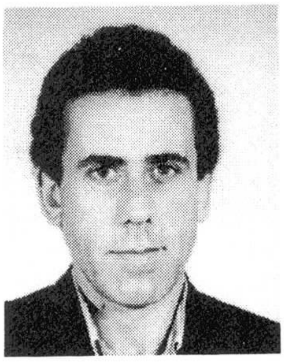

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Summary

Chloride induced corrosion is the main cause of distress of concrete structures exposed to the marine environment. This deterioration mechanism is very fast and often leads to a reduced service life. In this paper, results are presented of the condition survey and the repair methodology of a concrete dockyard, localised in the west coast of Portugal. The dock, with dimensions of 350 x 55 m, was built in 1974-75 and exhibits severe deterioration due to reinforcement corrosion. Some evidence of chemical attack on concrete has also been observed. Several tests including concrete chloride content analysis, concrete resistivity, potential mapping, corrosion rate measurements, petrographic analysis and scanning electron microscopy have been performed to allow a careful diagnosis and a full understanding of the deterioration process.

Keywords: reinforcement corrosion; concrete deterioration; marine environment; concrete repair; concrete dockyard

1. Introduction

This paper presents a condition survey and the repair methodology to be implemented on a concrete dockyard presenting significant deterioration. The structure (dock 22) is integrated in a large complex belonging to a shipyard, located in the estuary of river Sado, in the west coast of Portugal.

The shipyard has three reinforced concrete docks (docks 20, 21 and 22) built during the 1973-1975 period. The quick deterioration of the structures led to a first intervention in 1984. At the time, 3.600m^2 of the internal face of the walls of dock 20 have been repaired. In 1988, 5.600m^2 of the internal face of the walls of dock 21 have also been repaired. The other part of the walls of docks 20 and 21, corresponding to a total area of 23.400m^2, has been repaired in 1991.

Dock 22 has been built with a low quality concrete. The concrete mix used presented a quantity of cement of 300 Kg/m^3 and a water/cement ratio of 0.7. The concrete covers specified for the structure were 4 cm for the walls and 6 cm for the bottom slab. Tests have been done to determine the concrete behaviour. As regards the compressive strength, a characteristic value of 17.6 MPa has been obtained for the walls and 20.6 MPa for the bottom slab. The concrete of the walls presented a water permeability coefficient of 14x10^{-11} m/s and the concrete of the bottom slab exhibited a value of 0.6x10^{-11} m/s. The open porosity measured on the concrete of the walls and slab was 17.8 and 16.4% respectively.

2. Assessment of deterioration

To assess the deterioration condition of the structure a survey plan has been established, which involved the following phases: visual inspection and in situ and laboratory tests.

The main type of deterioration observed in the visual inspection has been the delamination and spalling of the concrete cover caused by reinforcement corrosion. It has been verified that the deterioration presents significant differences in the various zones of the dock associated to different exposure conditions. The walls of the dock exhibited a deterioration rate much higher than the one observed on the bottom slab. Generally, it has been verified that the walls presented a delamination higher than 30 % of their area, whereas on the bottom slab the area of delaminated concrete was about 10%.

The chloride contents measured at the reinforcement level range usually from 2 to 4% of the cement mass. The carbonation depth of the concrete is comparatively low. The maximum values measured are about 20 mm, values less than 10 mm having been usually obtained.

Tests on the electrical potential of reinforcement at the zones without delamination showed values ranging from 300 to 400 mv, reaching values of about 500-600 mv on some places. The values measured for the resistivity of the concrete range generally from 1.5 to 3 KΩcm on the walls and are below 1.5 KΩcm on the bottom slab. The values of the instantaneous corrosion rate measured on the bottom slab are within 17 to 23 μm/year.

The petrographic and microscopic analysis showed that the superficial concrete layer with thickness of about 10-40 mm presented evidence of chemical attack of concrete by magnesium salts and sulphates from seawater.

3. Repair methodology

In view of the high deterioration level of the dock, the extensive contamination of the concrete by chlorides as well as the deterioration of the superficial layer of the concrete by chemical attack, a repair methodology has been selected. This consists of the total removal of the superficial concrete layer and its replacement by a new high quality concrete surrounding the reinforcement. The minimum cut depth of the concrete is 10 cm, thus ensuring the cut of the concrete of at least 2 cm beyond the reinforcement. The specified concrete cover is 6 cm.

The owner intended a service life for the work of about 20 years. This period can be ensured using this repair methodology, as indicated by the results obtained through monitoring of repairs performed on other docks of the shipyard in 1990-92.

The specifications imposed as regards the concrete to be adopted for repair were as follows:

Composition:	$A/C \leq 0.4$		
	Cement	$350 \leq C \leq 400$ Kg/m^3	
	Micro silica	$15 \leq C \leq 30$ Kg/m^3	
Properties:	Water permeability	$d \leq 20$ mm	(ISO 7031)
		$K_w \leq 1 \times 10^{-12}$ m/s	
	Capillary absorption	$a \leq 0.1$ mm/min$^{0.5}$	
		$h \leq 10$ mm	(4 hours)
	Chloride diffusion	$D \leq 1 \times 10^{-12}$ m^2/s	

The specification of these properties is related with the transport mechanisms into the concrete that occur inside the dock: permeation, absorption and diffusion. A monitoring system has been specified for the repair works to follow the evolution of the penetration of the critical chloride content and assessing the parameters having highest influence on the corrosion rate: resistivity, temperature and concrete moisture.

Analysis of the Ultimate Response of Externally Prestressed Beams

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ABSTRACT

The deterioration of existing bridges due to progressive structural aging, severe weathering conditions, and corrosion of reinforcement has become a major problem around the globe. Also, many of these bridges are structurally deficient due to fatigue damage caused by increased traffic volume and truck loads beyond these estimated during the time of their design. To remedy such problems, several technologies and new materials have been developed for strengthening existing structures and have grown recently to occupy a significant share of the construction market. These include the use of high strength steel, continuous fiber reinforced plastics (FRP), carbon fiber reinforced polymer (CFRP) as reinforcement and prestressing material, and FRP laminated products glued to the weak tensile zones of the structural members. Among these technologies, external prestressing has proven to be efficient and cost-effective. Also, because of the economy and ease of maintenance associated with the use of external tendon system, external prestressing has been recommended recently in the design and construction of new segmental bridge structures.

In externally prestressed members, the prestressing tendons are unbonded to the concrete because they are located outside the concrete member. Consequently, the behavior of externally prestressed members is expected to be influenced by the same parameters that are known to influence the behavior of beams with internal unbonded tendons. However, there is one major difference between the behavior of internal and external unbonded tendon systems. That is, unlike internal unbonded members where the tendon eccentricity remains practically constant during the load-deflection response, the tendons in externally prestressed members are free to displace vertically relative to the axis of the beam between the deviator points or between the deviator points and the anchorages. This leads to progressive change in their eccentricity with increasing member deformation to failure and therefore may influence the response as compared to beams with internal unbonded tendons.

It is possible to develop accurate numerical models to predict the entire load-deflection response of externally prestressed members by using constitutive material stress-strain relationships for the concrete and steel and a multilevel iterative procedure to satisfy the deformation and force equilibrium requirements at any stage during the response. These models are however cumbersome, and relatively complex, particularly if only the ultimate limit state characteristics are of interest.

In this paper, a simple and computationally efficient analytical model, based on strain compatibility approach, is proposed to evaluate the nominal flexural strength characteristics of concrete members prestressed using external tendons. The analysis is based on idealized curvature distribution along the member length and makes use of the requirements of force and moment equilibrium as well as compatibility between the strain in the external tendons and their elongation between the anchorage ends at nominal strength. The analysis takes into account the reduction in the depth of the external

tendons with increasing member deformation to failure, and considers most of the important parameters that influence the response. These include: areas of external prestressing steel and internal bonded reinforcement, geometry of applied load, and span-depth ratio of the member.

The validity and accuracy of the proposed strain compatibility analysis were verified by comparing with experimental results of externally prestressed members reported in the literature. The stresses in the external prestressing steel and nominal moment capacities predicted by the analysis were in very good agreement with the experimental results. Also, with its simplicity in application and considerable efficiency in computation, the discrepancy between the results predicted by the analysis and the experimental results is quite identical to the discrepancy that could be obtained using more elaborate, and presumably more accurate, nonlinear analysis methods developed in the technical literature.

A limited parametric study was undertaken using the strain compatibility analysis to evaluate the effect of several design variables on the stress response and nominal moment capacity of externally prestressed members. These include content of tension reinforcement or reinforcing index (between 0.1 and 0.35); geometry of applied load (two-third point loads, uniform load, single concentrated load); deviator configuration (undeviated tendons, deviated tendons); and span-depth ratio of the member (between 10 and 50).

Based on the results of the parametric evaluation, it was found that irrespective of the geometry of applied load, the stress increase δf_{ps} in the external prestressing steel (above effective prestress) decreases with increasing content of tension reinforcement or reinforcing index as expected. However, the magnitude of δf_{ps} is very much dependent on the length of plastic region of the member expected to develop at nominal flexural strength. Accordingly, two-third point loads and uniform loads produce significantly larger δf_{ps} in comparison with single concentrated loads, particularly at low level of reinforcing index. Also, within the practical range of reinforcing index, the magnitude of the nominal steel stress f_{ps} remains below yield, which is similar to the behavior of beams with internal unbonded tendons.

Because the depth of external undeviated tendons decreases with increasing beam deflection to failure, beams with undeviated tendons mobilize significantly lower δf_{ps} and also nominal moment capacity M_n as compared to beams with deviated tendons. Since the nominal deflection produced by two-third point loads is considerably larger than that produced by single concentrated load (larger equivalent plastic region length), the reduction in steel stress and nominal moment capacity associated with the use of undeviated tendons in comparison with deviated ones are much more substantial for two-third point loads in comparison with single concentrated loads.

Also, as the member span-depth ratio increases, δf_{ps} decreases, which is similar to the behavior of beams with internal unbonded tendons. However this trend is negligible for beams loaded with two-third point loads, and it tends to be most significant for beams loaded with single concentrated load; reinforced with low reinforcing index; and having span-depth ratios less than about 20.

Recent Experience of Upgrading Berthing Facilities in Japan

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1. Introduction

While berthing facilities in Japan have been designed with 30 to 50 years of design life spans, approximately 15 or more facilities begin to be upgraded every year before reaching their intended life spans. Most of such upgraded facilities faced the problems associated with insufficient water depth in front of the berth, ageing of materials, lack of cargo handling spaces, containerisation, and so on. This was because they have been designed to service comparatively small vessels and have been equipped with less sophisticated and lighter cargo handling equipment.

To understand the actual upgrading measures to such less valuable port facilities, a nationwide survey has been undertaken and 90 examples have been collected. These examples were analysed with focus on timing, structural and design details, and reasons for upgrade. Beside the upgrade analysis, maintenance and major repair costs before and after upgrading were also analysed and estimated. It is getting more important to estimate the life-cycle cost to decide future maintenance strategy including repair, rehabilitation, upgrade, or demolition. This paper presents the analytical results of upgrade and life-cycle cost of berthing facilities in Japanese ports.

2. Examples of upgrading existing berthing facilities

Among all berthing facilities with more than -4m depth in Japan, a total of 90 facilities in 48 ports upgraded after 1988 were surveyed and collected. Figure 1 shows the time of upgrade after initial construction with the description of structural types. The age group of 20 to 29 years after initial construction showed the largest numbers of upgraded cases and sheet-pile bulkheads accounted for the majority of the upgraded structures. Among all upgraded examples, those upgraded before 30 years after construction were 52% of gravity type structures, 58% of piled platforms, and 76% of sheet-pile bulkheads. It can be concluded that steel structures have more possibility to be upgraded than concrete structures because of possible cause of corrosion. About 30% were taken the reason for meeting the requirements on physical conditions such as loss of strength and large deformation. For functional reasons such as enlargement of vessels and shortage of cargo handling areas was about 55% of all.

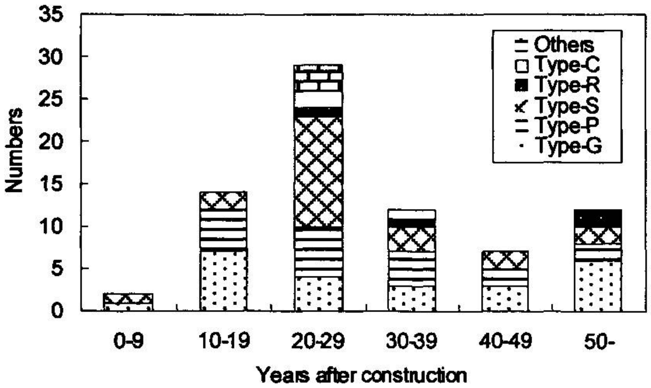

Fig. 1 Timing of upgrading after initial construction

3. Life-cycle Cost Estimation and Evaluation

The relationship between operational years after construction and the average annual maintenance cost is shown in Figure 2. The maintenance cost began to appear after 15 years and increased with operational year. There were two groups

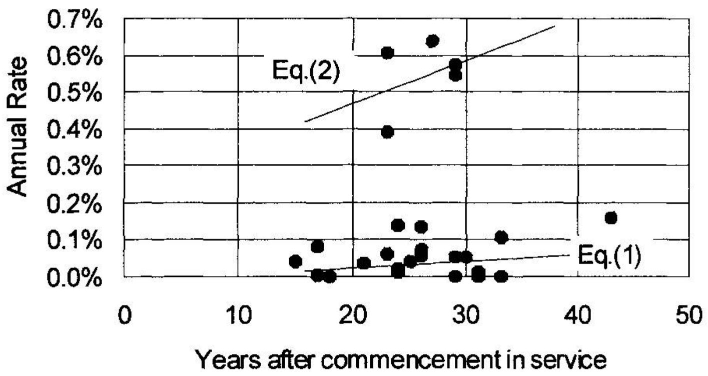

Fig. 2 Annual cost ratio of normal maintenance

of the relationship: higher cost rate and lower cost rate. The higher cost tended to appear in well maintained facilities. The best fit lines, though there are much varied, for maintenance cost was proposed.

4. Life-Cycle Cost and Maintenance Strategy

The life cycle maintenance for port facilities was based on 1)Evaluation of present structural performance and function and 2) Remaining life to be required. When the estimated remaining life is longer than the initial design life, there would be no countermeasures at the time of consideration. In other cases, however, countermeasure to extend the life of facilities should be taken and this should be done according to the life-cycle cost estimation if available.

On the basis of the maintenance strategy, three scenarios were considered here. Scenario 1: leaving this condition until the stress ratio reached 0.9 and after that very high quality countermeasures such as petrolatum and protection cover for steel and resin lining for concrete deck would be installed. Scenario 2: repairing or strengthening to some extent such as reinforced concrete covering for steel and replacement of deteriorated part of concrete and the same corrosion rate would be expected. Scenario 3: installing countermeasures enable to decrease the corrosion rate. The required life was set to be 60 years after construction and normal maintenance happened after 15 years. Steel piles and concrete deck were considered to need countermeasure at 40 years and 30 years after construction respectively.

Figure 3 shows the life-cycle cost calculation with the three scenarios. Scenario 3 gave the most economical results at the end of life span. Accumulating cost data and establishing its database will enable to provide more accurate cost estimation. Life-cycle cost estimation is one of the sophisticated decision support tool for establishing maintenance strategies for port facilities. The life-cycle cost estimation was proposed for providing a prospective answer for future maintenance strategy.

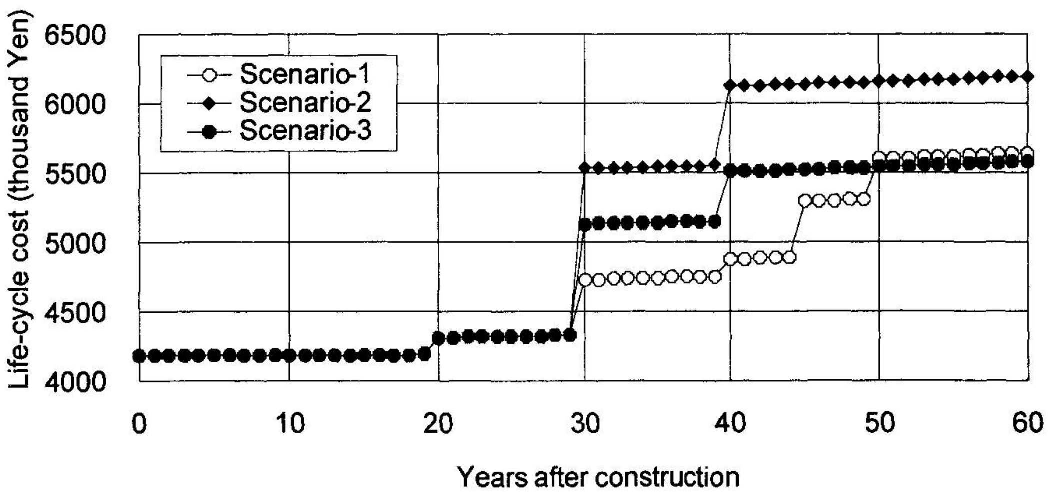

Fig. 3 Life-cycle cost calculation with three scenarios

Reinforced Concrete Deep Beams Strengthened in Shear

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Summary

Results of tests in nine reinforced concrete deep beams, simply supported and submitted to two top point loads (seven strengthened in shear) are shown. The seven strengthened beams were 800 x 150 x 1600 mm with f_c around 50 MPa, designed with insufficient shear reinforcement and loaded previously until service load. The main strengthened variables were the type, position and amount of the reinforcement positioned. The strengthening reinforcement were positioned with special mortar in ducts sawn on the surface of the beam. The results showed that the strengthening technique adopted worked properly without any problem of anchorage of the glued reinforcement. Besides this, the strengthened beams reached considerable higher ultimate loads and behaved very well in comparison with the two reference ones.

Keywords: Deep Beams, Reinforced Concrete, Repair, Shear Strengthening.

1. Experimental Program

The main objective of the research was to investigate the behaviour of reinforced concrete deep beams strengthened in shear, after being loaded to service load, in comparison to a monolithic beam with the full reinforcement. This work followed experimental investigations done at the University of Brasília in deep beams in 1994 and in strengthened columns in 1996, under two of the main research interests of the Structures Group, *"Experimental Analysis of Structures"* and *"Pathology and Recuperation of Structures"*. The investigation was basically done in four steps: A reference beam with insufficient shear reinforcement was initially tested until rupture (1ststep). It was adopted after this first test about 70% of the ultimate load was the level that all others beams would be loaded (2ndstep) before strengthening (3ndstep). These beams and beam 2M (monolithic) were then tested up to failure (4thstep). The strengthening reinforcement were positioned with high performance mortar in ducts sawn on the surface of the beam. The behaviour of the beams tested were analysed through the strains of the flexural and shear reinforcement, and of the concrete, by the horizontal and vertical deflections, by the developing and widths of the cracks, and by the ultimate load and mechanism of rupture. The shear reinforcement of the reference beam (1) and for all the beams to be strengthened was similar, an orthogonal mesh with smooth bars. The strengthening shear reinforcement were also of smooth bars, as horizontal and vertical bars (2/S to 4/S, 6/S and 8/S), and of inclined reinforcement (5/S and 7/S). Figures 1 and 2 present the detailing of the beams 5/S and 7/S respectively. The dotted lines in these figures are the initial reinforcement and the continuous line are the strengthening reinforcement positioned later. Beams 2/S, 3/S, 4/S, 6/S and 8/S were strengthened with the same amount of orthogonal reinforcement, but with different detailing.

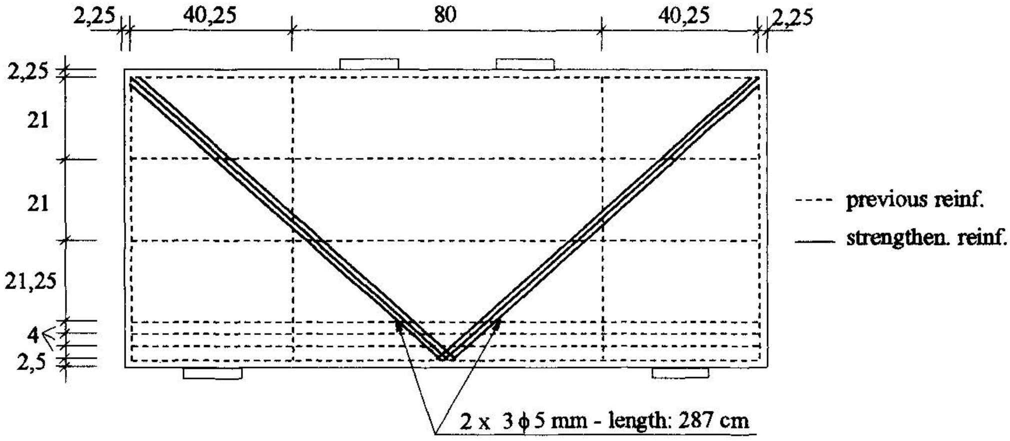

Fig. 1 – Beam 5/S strengthening reinforcement detail (cm)

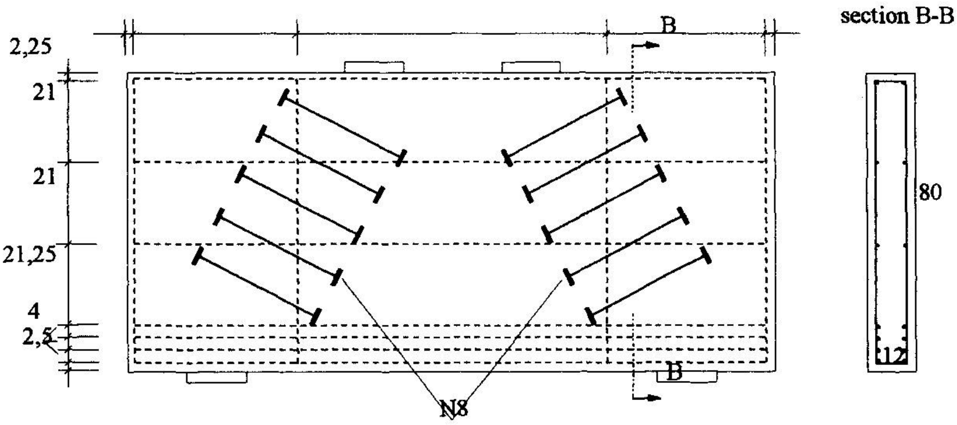

Fig. 2– Beam 7/S strengthening reinforcement detail

Repairing to Extend the Lifespan: the Multi – Strategy Criteria



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The paper aims to describe the story of the "APS Terminal Petroleiro Access Viaduct", located at Sines, south of Portugal, some ten metres distant from sea, a prestressed reinforced concrete structure with a very low thickness of concrete cover that has been exposed, since its construction, to a heavily aggressive marine environment.

The careful survey that has taken place before the rehabilitation works showed that the main cause for the decay of the concrete in the viaduct was the reinforcement corrosion due to the high chloride content at the reinforcing steel, as shown in the Table bellow:

Minimum reinforcement cover		Depth of carbonation	
Piers	1,8 – 3,0 cm	Piers	0,1 – 0,6 cm
Deck	1,1 – 2,3 cm	Deck	0,1 – 2,3 cm
Abutment	2,1 – 3,5 cm	Abutment	0,5 – 1,8 cm
Chloride content referred to weight of cement around reinforcements		**Probability of active corrosion**	
Piers	0,11 % - 1,26 %	Piers	Minimum
Deck	0,42 % - 1,12 %	Deck	50 % - 95 %
Abutment	0,52 % - 0,81 %	Abutment	Minimum to 50 %

The adopted repair system has favoured the removal of the aggressive substances from the concrete (desalinisation) instead of using surgical treatments (concrete removal), that has been used only when indispensable (spalling).

As this work involved removal of a lower volume of contaminated concrete than initially foreseen as well as the elimination of the cause for the deterioration of the structure, the final cost was much smaller than the amount initially foreseen.

Desalination of concrete is an electrochemical treatment intended to stop and avoid corrosion of steel in reinforced concrete when it is contaminated by chlorides and to reduce the chloride content to permissible values by applying an electric field (a current with a density nearly equal to 1 A/ m_) between the concrete reinforcement and an external anodic system, a titanium mesh fastened to the structure by insulating nails in plastic material and coated through projection of cellulose fibres, forming the electrolyte as the fibres are kept damp.

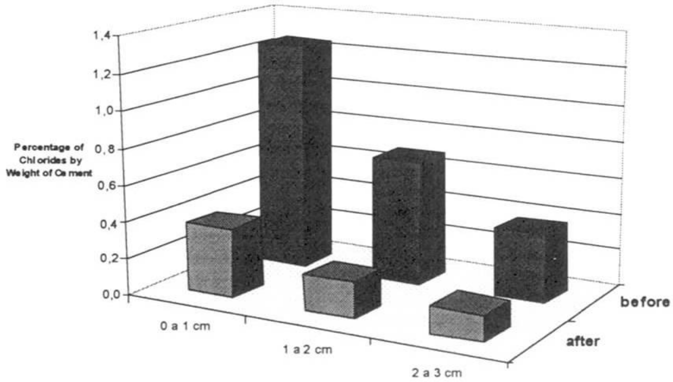
During the treatment, the negative chloride ions migrate to outside the concrete whereas an electrolysis phenomenon occurs on the reinforcement surface to produce alkaline environment and re-passivation. Moreover, the attraction of positive ions towards the reinforcement zone leads to compactness and imperviousness of concrete around the reinforcement steel.

Since the reinforcement cover was less than 10 mm in some zones, the thickness of the cover was improved by shotecreting (dry mix procedure), in order to ensure a minimum cover of 40 mm. The defective zones of the structure were repaired with similar procedures.

An automatic system has been used to monitor desalination, which includes a computer-controlled rectifier unit controlling each cell. In real time the control system informs the operator of any failure or of anomalies that can be solved so that the process may reliably and efficiently go on. This system also allows the operators to control the indicators of the evolution of the process, such as resistance, intensity of current, quantity of Amperes $\times$ hour per m_ of reinforcement so far supplied to the cells.

Each desalination cell shows differences in concrete resistivity (depending on moisture in concrete or on the extent of repaired areas), reason why drawings are previously prepared to indicate location of the cell, its designation and the zones repaired are mapped as well as the control points for chloride content. This information is of utmost importance during the work, to permit to interpret both messages pointing to error of the system and the values that indicate the evolution of the treatment.

About once in a week, the structure's concrete is sampled for the determination of the chloride content and to support de decision of concluding the treatment, if the required level (less than 0,3 % of the cement weight) is achieved. Now, desalination is in progress and the removal of chlorides has already been successfully performed on 1400 m_. Values of about 0,2 % of cement weight were obtained at reinforcements, against the initial values of about 0,6 % (see figure bellow).



Subsequently it was decided to perform the protection of the structure by applying an appropriate painting structural rehabilitation, thus characterising a multiple strategy of interventions regarding the durability of the construction.

Rehabilitation of the Highest Wooden Telecommunication Tower

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Summary

The unique telecommunication tower, over 110 m high, was built in 1933 as a spatial truss completely from larch wood. All connections were made with brass bolts. During 65 years of continuous service the tower was examined and protected several times but there were also long periods without proper maintenance. In 1998 the structure was carefully inspected, measured and checked by calculations. Imperfections in geometry of truss joints as well as damages in wooden members have been recorded. On the basis of tests of material specimens and computer analyses the range of necessary strengthening has been assumed. The tower was old enough to be treated as a monument of technology, so, the methods of repair, strengthening and protection were limited. The specific method of strengthening by means of carbon fibre polymer strips was introduced in the most endangered members of the structure. General protection works have been used for the entire tower.

Keywords: space structures; strengthening; towers; trusses; wooden structures.

1. Introduction

Among a dozen or so wooden telecommunication towers over 100 m high, erected in Europe before the Second World War, the tower in Gliwice, Poland, survived as the only one to the present. With the height of 110.7 m this is the highest wooden tower in the world. In the long periods, at the time of the Second World War and just after the War, the maintenance of the structure was poor or none. Apart from the influence of natural climatic conditions typical for the middle Europe the tower has been all the time subjected to specific impact of polluted atmosphere (acid rains) due to the neighbourhood of heavy industry and chemical plants.

2. Description of the structure

The tower is a spatial truss structure with the size of square base 19.8 m × 19.8 m. The main corner members were arranged according to parabolic curves, so, the four external trusses are not flat but curved along the height (Fig. 1). The four platforms are located on levels 40.0 m, 55.3 m, 80.0 m and 109.7 m; on these levels the spandrel beams and horizontal trusses create the main bracing of the structure. The structure was built from larch wood of very good quality. Cross-sections for all bars were precisely selected and differentiated according to design. In 1998, after 65 years of constant service the first complete and precise measurement was done. The exact initial shape of the tower was not known, so, the present results could be compared with designed shape only.

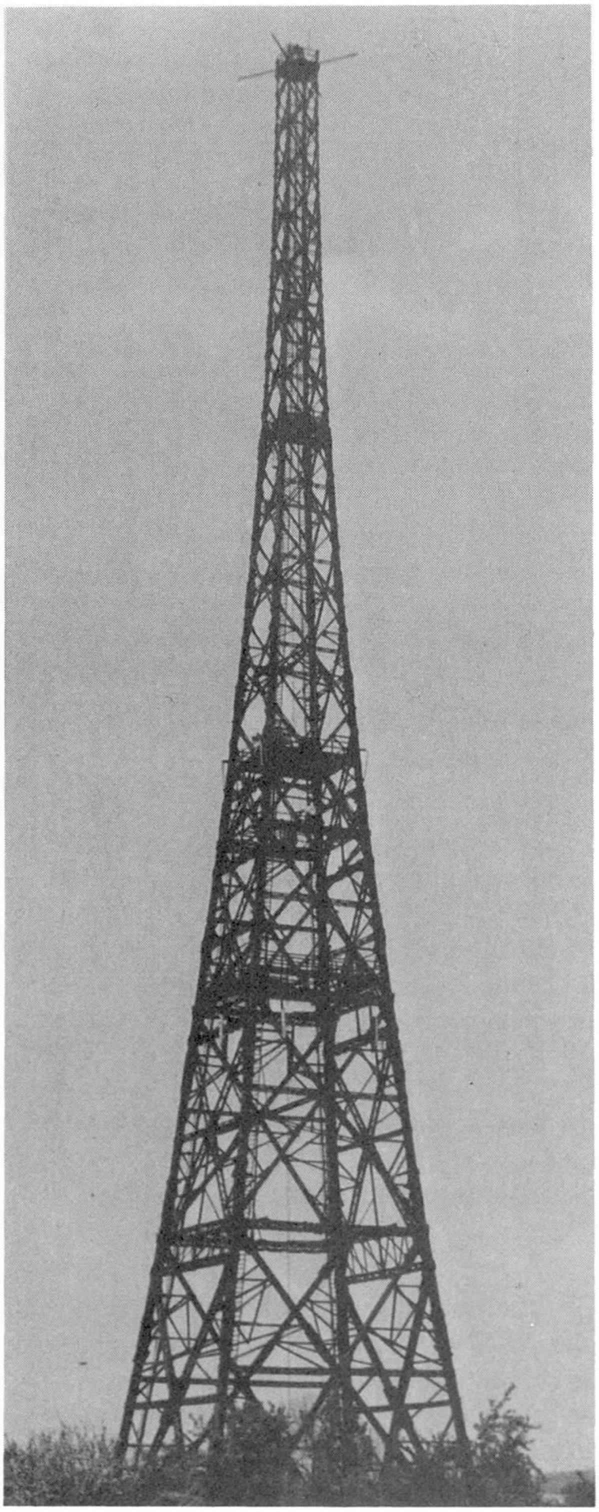

Fig. 1 General view of the tower

3. Rehabilitation program

According to inspection, measurements and calculation results the following works were recommended:

(a) Successive reconstruction of joints and filling of all slits and openings by protective injection;
(b) Strengthening of bars endangered with longitudinal slits – main compressed bars were indicated to be strengthened with transversal carbon fibre polymer strips;
(c) Impregnating of all members after cleaning surfaces.

The selection of methods for reconstruction and strengthening was restricted within narrow limits because, since 1964, the tower has been announced a monument of technology under care of conservation services. Therefore, the shape and appearance of the tower should be remained, as far as possible, without changes.

4. Conclusions

The aim of action containing inspection, measurements, control calculations and accompanying tests was the selection of proper process for repair and rehabilitation to extend the lifespan of the 65-years-old wooden telecommunication tower.

Because the tower has to be treated as a monument, not only technological but also aesthetics aspects have been considered. It was the reason of introduction of unique method for transversal strengthening of wooden bars significantly endangered with slits. Carbon fibre polymer strip glued with selected epoxy glue has been selected.

Rehabilitation of Masonry Buildings and Monuments

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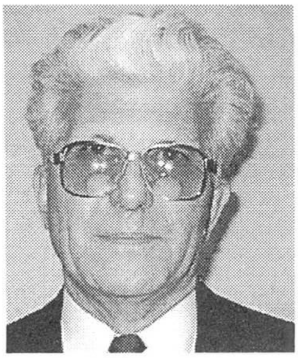

Ramiro Sofronie, born 1936, received his degree in civil engineering from Technical University of Timisoara in 1958 and his doctoral degree from Technical University of Bucharest in 1970. He is teaching Strength of Materials and Reinforced Concrete to University and is heading the UNESCO Chair in Integrated Rural Development *ECOLAND*.

Summary

The poster brings new proofs, obtained by static and dynamic testing, for using the innovative method of reinforcing both ancient and new masonry members with polymer grids. Synthetic reinforcement replaces masonry lack of ductility and enhances its intrinsic resources of strength. In this way *the non-homogenization* of masonry buildings and monuments with RC and steel members is avoided. The poster also presents the non-destructive method impact-echo for discovery and location the hidden damages and calls for using the language of official provisions in all rehabilitation works.

Keywords: confining, jacketing, preserving, reinforcing, remodeling, repairing, restoring, retrofitting, shaping, strengthening.

Rehabilitation means reconstruction or renewal of a damaged building to provide the same *level of function* that the building had prior to the damage. When from all possible functions only the structural function is concerned then it uses the term of retrofitting, and it includes strengthening, repairing and remodeling. When only monuments and historical sites are concerned or the meaning of the two keywords is combined then a more general and holistic concept of *restoring* is used.

Sometimes rehabilitation works change either the original shapes of masonry buildings or spatial distribution of their masses. The influence of such changes on building equilibrium in gravitational field is checked with the aid of gravity or mass center. In seismic zones the other intrinsic point of major interest is rotation or shear center.

For safety reasons the relative distances between the two centers, before and after rehabilitation, should be carefully checked. Few damages are visible, most of them are in a way or other hidden. They should be located and accurately mapped. Ancient builders used light hammers for this purpose. Many of still existing masonry buildings have been checked by auscultation. Nowadays, among non-destructive methods the Impact - Echo proved efficient and easy to handle. It was produced by Cornell University and is based on propagation of stress waves through solid members. Internal flaws and voids reflect stress waves, and the crossing times are registered. By knowing the speed of waves in the checked material the positions and dimensions of flaws are immediately identified.

During last years new retrofitting techniques have been checked. Most of them are based on different kinds of fibers. Polymer grids are also convenient for their simplicity of applying and low cost. In the new parts of masonry buildings they are inserted in the horizontal layers of bricks, while the old parts of those buildings should be covered or confined. In both cases, with the aid of polymer grids, the original homogeneity of masonry is preserved. The relative distance between the two intrinsic centers is also maintained, while by replacing from mortar and plasters the cement with lime both aesthetics and comfort of masonry buildings is highly increasing.

The program of static testing continued with six wall panels of reinforced and confined masonry. Three of them were tested to axial compression, while the others to diagonal tension. Compression testing shows that $\sigma_{el.}/\sigma_{max} = 0.76 > 0.33$, $\sigma_u/\sigma_{max} = 0.87 > 0.66$, and the ductility $\varepsilon_u/\varepsilon_{el.} = 8.44$ since $\varepsilon_u = 13.5$ ‰ >> 3.5 ‰. Diagonal tension testing shows that $\tau_{el.} \approx \tau_{max}$, $\tau_u = 0.5\,\tau_{max}$, and the ductility $\gamma_u/\gamma_{el.} = 19.6$ with a drift $\gamma_u = 23.2$ ‰. The relation between elasticity modules is $G_{el} = 0.45\,E_{el}$, close to the value recommended by EC6 ENV 1996-1-1: 1995, clause 3.8.3, and for Poisson ratio $\upsilon = 0.12$ it was found.

The program of dynamic testing started several years ago with a two-stored masonry building. The model scaled at 1:2 was tested on ISMES' shaking table in Bergamo, Italy, and testing details together with the main results obtained have been recently presented in Paris. After the first series of testing the masonry model was dramatically damaged. Most of the cracks have developed between the openings for windows and door as well as near the corners at the bottom and top of model, being caused by shearing stresses. During retrofitting the model the cracks have not been treated in any way, neither by filling them with mortar. The two zones strongly cracked have been simply covered with polymer grids, well fixed with stainless nails and then plastered. Tested again on the same shaking table according to the same testing program the retrofitted model resisted to a much higher seismic intensity. The program was stopped when at the surface of the two belts vertical cracked appeared. No more inclined cracks were observed. The reinforced plaster works like in the static test. These new cracks have been clearly caused by the tension stresses developed in the two belts. In this way it was definitely proved that polymer grids replace masonry lack of ductility and enhance its intrinsic resources of strength.

There are strong reasons, proved by static and dynamic testing, to use polymer grids for rehabilitation of masonry buildings and monuments. The proposed method allows preserving the original homogeneity of masonry with the lowest financial effort, maintains the relative distance between the two intrinsic centers of gravity and rotation, while both aesthetics and comfort of buildings are increasing by replacing cement with other binders. For rehabilitation of masonry buildings and monuments one recommends to confine the damaged structural members, while in the case they are replaced to with new ones to reinforce the masonry in horizontal layers between bricks before confining.