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Corrosion Protection by Means of Dehumidification

Matthew L. BLOOMSTINE
Project Manager
COWI AS
Lyngby, Denmark

Matthew L. Bloomstine, born 1955, received his engineering degree from the Technical Univ. of Denmark in 1987. He has specialised in inspection, maintenance and rehabilitation of major bridges.

Frank RUBIN
Civil Engineer
Danish Road Directorate
Copenhagen, Denmark

Frank Rubin, born 1942, received his civil engineering degree from the Technical Univ. of Denmark in 1966. He is Area Manager for Operation and Maintenance in the Bridge Department.

Ejgil VEJE
Department Head
COWI AS
Lyngby, Denmark

Ejgil Veje, born 1954, received his engineering degree from the Technical Univ. of Denmark in 1981. He is the Head of Department for Maintenance and Rehabilitation of Major Bridges.

Summary

Painting has traditionally been used to protect steel structures from corrosion. Over the last 30 years dehumidification has been adapted and implemented as a means of corrosion protection for enclosed steel surfaces in bridge structures. For this use dehumidification has been proven to be superior in all respects, i.e. technically, economically and environmentally. It has been chosen as the sole means of corrosion protection for the internal surfaces of steel box girders and other components on many major bridges in countries over the entire world.

The paper presents descriptions and examples of dehumidification systems, the advantages of dehumidification, key technical and economical figures, as well as experience from operation and maintenance.

Abstract

Many of the main components of major bridges are steel structures. In order ensure a long service life and provide an appropriate level of safety, these structures must be protected from corrosion.

Corrosion protection has traditionally been provided by means of surface treatment, i.e. blasting and painting. In the course of the last 30 years an alternative method of corrosion protection for the internal surfaces of steel structures has been developed, implemented and proven. This method is dehumidification and is based on the fact that steel does not corrode when the relative humidity is below 60%. Dehumidification has been proven to be superior to painting in all respects, i.e. technically, economically and environmentally.

The most widespread application for bridges is the protection of the internal surfaces of closed box bridge girders. However, there are many different applications on bridges and one of the most recent developments is dehumidification of the main cables on suspension bridges. Dehumidification systems are implemented in new bridges and in existing bridges, which may have insufficient protection or need renewal of corrosion protection.

The following concerning dehumidification is presented in the paper:

- Corrosion problems
- Principles of dehumidification and typical equipment
- Advantages of dehumidification - technical, economical and environmental
- Experience from major bridges including bridge girders and other bridge structures

- Key technical, economical and service figures
- Operation and maintenance experience

Corrosion is mainly dangerous in the following two ways:

- Severe corrosion can lead to an appreciable loss of the thickness of the plates in the cross section and reduce the load carrying capacity. If the corrosion is extreme enough or occurs at especially critical areas it can lead to collapse of the structure.
- Lighter corrosion on fatigue affected areas, such as the deck of a bridge girder, can significantly reduce the fatigue resistance leading to premature cracking and a reduced service life.

The concept of dehumidification has been known for many years and it has been successfully applied to a wide range of applications. A dehumidification plant, such as used in bridge structures, is a well know and reliable means of keeping the relative humidity in a closed room under an acceptable level. The use of dehumidification for the corrosion protection of enclosed steel surfaces in bridges has been pioneered by COWI over the last 30 years. During this time the dehumidification concept for bridges has won international acceptance and is well on the way to becoming a world-wide standard.

The sorption method of dehumidification is to the best of our knowledge the only method used for bridge structures. A sorption system contains a rotor which is built up of many small pipes, which are coated with a sorbent, most commonly lithium chloride. The process air is forced through the rotor and its moisture is absorbed under this process, resulting in dry air. The rotor turns very slowly, allowing time for the process. On the opposite side of the rotor heated intake air is blown through, which dries out the sorbent coating. This air becomes moisture laden and is subsequently discharged. The other method of dehumidification is condensation. This method has a number of limitations and drawbacks, which make it unsuitable for use on bridge structures.

Dehumidification is technically superior to traditional painting, as it provides 100% protection and does not have the many weaknesses and execution requirements that painting does. Dehumidification provides great initial savings as compared to painting and the operation and maintenance costs are also much lower. Dehumidification is also more environmentally friendly, as it does not have the short term and long term effects that painting does.

The use of dehumidification as corrosion protection for enclosed steel surfaces in major bridges was first applied to the box girder of the Little Belt Suspension Bridge, Denmark, which opened in 1970. Since then dehumidification of steel box girders on bridges has become the standard method of corrosion protection in Scandinavia. During the last 30 years the dehumidification concept for bridges has won international acceptance. Outside of Scandinavia it has also been applied in England, France, Germany, Japan and other countries.

The dehumidification systems of the Little Belt Suspension Bridge and the Faroe Bridges, both in Denmark, have been in operation respectively for 29 and 14 years. The Little Belt Suspension Bridge has dehumidification plants in the 1,080 m long box girder as well as both anchor houses. The Faroe Bridges have plants in the 1,596 and 1,726 m long box girders, in all four abutment rooms and in the two cable anchor boxes on top of the pylons. The individual dehumidification plants are serviced once a year, which entails a check of the individual components and control of effectiveness. As of yet only wearing parts have been repaired or replaced. No entire plants have been replaced and this is not expected to be necessary.

Low Maintenance Cable-Supported Structures - A New Concept

J.P. FUZIER
Scientific Director
Freyssinet International
Vélizy, France

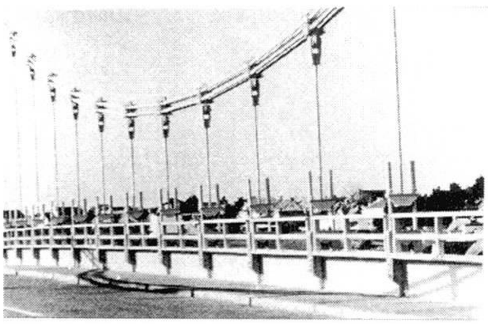
J. STUBLER
Technical Director
Freyssinet International
Vélizy, France

P. JARTOUX
Research & Development Director
Freyssinet International
Vélizy, France

Summary

Replacement of cables had to be carried out recently on bridges in Argentina (General Belgrano and Zarate - Brazo Largo bridges) and in France (Tancarville bridge). A huge programme has been undertaken in the United States. Was the cable technology used on these structures meeting this requirement ? Will the Williamsburg suspension cable be replaced one day ?

This paper reviews the problems encountered which had to be solved to replace the cables and proposes a new concept of structural cables together with a new layout and installation methods for cable-supported bridges, providing excellent corrosion and fatigue resistance associated to a low-maintenance cost.



The Lorois bridge

1. Cable replacement

The replacement of suspension cables will require a sequence of delicate operations in order to ensure the stability of the structure at all times and to maintain the traffic as much as possible during the works. As an example is the sequence of the Lorois suspension bridge (France).

Very often the pipes were injected with cement group to fill the voids between wires and jacket. It is highly uncertain whether this fill material completely fills all voids and prevents moisture from the curing cement grout, from natural condensation or penetrating into the pipe jacket through external cracks from attacking the wires. The effectiveness of grout protection is further questioned because of its shrinkage-induced cracking and the differential movement between steel wires and cement grout due to vibration and stress and temperature-induced length changes. Several failures have been observed such as :

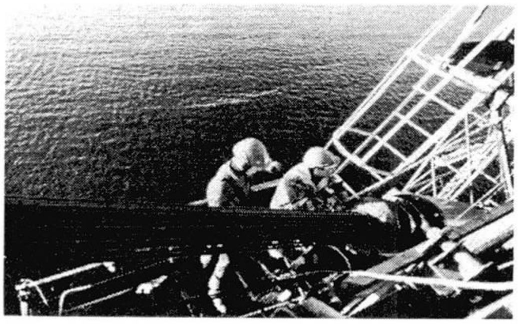
- General Belgrano bridge (Argentina) : 80 locked coil strands 78 to 116 m long.

- Zarate – Brazo Largo bridges : Cables Hi-Am type are made of 7 mm parallel wires encased in a polyethylene duct and cement grouted.

2. New cable technology

The new cable technology meets the following requirements which are of the greatest importance for building durable structures :
- high fatigue resistance, high stiffness and mechanical strength
- excellent corrosion protection
- simplicity of installation
- easy maintenance and replacement without any traffic disruption.

The strands are individually protected as follows :
- hot dip galvanisation before wire drawing ;
- extrusion around the strand of a high density polyethylene sheath (i.e. 1.5 mm thick minimum) after coating the wires with wax.



Cable installation

The modern technology which is used on all the major cable-stayed bridges (Normandie, Vasco de Gama, Ting Kau and Øresund) provides the facilities for maintenance which are requested by the owners :
- the system is a perfectly reversible assembly ;
- placement and arrangement allow for easy inspection ;
- monitoring of the cable force is always possible ;
- detensioning, dismantling and replacement are done without any traffic disruption.

Design of Parking Structures for Reduced Maintenance

Predrag L. POPOVIC
Vice President
Wiss, Janney, Elstner Assoc.
Northbrook, IL USA

Predrag L. Popovic, born 1944, received his structural engineering degrees from the University of Belgrade, and the Illinois Institute of Technology. His work included investigation, load testing and designs of repairs for existing structures, including over 200 parking garages.

Summary

Unique characteristics of parking structures affect their design and maintenance. Moving loads and severe environmental exposure require that special durability features be provided in the design of parking structures. Time tested good design practices and internal and external protection methods are available to the designers to minimize future maintenance of parking structures.

Keywords: parking structures; maintenance; durability; design.

1. Special Characteristics of Parking Structures

When not enclosed, parking structures are subject to ambient weather conditions, which may vary widely based on the geographical location. In cold climates, they are often exposed to snow, ice and water, and to corrosive action of deicing salts. Unlike a bridge deck, the inside of a parking structure is not rinsed by rain, and its exposure to chlorides may be aggravated by poor drainage.

The parking structures are primarily subjected to the loads from moving vehicles and their roof levels exposed to weather similar to bridge decks. Since they are frequently very large in plan view, they experience greater volume changes (temperature, shrinkage and creep) than the enclosed structures, which are usually smaller and are exposed to more uniform temperature, humidity and moisture.

2. Commonly Encountered Problems

The most common types of deterioration and undesirable performance in the parking structures are the corrosion of reinforcing, cracking, spalling, freeze-thaw damage, leakage and ponding of water.

When the protective concrete cover is reduced or when the chlorides are present in sufficient quantities, the reinforcing steel will begin to corrode. The volume of corrosion by-product (rust) is many times the volume of the original metal. This increased volume results in an expansive force which causes the concrete to fracture. Also, the loss of cross-sectional area of reinforcing will reduce the structural capacity of concrete elements. When the corroding bars are closely spaced, horizontal fractures may form parallel to the surface, but remain invisible on the surface. This subsurface fracture creates a concrete delamination at the level of reinforcing steel. Finally, the delamination will spall creating potholes on the top surface or a large piece of concrete may fall from the underside of the deck.

Many problems observed in parking structures are related to inadequate considerations of volume changes. They include variable height columns due to sloping ramps, tying structural floor to the stair or elevator shaft, nonfunctional sliding bearing joints for the floor beams and at expansion joint locations, shortening of the first supported floor framing in relation to its fixed foundations, differential movements of the different areas of the structure built partially below grade, etc.

3. Good Design Practices

The structural system should, in addition to its load bearing function, primarily be designed to minimize the cracking and to accommodate expected volume changes. Properly spaced and sealed expansion joints will allow for the movement of the structure as a whole and will not allow the water leakage through the joints. Post-tensioned concrete decks may also have pour strips, which have to be continuous vertically and horizontally through the entire structure.

In addition to a required concrete strength, low permeability of concrete will help it resist the penetration of water, chloride and oxygen. Silica fume (microsilica) concrete is more dense than regular concrete and is known to be more resistant to chloride intrusion. The proper concrete mix for the garage slabs should have a water-cement ratio of 0.40 or less and its aggregates should not be porous or reactive. Where freeze-thaw protection is required, the concrete should be air-entrained by adding an air-entraining admixture to the concrete mix. However, calcium chlorides and admixtures containing calcium chlorides should not be used in concrete for parking structures.

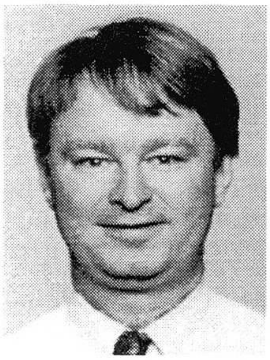
Parking structures with initially built-in protection systems are more durable and have significantly less future maintenance. The internal protection measures include coating of steel reinforcement, protection of post-tensioning tendons, and corrosion inhibitors in concrete mixes. One of the most important measures for reduced maintenance is to specify an appropriate concrete cover over reinforcing bars, particularly in the top of the floor slab. The external protection systems include application of concrete sealers, traffic bearing membranes, and joint sealing systems, including special expansion joint seals.

4. Conclusions

The unique loads and characteristics of parking structures require special design features to reduce their future maintenance. In addition to selecting an appropriate structural system, a proper material selection, specifying adequate concrete cover for steel reinforcement, and installation of internal or external protection measures will reduce future maintenance. Also, the cost of future maintenance will be minimized when a regular periodic maintenance program is implemented.

Deflection Monitoring of Prestressed Concrete Bridges Retrofitted by External Post-Tensioning

Olivier BURDET
Dr
Swiss Fed. Inst. of Techn.
Lausanne, Switzerland



Olivier Burdet, born 1960, is a lecturer in reinforced and prestressed concrete. His research interests include serviceability, bridge design and monitoring techniques.
He is a member of the Publication Committee of IABSE SEI Journal.

Marc BADOUX
Prof. Dr
Swiss Fed. Inst. of Techn.
Lausanne, Switzerland

Marc Badoux, born 1961, is an assistant-professor of Structural Engineering. His research interests include the serviceability behaviour of concrete structures, high performance concrete applications and the seismic protection of existing buildings

Keywords: prestressed concrete, in situ measurements, bridge monitoring, long-term deflections, bridge retrofitting, external post-tensioning

Abstract
Additional post-tensioning has become one of the leading techniques for the retrofitting of bridges, in particular prestressed concrete box girder bridges. This paper describes the use of external post-tensioning for the retrofitting of three Swiss highway bridges. These bridges are variable depth prestressed concrete box girder bridges built between twenty and thirty years ago by the balanced cantilever method. Monitoring of these bridges long-term deflections showed that the downward girder deflections were large and did not stabilise as expected. In all three cases, the decision to retrofit was governed by serviceability considerations, namely deflection control, rather than structural safety considerations. External post-tensioning has proven well suited for this type of intervention. Findings of the long term monitoring of the bridge deflections are presented, with an emphasis on the effect of the additional post-tensioning on the bridge behaviour in the years following the retrofit. The situation of the Fégire Bridge is described briefly in this abstract.

Example: Fégire Bridge

Construction of the cast-in-place Fégire bridge was completed by the balanced cantilever method in 1979. The bridge is 512 m long, its three main spans are about 107 m long, and it does not feature intermediate joints. The bridge was retrofitted in 1995 with additional post-tensioning to counter excessive vertical deflections that were not stabilising as expected. The additional post-tensioning is parabolic (fig. 1) and consists of 2x4 cables with an initial prestressing force of 28'000 kN. The additional cables are unusually long since they are anchored at both abutments. The massive anchorage blocks were designed for the case that a future retrofit would double the additional post-tensioning installed to date.

The long-term deflections of the girder of the bridges reported in this paper are monitored with a hydrostatic levelling system. This simple and robust system was developed by IBAP and is operational in over 10 bridges. The measurements of the vertical mid-span deflection of the central span of the Fégire Bridge are presented in fig. 2. They show the instantaneous upward deflection at mid-span (middle span) caused by the post-tensioning is just under 30 mm. The previous downward tendency clearly observable in fig. 2 appears to have been reversed by the additional post-tensioning.

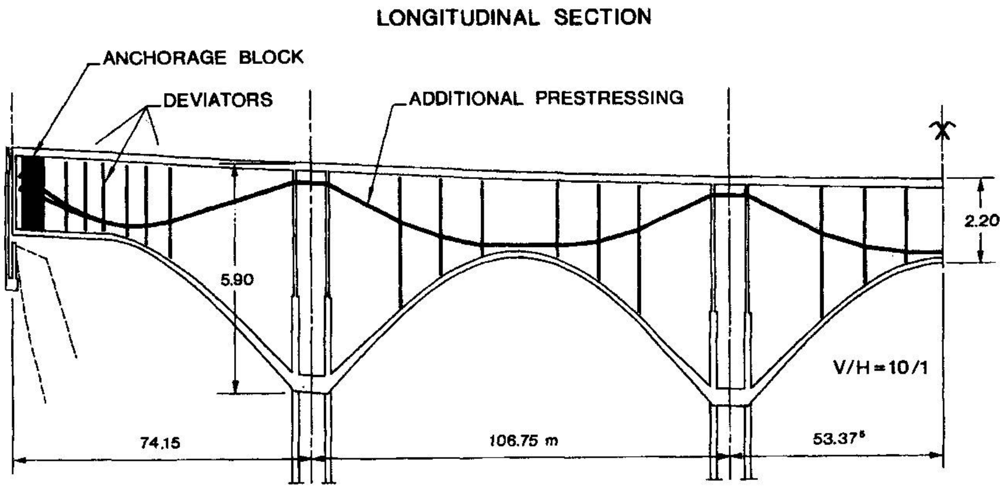


Figure 1: Additional post-tensioning for the Fégire bridge

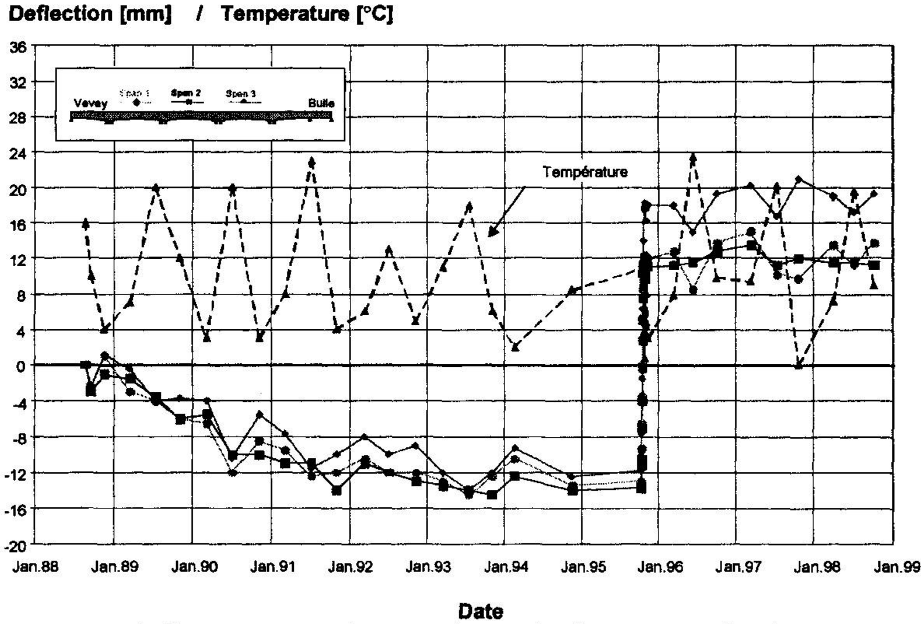


Figure 2: Long-term deflections at mid-span of span 2 of the Fégire bridge

Conclusions

The Lutrive, Chillon and Fégire bridges were retrofitted with additional post-tensioning to counter excessive long-term deflections. on the basis of these examples, it appears that:

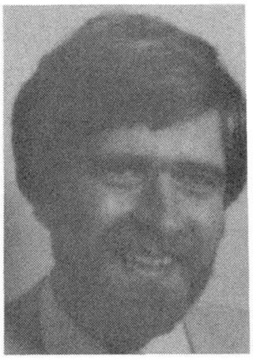
- The addition of external post-tensioning can be a successful retrofitting technique for structures with serviceability limit state problems, such as excessive long-term deflections. It is a relatively flexible technique which can be fine-tuned to achieve the required results. It is for example possible to reverse a downward deflection trend, as shown in the case of the Fégire bridge.

- Long-term monitoring of concrete bridges can be very useful. In the examples reported in this paper, the deflection monitoring data proved extremely valuable for the evaluation of the bridge, and for the selection and design of the retrofitting scheme. It also continues to provide valuable feedback on the actual behaviour of the retrofitted bridges.

Advanced Composite Bridges for the 21st Century

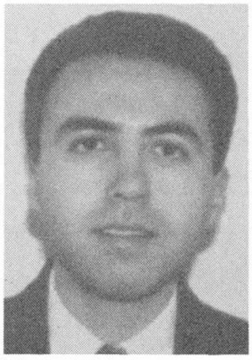
Frieder SEIBLE
Professor
UC San Diego
La Jolla, CA, USA

Born in 1952, received his Ph.D. in 1982 in Civil Engineering from the University of California, Berkeley.



Rigoberto BURGUEÑO
Graduate Research Assistant
UC San Diego
La Jolla, CA, USA

Born in 1969, received his MS in 1994, and currently a Ph.D. candidate in Structural Engineering at the University of California, San Diego.



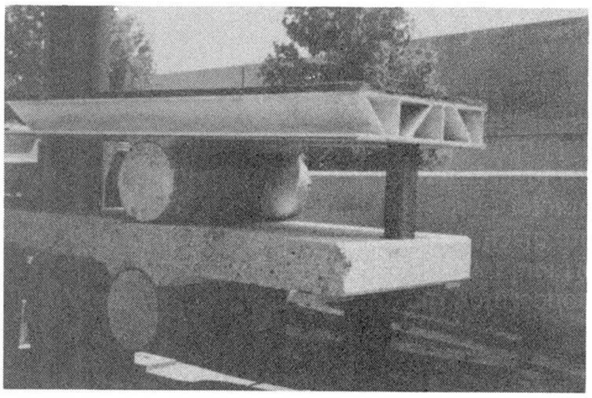
Abstract

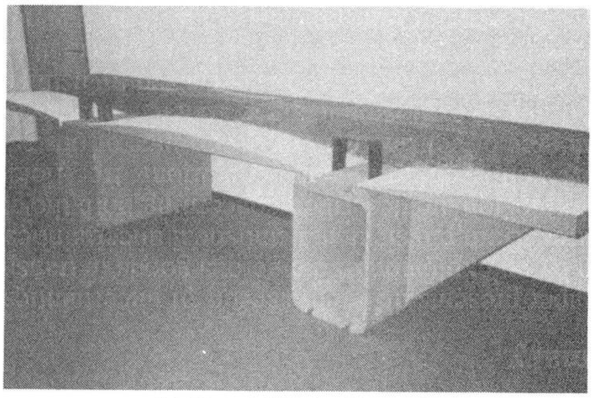
The successful implementation of advanced composite materials, or Fiber Reinforced Polymers (FRP) in civil infrastructure depends on the development of new structural concepts and systems that combine these “new” materials with conventional construction materials such as concrete and steel. Two new design and construction systems for short- and medium-span modular bridges have been under development at the University of California, San Diego (UCSD). The concepts, consisting of carbon/concrete and carbon/glass composite systems, have been applied to columns and girders in the form of FRP tubular members. The research has initially focused on the application of both concepts to beam-and-slab bridges to provide the basis for design and analysis of new modular FRP bridge systems. The research consisted of the concept development, the development of analysis and design tools, and complete structural performance assessment, validated by full structural characterization of the different components through full-scale laboratory testing.

The concrete filled Carbon Shell System concept allows for new structural elements by using prefabricated filament-wound carbon/epoxy thin shells filled on-site with concrete. The shell serves the dual function of reinforcement and stay-in-place formwork for the concrete core. The concrete provides compression force transfer, stabilizes the thin shell against buckling, and allows the anchorage of connection elements. For superstructure components, the concrete filled carbon shells are combined with a structural deck system, which may consist of either a conventional cast-in-place reinforced concrete (RC) slab, or a modular FRP deck system, see Fig. 1a. The connection between the deck and the carbon shell girders is accomplished with conventional dowel technology by embedding shear connectors into the shell system during grouting. In the slab the dowels are either cast directly into the RC deck or anchored in polymer concrete filled sections of cellular E-glass deck systems.

Large-scale experimental flexural characterization and analytical modeling of the concrete filled carbon shell system showed that the concrete core contribution to stiffness and strength is minimal and its function reduces mainly to that of stabilizing the thin shell and allowing connector anchorage. This motivated the development of a new FRP girder system that features thicker walled sections and an anchorage concept for connectors that allows the girder to be used hollow. The new modular FRP bridge system was developed based on the pultrusion and hand-layup of a carbon/glass hybrid material system. The Hybrid Tube System beam-and-slab bridge consists of hollow E-glass beams connected along their tops with a fiber reinforced concrete deck system. The girders are pultruded E-glass rectangular sections with longitudinal carbon reinforcement in the

flanges. The tubes are left ungrouted except for the ends of connection regions. An FRP form panel is snap-locked to the pultruded girders providing a tension tie between girders and the stay-in-place form for a fiber reinforced arch-action-type concrete deck. To provide stiffness to the form panels, the transverse FRP membrane is overlaid with lightweight polymer concrete in a parabolic shape to allow for full construction loads. Prefabricated pultruded snap-in stirrups provide the horizontal shear transfer between the concrete deck and the hybrid tubes. This modular bridge system is depicted in Fig. 1b.





a) Concrete Filled Carbon Shell System *b) Hybrid Tube System*

Fig. 1 Modular FRP Beam-and-Slab Bridge Systems

In order to implement the carbon shell and hybrid tube technology to complete bridge systems and develop appropriate performance-based design guidelines, the experimental evaluation of the critical components and connections is required. A comprehensive experimental program that used a building-block approach to characterize different components as steps towards the development of a carbon shell prototype bridge system was undertaken. The experimental characterization of carbon shell beam-and-slab assemblies for short and medium span bridges was investigated on two full-scale four-point bending tests. The geometry and dimensions for the test units were determined from a design study performed for a two span beam-and-slab bridge.

To demonstrate the application of the developed advanced composite bridge systems presented above, a 137m long cable-stayed bridge supported by a 46m high A-frame pylon was designed to be built at UCSD utilizing advanced composites. The bridge structure is designed for two 3.7m vehicular lanes, two bicycle lanes, pedestrian walkways, and a utility service tunnel. Based on a preliminary study of different structural systems, the solution presented in Fig. 2 was selected. The structure proposes the use of the carbon shell system for the pylon and edge girders with a dual cable plane system. In the transverse direction, partially grouted carbon tube cross-beams are employed, which in turn support longitudinally spanning prefabricated E-glass or RC deck panels.

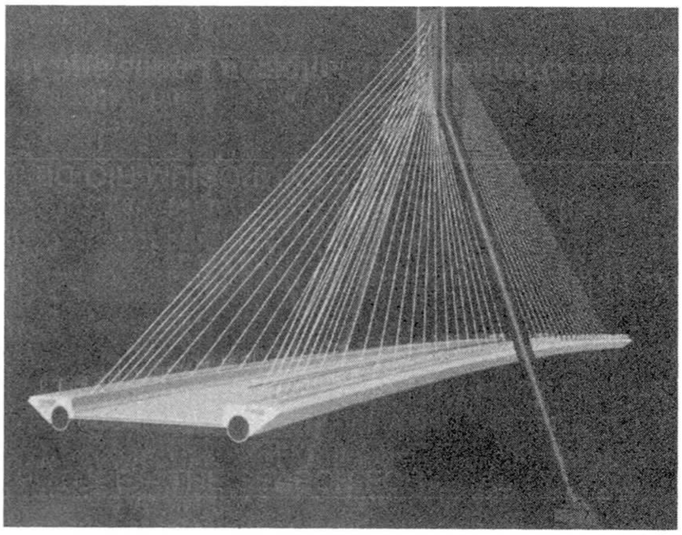
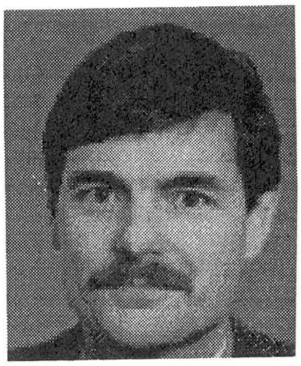


Fig. 2 The I-5/Gilman Advanced Technology Bridge

Acceptable Reliability Levels for Existing Road Bridges

Max KARLSSON
Head of Department
RAMBØLL
Copenhagen, Denmark

Born 1955. M.Sc. in structural Engineering and Applied Mechanics from the Technical University of Denmark in 1980. B.Com in 1983. Mr. Karlsson is highly experienced in the field of risk, safety and structural reliability.



Hanne E. ANDERSEN
Civil Engineer
RAMBØLL
Copenhagen, Denmark

Born 1970. M.Sc. in Civil and Structural Engineering from the Technical University of Denmark in 1995. Ms. Hanne E. Andersen is experienced within the field of risk and fire analysis.

Abstract

When analysing existing bridges practising engineers are often facing the problem that repair is very expensive for a bridge not having the structural reliability as required according to the code of practice. Intuitively one may conclude that if the code of practice leads to an optimum reliability level for the design of new bridges a minor decrease in this reliability level is acceptable before an extensive repair of an existing bridge shall be carried out. The point is that at the design stage the expenses involved in increasing the structural reliability will in many cases be relatively marginal as the structures easily can be altered. The same increase in reliability for an existing bridge will, however, in general be relatively expensive or even not economically feasible.

On the other hand it has been argued that the required reliability for an existing bridge must be the same as for a new bridge because the society will not accept a lower reliability even though it is demonstrated that such a decision would be cost optimal.

In this paper a rational way of defining the acceptable reliability level for an existing structure is presented along with practical values and examples. The approach is based on the statistical decision theory, and it is not required that the reliability for design of a new bridge necessarily shall be used for an existing bridge.

The problem of determination of acceptable reliability levels has for existing structures been treated in several other publications. The present paper is focusing on road bridges and emphasis is put on the use of economical values for construction and repair that are relevant to this type of structures.

It is demonstrated that assuming that the design practice is optimal the social failure costs associated with road bridges are very high. The high failure costs can only be explained by the psychological factors associated with the collapse of a bridge.

In the paper cost benefit studies assuming a variety of repair costs and failure costs are presented. With the economical values for construction and repair that are relevant for road bridges and the safety levels of this type of structures it is found that only a rather small decrease in structural reliability of a bridge should be allowed before repair is initiated.

If the same failure costs are associated with existing bridges as with similar new bridges, only a small decrease in reliability should be allowed before repair is economically feasible. As a rough rule of the thumb: If the reliability of an existing road bridge decreases with $\Delta\beta=0.5$ repair should be initiated.

If the repair costs are very low, repair is optimal even if the decrease in reliability is marginal. If the repair costs are very large, repair can in some situations be postponed, but it is worth mentioning that it is found that even if the repair costs is equal to the cost of constructing a similar new bridge it is found that repair is optimal if the reliability of the bridge is decreased with less than $\Delta\beta=1.0$.

If lower failure costs associated with an existing bridge are accepted compared to the failure cost of a similar new bridge, repair can in some situations be postponed. However, even if the failure costs of an existing bridge are judged to be only 20% of a similar new bridge (which is an extreme case), repair should be performed if the reliability decreases with $\Delta\beta=1.0$.

Considering Danish road bridges that are to be repaired for structural reliability reasons, a decrease of $\Delta\beta=1.0$ is only a small decrease. Loosely a decrease in reliability of $\Delta\beta=1.0$ is approximately equivalent to a decrease of 10% in carrying capacity for such structures.

Consequently the main conclusion of the present paper is that only a rather small decrease in carrying capacity of an existing road bridge should be allowed before repair is initiated.

It is emphasised that the paper is dealing with the larger repair works. The numerous minor and repetitively repair works are not considered. Furthermore, the problem concerning the upgrading of bridges to a higher load level is not treated. However, the general methods used in the paper can be applied also to these problems.

Uncertainties of Explosion Loads and its Influence on Reliability

Elisabeth AHLENIUS
Ph.D. Student
Royal Inst. of Technology
Stockholm, Sweden

Elisabeth Ahlenius, born 1953, received her Master of Science degree from Technical University of Gdansk in 1977 and from Royal Institute of Technology in Stockholm in 1983. During the last years she worked with risk analyses of road and rail tunnels.

Summary

This paper presents different aspects of uncertainties concerning the estimation of accidental loads caused by an explosion of dangerous goods in transport and its influence on the reliability of structures. Both physical and statistical uncertainties are discussed. The physical uncertainties about explosion are described concerning both the character of the source which drives the blast wave as well as environmental conditions: free-field explosions and explosions in fields with some degree of confinement or occurrence of obstacle. Uncertainties concerning estimation of probabilities of explosion using available statistics about reported accidents are discussed. Some sources of faults or misjudgements caused by using statistical information about low-probability events are highlighted. Methods suitable for use in cases of limited data availability are presented. Finally - the structural response, damage criteria and reliability of the structure are discussed.

Keywords: explosion; dangerous goods; statistics; uncertainties; low-probability event.

Abstracts

The lifetime of structures we design and construct today makes it obvious that we build for future generations. Structural safety must be sufficient, but on the other hand, because of economic reasons and concerning the inevitable use of natural resources, the structure must be optimised. After years of research and laboratory tests we can quite exactly predict the resistance of the structure. Unfortunately, the value of loads and especially accidental loads acting on the structure can not be predicted with the same accuracy neither concerning the probability of occurrence of a load nor its magnitude.

The values of loads are necessary for the estimation of the safety level of the structure and comparison with the demanded safety. Applying reliability theory makes it possible to handle uncertainties in this problem. However, reliability is a function of the information used in the analysis.

This paper describes some aspects of physical and statistical uncertainties about blast wave caused by an explosion of dangerous goods in air. As an introduction, the structure of the ideal blast wave is described in general terms. Differences between characteristics of the blast wave caused by explosive substances as TNT and fuel-air-mixtures are presented.

Some general uncertainties of estimation of the value of the load, as:

- the model uncertainties connected with use of the TNT model,
- metrical uncertainties of measurement of pressure with great magnitude and
- translational uncertainties of interpretation of results of small-scale laboratory tests are pointed out and described.

The physical uncertainties of explosion concern both character of the source, which drives the blast wave and the environmental conditions at the place of the explosion. The influence of the environmental conditions on the phenomena of explosion is described in brief concerning free-field explosions, explosions in fields with some degree of confinement or occurrence of obstacles. The result of the computer calculation of the detonation of 333-kg of TNT in the centre of a tunnel is illustrated.

Statistical uncertainties concerning probabilities of an explosion are discussed. As a rule, probabilities of explosive events are estimated using statistics about reported accidents. Some sources of faults or misjudgements caused by using statistic information about low-probability events will be highlighted. Application of some methods useful for the analysis under conditions of limited data availability as Fault tree and Beyesian Analysis is illustrated.

Finally – specification of the requirements according to the codes as well as the damage criteria are presented. The appropriate degree of the reliability of a structure in regard to the possible consequences of failure and the cost of safety measures are discussed. Some guidance about the character of the loads as well as certain types of construction, which can be recommended, are presented.

Influence of the local character of the explosion load on the principles of the design of tunnel structures is pointed out.

Some guidance about good practice of the design of structures to provide an adequate margin of safety against catastrophic collapse is presented.

Designing Buildings for Maintenance Using Property Manager Input

David ARDITI
Professor
Illinois Inst of Tech
Chicago, IL, USA

David Arditi, born in 1945, received his PhD from Loughborough University in the UK, is the author of more than 150 scholarly publications, and is currently Director of the Construction Engineering and Management Program at IIT.

Manop NAWAKORAWIT
Graduate Student
Illinois Inst of Tech
Chicago, IL, USA

Manop Nawakorawit, born in 1970, received his Bachelor's of Architecture from Silpakorn University in Thailand and his Master's degree in Construction Engineering and Management from Illinois Institute of Technology

Summary

A survey was conducted of the largest 230 property management firms in the U.S. to investigate their current maintenance practices. Among other things, the findings shed light on the severity of the maintenance-related complaints property managers receive from tenants; the extent to which they are involved in the design process of the buildings they subsequently manage; and the frequency with which designers come back to assess their building's performance after the projects are completed. Property managers should be cognizant of building users' concerns and should make designers aware of these concerns too; the performance of buildings is likely to be enhanced if designers and property managers take action in the design and the operation phases respectively, in response to users' concerns.

Keywords: Building design, maintenance practices, property management.

1. Introduction

All buildings start to deteriorate from the moment they are completed, and at that time begins the need for maintenance. Increasingly building owners are beginning to accept that it is not in their best interest to carry out maintenance in a purely reactive manner, but that it should be planned and managed as efficiently as any other corporate activity. Inevitably this has placed new demands on property managers requiring them to adopt a more systematic approach to their work, and on designers requiring them to design buildings for low maintenance.

In many instances, building owners and users spend billions of unnecessary dollars each year on excessive maintenance and replacement components for their buildings; in other instances, they let buildings deteriorate into a state from which it is very difficult and costly to recover. Building maintenance is a seriously neglected area of research and study. Few schools of architecture and engineering include it in their curricula and only recently has work commenced on the research and development of this subject. This study is therefore concerned with maintenance problems that can be avoided through proper design.

2. Methodology

A questionnaire was designed to study the maintenance practices of property management firms. It was mailed to the largest 230 property management firms in the U.S. The mailing list of property management firms was obtained from the "Top Property Managers Survey" [1,2] ranking the most active property managers in North America based on total space in their portfolios. The 230 questionnaires were mailed to the top executives (e.g., presidents, chairmen of the board, or CEOs). By the cut-off date, 70 questionnaires were received, constituting a rate of response of 30%.

The questionnaire includes questions regarding the characteristics of the firms and of the buildings they manage, the relationship between property managers, building designers and users, the maintenance organization, the maintenance services provided, and the maintenance-related complaints reported by building users. Only the findings regarding maintenance issues in design are reported in this paper.

3. Findings and Conclusion

Property managers reported that the most important problem they face in building operation and maintenance is building design inefficiencies. Contrary to the popular belief that the management of the maintenance of a building begins only after the building is built, the findings indicate that the large majority of property managers are very much involved in one way or another in the design process of the buildings they subsequently manage. On the other hand, only slightly less than half of the property managers indicated that designers come back to assess their buildings' performance after the projects are completed, implying that only about half of the designers are aware of the complaints most commonly cited by building users. Functional design alternatives and choice of building materials and equipment constitute the inputs that most property managers would like to give to the designers in order to minimize anticipated maintenance problems.

It appears that comfort factors constitute the major area of complaint on the part of building users. The performance of buildings is likely to be enhanced if property managers efficiently communicate with building users, are aware of users' concerns and take action to eliminate these concerns. Some of the users' maintenance-related concerns are not likely to develop if property managers are in constant communication with designers during the design phase and after the building has been put in service, and make the designers cognizant of maintenance-related matters.

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Urban Database for the Sewer Networks Management

Katia LAFFRECHINE
PhD student
Univ. of Bordeaux I
Talence, France

Katia Laffréchine, born 1972.

Denys BREYSSE
Professor
Univ. of Bordeaux I
Talence, France

Denys Breysse, born 1958, received his civil engineering degree from the ENS Cachan, 1980

Summary

Sewerage systems ageing is a real problem in old-industrialized countries, since the loss of performances with time results in economic and environmental problems. Sewer systems managers try to rationalize the long-term maintenance, but they lack practical tools to handle this problem. This paper shows how it is possible, from data obtained during video-inspections of unvisitable sewers, to build a statistical model of ageing.
Practically, using statistical models, we will show what cost/benefit balance would be expected if they would be used to plan the inspection strategy and the repairing program.

Keywords: Unvisitable sewer system, urban database, ageing model, maintenance, ageing factors.

France accounts for a linear of about 150 000 km of urban sewer networks, constituting a patrimony valued at 45 billions Euros. Facing the high maintenance costs to optimize performance functions, the sewer manager must be able to define essential parameters for a good description of the ageing process. Further these economic stakes, technical stakes appear: to increase performance of future networks, to limit environmental risks (pollution) and urban system dysfunction.
Our effort focus on a database development in order to establish a strategy for the study of the ageing process, with has to allow the optimization of sewer system maintenance. The UCB's (Urban Community of Bordeaux) sewer network (diameter < 1200 mm) served as a basis for our research.

Available information are pathologies listings. Pathologies are interpreted in nature terms (crack, break, and collapse...) but also in gravity terms. The binary rating 0/1 is the simplest way to evaluate the section performance; 0 for a virgin pathology section and 1 if at least one pathology has been tacked, whatever it may be.

Ageing model can be written under the form N(t, *Xi*), N being the average mark of the section population, t the section's age and *Xi* describing of parameters vector that conditions the section performance evolution with time:

$$N(t) = N0(Xi) + V(Xi)*t + f(L/D) + g(\text{location})$$

with N0(*Xi*) is the damaged new section ratio in the *Xi* configuration,
Xi = (backfill height, traffic)
$f(L/D) = 0$ if $L/D \leq 5$
$f(L/D) = 0.179 - 0.0124 * (L/D)$ if $L/D > 5$

and g(location) = -0. 045 if location = Bordeaux (downtown)
g(location) = 0.011 if location = Bordeaux suburbs

Figure 1 compares the experimental data and the model prediction for the two extreme subset. The difference between the two ageing speeds is clear, and we accounted for by the model. One can mark that it remains local differences between model and reality, this noise may be due to factors not accounted for in the model. However, if one aims at a model that can be currently used by the sewers manager, the model must be simple and not to ask for an important quantity of data.

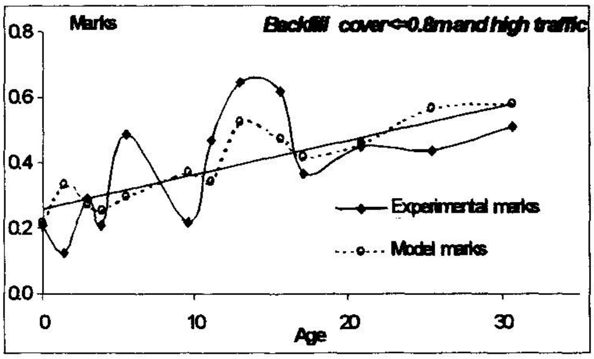
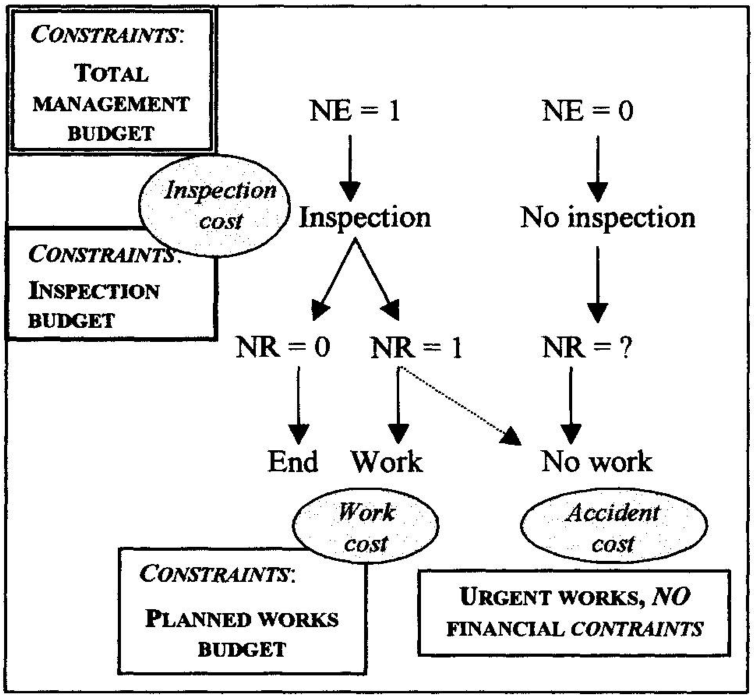


Fig. 1: Backfill cover ≤ 0.8m and high traffic

Ageing model allows to evaluate the network's performance evolution in time using Bayesan probabilities. A short or long term perspective permits, for example, to orient video inspections or work strategies, to simulate future ageing of pipes, to anticipate investments and therefore to optimize the patrimony management.
A first database exploitation will permit to quantify administrator's gain using ageing model. Distance between real performance marks and estimated ones will quantify this information gain.



To quantify replacement strategy's gain using ageing model we put into works

a gain function (make up maintenance cost, inspection cost, work and accident cost) to optimize by integrating model contributions and accounting for financial constraints.

Urban database construction for unvisitable network management's optimization comes up against quality and quantity required information. Data from various sources (geotechnical, technological, materials, socio-economical) are often vague, incomplete and uneasily accessible.
A statistical approach, determinist and critical of available information has allowed to establish a first ageing model. This model describes correctly the UCB's individual database behavior. Backfill cover above the pipe, traffic intensity, length diameter ratio (L/D) and section's pipe in the urban fabric have been identified as more influent parameters. More, this study points up indispensable factors to understand the phenomenon, but not documented, relative especially to laying conditions. Ageing model used for management strategy gives possibility to optimize network administrator's costs.
Which potential database, simulations of inspection and work strategies, show how the manager can improve his knowledge on the system efficiency of alternate managing strategies.

Zárate-Brazo Largo Bridges, Rehabilitation Design Basis

Dietrich HOMMEL
Civil Engineer,
COWI
Copenhagen, Denmark

Dietrich Hommel is project manager for the assessment and rehabilitation of the Zárate-Brazo Largo bridges.

Ejgil VEJE
Civil Engineer,
COWI
Copenhagen, Denmark

Ejgil Veje is head of the department: Major Bridges, Maintenance and Rehabilitation.

Svend ENGELUND
Dr Eng.
COWI
Copenhagen, Denmark

Svend Engelund is a specialist in probabilistic methods and assessment of existing structures.

Summary

One stay cable of the Guazú bridge ruptured in November 1996 after 20 years of service without prior indication of the critical situation. Using information from tests and measurements at the bridge together with a definition of the safety requirements for various emergency and long term conditions a rehabilitation design basis was developed. This rehabilitation design basis provides a basis for the evaluation of the load carrying capacity and service life of the cables.

Keywords: Stay cable, Corrosion, Fatigue, Rehabilitation, Reliability, Probabilistic models, Inspections, Tests, Design basis

1. Introduction

One stay cable of the Guazú bridge - one out of two quasi identical, combined road and railway cable stayed bridges across the Paraná river between Zárate and Brazo Largo, Argentina - ruptured in November 1996 after 20 years of service without prior indication of the critical situation. Subsequent inspection revealed a substantial corrosion of the wires and that the fractures were fatigue-like. In Figure 1 the Guazú Bridge is shown.

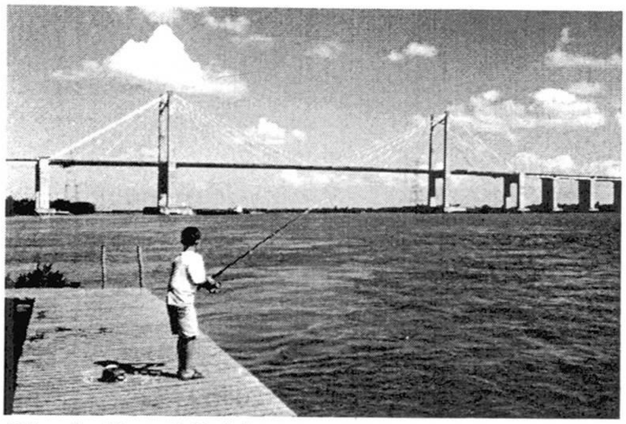


Fig. 1: Guazú Bridge.

COWI Consulting Engineers and Planners was entrusted the job of evaluating the residual load carrying capacity of the cable stayed bridges and planning of the emergency replacement of the heavily deteriorated stays.

The evaluation of the residual strength and the planning of the emergency replacement of these stays were performed using a rehabilitation design basis developed specifically for the Zárate-Brazo Largo bridges.

A bridge specific rehabilitation design basis is developed in order to ensure that the reliability of the bridges is acceptable while at the same time ensuring that the bridges are open for the largest possible amount of traffic. In other words, the bridge specific rehabilitation design basis is developed in order to minimise the consequences of the traffic restrictions while ensuring an acceptable level of safety to the user.

2. Development of the rehabilitation design basis

The rehabilitation design basis is developed within the framework of the Load and Resistance Factor Design method (the LRFD method). In the rehabilitation design basis characteristic values of loads and resistance factors are given together with partial safety factors which ensure an acceptable level of safety.

The characteristic traffic load is developed for different traffic situations, e.g. the traffic is restricted to a given number of lanes, the bridge is closed for trucks exceeding a certain weight or unrestricted traffic. The traffic load is specified as an Equivalent Uniformly Distributed Load and a set of axle loads. The traffic load is determined on the basis of a state-of-the-art probabilistic model developed by Madsen and Ditlevsen [2] for the Great Belt Bridge in Denmark. The model is based on information on the traffic intensity, the amount of trucks and the mean number of trucks in a platoon. This information can be obtained from observations at the Zárate-Brazo Largo Bridges.

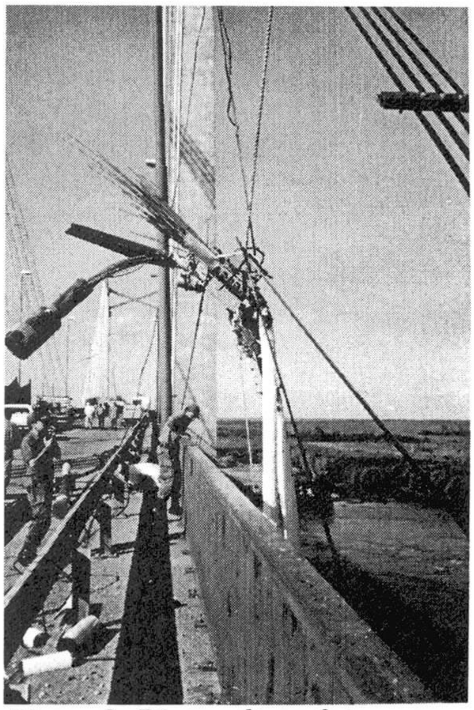


Figure 2: Dismantling of stay.

The characteristic values of the load carrying capacity of the stays are determined on the basis of tests of the ultimate strength of wires from the dismantled stays (see Figure 2) of the Zárate-Brazo Largo Bridges.

On the basis of an analysis of the reliability of the bridge in the ultimate limit state, the partial safety factors are calibrated using the methodology given in the EuroCode [1]. In the calibration of the partial safety factors the deterioration of the stays is taken into account such that the partial safety factors depend on the expected service life of the stays.

The deterioration of the stays is predicted using an advanced fatigue model for parallel wire cables suggested by Rackwitz and Faber [3]. The model parameters are determined using results of fatigue tests. These tests were performed on test specimens from the dismantled stays of the Zárate-Brazo Largo bridges. The effect of corrosion is taken into account by calibrating the model using test results valid for corroded wires. Further, the rate of deterioration is updated for the individual cables using information from ultrasonic tests performed on site.

Finally, it is possible to develop a design basis for individual cables as well as the entire bridge. By developing a design basis for individual cables it is possible to use very accurate models and very exact information implying that the partial safety factors generally are lower.

3. Conclusions

The development of a rehabilitation design basis using all available information from the Zárate-Brazo Largo bridges concerning the traffic conditions and the actual condition of the stays ensures that accurate predictions of the traffic load and the load carrying capacity of the structure can be made. This implies that cost-effective solutions regarding the traffic restrictions on the bridge and the replacement of the stays can be made for the benefit of both the owner and the users.

4. References

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External Beam-Column Joints - The Importance of Stirrups

Icléa REYS DE ORTIZ
Civil Engineering
Univ. Federal do Rio de Janeiro
Rio de Janeiro, Brazil

Icléa Reys de Ortiz received her Civil Engineering and Master of Science degrees from the Catholic Univ. of Rio de Janeiro, Brazil. In 1993 she received her PhD from the University of Westminster - London - England.

Abstract

This paper presents the results of experimental tests on reinforced concrete external beam-column joints (T joints). The seven test specimens were designed planning to have real practical dimensions avoiding major size effects and at the same time suiting the rig arrangements that simulates the points of contraflexure in framed structures – see Figure 1. The concrete strength, the beam and the column reinforcement were intended to be the same for all the specimens. Different reinforcement details were used in the joints and in some tests the columns were axially loaded to simulate other floors in the building. The Beam-Column-Joint (BCJ) and the corresponding control specimens were tested the same day.

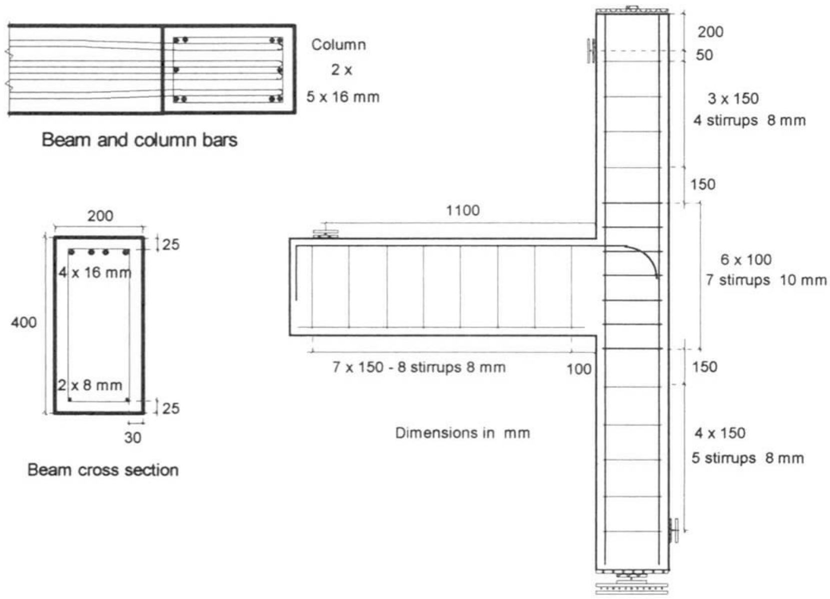

Figure 1 - Details of BCJ7

The aim of the tests was to study the internal mechanisms of stress transfer in the joints and the effects of the stirrups in the connection. The tests were carried out accompanied by strain measurements on the most important sections and on several points of the reinforcement. The crack patterns were observed and marked at each step of the tests.

Based on the measurements and on the observations during the tests, a strut-and-tie modelling is developed showing how the stirrups work.

A resistance analysis is developed based on the shear that acts in the connection (beam bars tension minus column shear) and the vertical forces coming from the column. This analysis shows that the transverse reinforcement has an important role at the ultimate resistance of the connection – see Figure 2 - but has no influence at the first thin crack appearance.

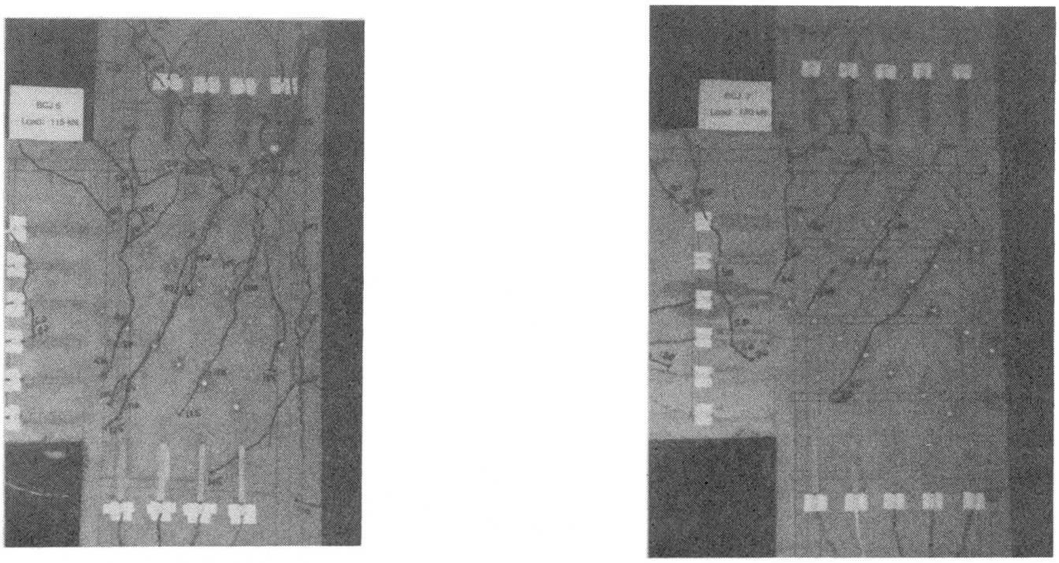

Figure 2 - a) BCJ6 – without stirrups – 115 kN *b) BCJ7 – with stirrups – 120 kN*

The effect of the stirrups is limited by the concrete resistance of the joint – concrete strength and connection dimensions. Any excess of transverse reinforcement does not increase the strength of the joint.

A theory is proposed to analyse the shear strength and to design the transverse reinforcement (stirrups) for this type of joint.

These tests have shown that the active stirrups have to be evenly distributed between the direction of the top of the beam chord and the main beam bars (at the top of the beam). At least two extra stirrups should be located up to half column depth above the connection to guarantee the region to where the cracks tend to run.

The observations, conclusions and analysis of this work have been seen to concur with published test results.