Search for Quality in the Vasco da Gama Bridge Project

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Summary

A description of the Vasco da Gama Bridge crossing is presented, highlighting the special design studies developed to achieve the quality standards defined for this large project.

Keywords: Bridge; design; durability; quality; Vasco da Gama

1. Introduction

The Vasco da Gama Bridge Project consists of a 12km crossing of the Tagus River in Lisbon, being the longest bridge in Europe. This large project was developed under the supervision of a Government Office, GATTEL, who was quite active during the tender, design and construction phases, aiming to obtain a bridge with high quality standards. In this paper some of the studies developed to achieve quality in terms of structural safety, durability and aesthetics are presented.

2. Structural Behaviour

The structural design was developed for all the structures of the crossing, considering national end recent EC codes and deep seismic analysis with soil-structure interaction. Due to the importance of the main bridge over the navigational channel (a cable-stayed solution, with 420m central span) special research studies were also developed at IST related to the behaviour of the main bridge.

The initial deck configuration led to aerodynamic instabilities when analysed at the wind tunnel. Considering an innovative CFD analysis and further tunnel tests, a stable final solution was achieved with two baffles under the deck. The dynamic behaviour of the deck under wind gusts was also analysed as well as the cable stays behaviour.

The flexible solution adopted for the deck led to important longitudinal displacements at the abutments under the seismic action. To reduce these displacements steel hysteretic dampers were then studied and their optimal characteristics were determined, leading to an innovative solution.

The situation of a ship fire under the deck was also studied to estimate its effects in the bridge safety and the time for protection measures.

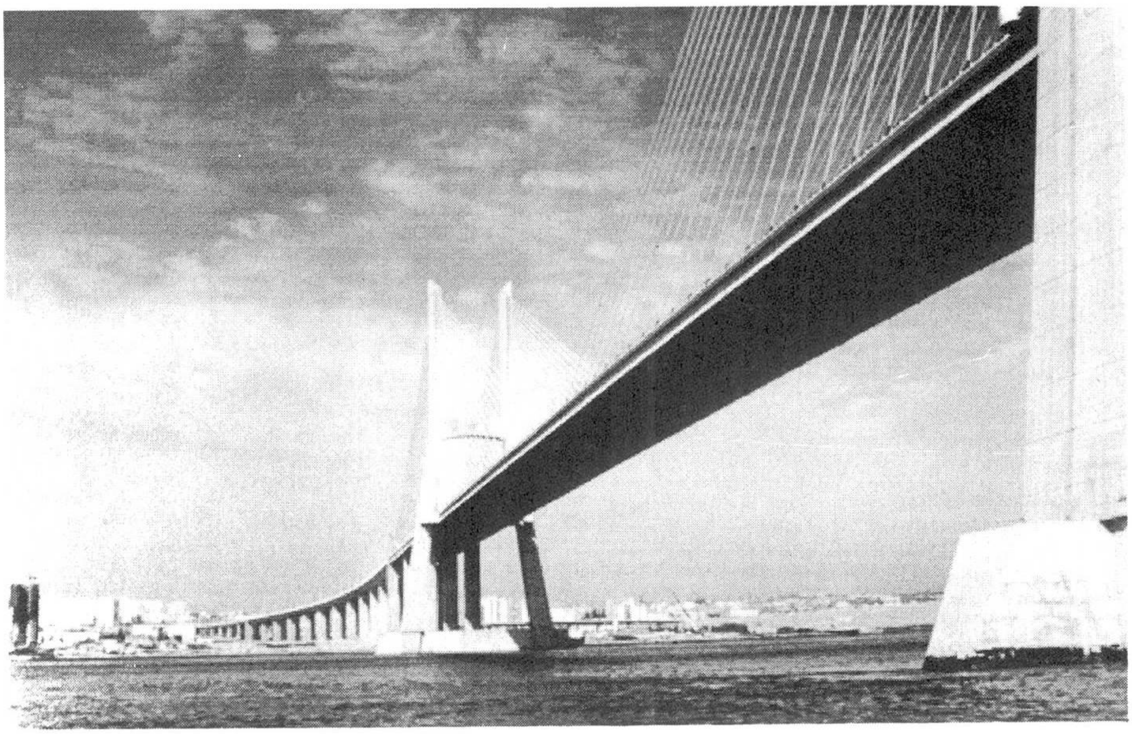

Fig.1 The Vasco da Gama Bridge

3. Durability and Maintenance

The bridge is located in the mouth of the Tagus River which is a sea type salty environment, subjected to tides and waves. The tender specifications defined for the crossing a service life of 120 years. The implementation of this requirement led to several actions, at design stage, as the definition of geometry and materials (considering the study of the degradation of the materials and components and the associated mathematical models) and the study of a durability monitoring system.

To achieve durability, during the construction stage, a quality control program was defined with procedures to fulfil the durability specifications. This included the implementation of periodic in-situ measurements of the degradation of the materials.

4. Aesthetics

Special attention was also paid to the aesthetics of the crossing. This included several studies related to the crossing layout in the river, global uniformity, gradual evolution of the structures geometry and colour. Local studies were also developed related to the shapes of the towers, cornices, lamps, etc., aiming to obtain a final aesthetic pleasant solution.

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Optimisation of Concrete for Durability and Accident Resistance

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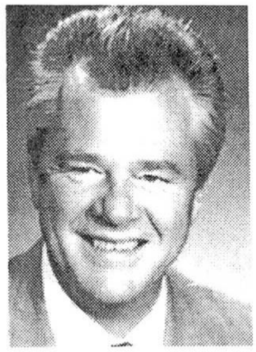


1. Conflicting requirements

The governing requirement for the overall optimisation of structures is that the lifetime cost must be kept at a minimum. In this respect "cost" may be understood in broad terms covering both economic, environmental and safety matters, as the case may be.

To ensure fulfilment of the important requirement of durability and performance, structures must be designed to ensure safety, constructability, serviceability, appearance, maintainability, etc.

In addition, the ability to withstand relevant accidental events is essential. For concrete structures especially fire resistance may be important. This is particularly important for tunnel structures.

These two sets of requirements may in many cases be in contradiction: For example may a long service life usually be obtained with a large and dense concrete cover which, on the other hand, is susceptible to spalling damage in case of a fire. Also a possible protection of the reinforcement against corrosion by using epoxy coating may lead to problems in the case of fire, especially for bond and lap splices.

To reduce the adverse consequences in case of fire it is possible to design the structure to resist such damage and to add specific fire protection.

The main question is: Does the - usually small - likelihood of f~re and the consequence of fire damage justify large initial expenses, and do they justify compromised durability? This is the critical question to be answered.

2. Mitigating measures and fire resistance

Degradation of concrete structures can be avoided or prevented through:

- Changing of the micro environment of the concrete or restriction of transport of combustible material. It can be done by tanking, membranes and coatings which provide a protective interface between the aggressive environment and the concrete surface.
- Selecting of inert material. An example is stainless steel reinforcement which could be used in marine structures or parts exposed to de-icing chemicals. Another, but apparently less reliable solution, is coated reinforcement to provide a protective barrier against corrosive agents.
- Inhibiting the reactions or selection of fire resistant concrete compositions. Cathodic protection is one way, another is air entertainment to avoid frost attacks.

- Selection of optimal material compositions and detailing within the design. Corrosion protection can be achieved by selecting appropriate concrete covers and concrete mixes or in the detailing by minimising the exposed surfaces and adequate drainage.

A strategy for fire resistance could include:
- Coating of the concrete surface with fire protection materials.
- Using of fire-resistant concrete which can withstand a fire without spalling.
- Installation of fire-suppression systems such as sprinklers, dilution systems, gas suppression or foam suppression.
- Ventilation as an effective measure in connection with evacuation.
- Adding of plastic fibres to the concrete. The fibres will melt during the fire and thereby produce
- pores for the migration of moisture.

3. Decision support

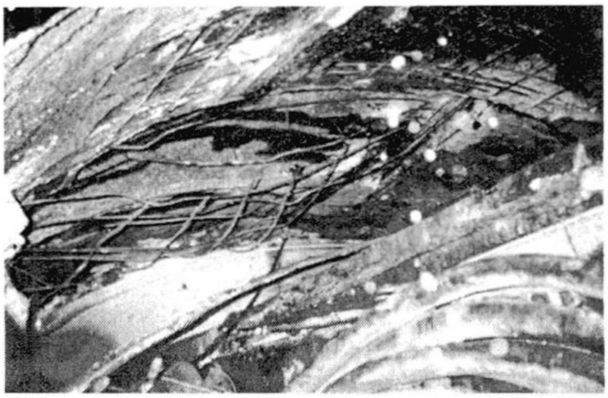
Back to the main question: Does the likelihood of fire and the consequence of fire damage justify large initial expenses, and do they justify compromised durability?

The answer is neither yes or no. It depends on the individual structural circumstances. However, today it is possible to model the circumstances and find an answer representing an overall cost optimal lifetime solution. The circumstances to be modelled may be: The type of structure, the expected lifetime, the type and aggressivity of the environment, the consequence of a durability failure, the consequences of a design fire, the possible consequence for the life of human beings, etc.

The optimisation of the concrete mix and the decision on other measures concerning these aspects, such as special provisions to enhance durability and provide fire protection, can be made based on detailed knowledge of the materials, structures, operation as well as on an analyses of the events, i.e. all in all performing a risk analyses. The framework is a decision theory. It will lead to a structural decision which is balanced in accordance with the aims, i.e. an optimised structure with respect to both durability and accident resistance.

The risk analysis must establish the probabilities of occurring deterioration mechanisms as well as fire scenarios and the combined assessment of the consequences of these actions. The probability is based on available information on deteriorating concrete structures as well as on statistical information on traffic, accidents and transport of flammable materials.

The effects and the costs etc. of all risk reducing measures must be established and used in the evaluation of the most suited set of measures.



Great Belt Tunnel after a fire during construction.

Different approaches are possible for the decision support model: Bayesian decision support, cost-benefit analyses, multi-criteria decision models, etc. The decision support models aim at the solution with the highest expected utility for the decision maker.

In practice the certain event of ageing will most often govern the decision. Therefore, concrete composition and reinforcement type and layout providing a durable concrete structure will usually be the optimal choice.

Design for Durability - A Matter of Good Choices and Good Codes

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Summary

Concrete structures' design must be based on a tripod: serviceability, safety and durability. Unfortunately, a significant number of works were carried out without due care with these three aspects. The consequence was that a lot of structures suffer premature damage due to inadequate choices during design, improper detailing or construction faults. This paper discusses some cases dealt by the author with during his professional life in Rio de Janeiro as well as the solutions proposed to repair the damages. The experience assembled shows that good codes are nor enough to prevent bad results if engineers are not prepared to design, detail and build for durability, serviceability and safety. Good colleges and teaching by experienced professionals are also very much needed. The public authorities must also guarantee external quality control of projects and constructions.

Keywords: durability, safety, serviceability, concrete structures, damage, code, quality control,

1. Introduction

Design of structures means to fulfill three fundamental concepts: safety, serviceability and durability. Any of them is essential for a successful enterprise. All them are important for the final quality of the structure. To neglect, even partially, anyone is to condemn, for sure, the final result of the work, if not the stability or lifetime of any construction. Engineers are responsible for the goodness of the response given by a structure to these three fundamentals.

Professionals can learn not only from big accidents, but also from small imperfections, small mistakes, or small negligences that might be the origin of serious disruptions and huge loses. This paper discusses this domain of small causes and big consequences of structural pathologies in urban constructions. They have quite different origins – design, construction technique, quality of materials, building monitoring, management during lifetime, and others. Discussion is based on the experience assembled by the author during his professional life in Rio de Janeiro and abroad.

In Brazil, there is no continuous and permanent surveyor of an engineer in many small and medium work sites, either from contractor, owner or public authority. This situation implies the occurrence of frequent construction faults that may compromise the final quality of the work. The most part of the chief engineers are obliged to manage more than one site or to deal with technical, financial and administrative tasks at the same time. This situation creates a lot of problems in building sites that could be avoided. Some examples are:

- construction joints badly treated, with lacks, voids, and disaggregated materials;
- reduced reinforcement cover due to bad placing of reinforcement cage, mainly in slabs;
- adaptation of design details done at site without consultation to the designer, leading to poor solutions, some times worse than the original designer's proposal;
- bad assemblage of braces and scaffolding conducing to drops or voids between wood planks, which disturbs placing and vibration of concrete and, therefore, its capability to protect reinforcing bars against corrosion;
- use of inadequate, bad quality or badly placed materials to cover facades, allowing water and aggressive infiltration into the walls and structural elements;
- modern tendency of humanising constructions with plants in concrete boxes connected to the structure, without a good insulation, allowing also water flow into the structure;
- embedding of metallic bolts into the structure without adequate corrosion protection and inadequate maintenance.

2. Buildings in Rio de Janeiro and their pathologies

Search for shorter construction time and economy of materials in these constructions brought as consequence the adoption of large slabs, few beams and slender structures. Design at ultimate limit state induced also a minimisation of geometric dimensions of structural elements, reducing them in relation to those normally gotten through classical design based on service limit stresses.

During the last few decades this practice caused that deformability of structures became a major parameter in the design of reinforced concrete buildings. Bigger spans, free to deflect, imply that time dependent deflections are more and more relevant and need to be carefully treated in conception, design and construction.

Climate is also the cause of several disorders in urban buildings. Improper choices, for the environment in which the constructions are placed, both architectural and structural, may shorten their lifetime or reduce their resistance. Deficient or non-existent maintenance procedures worsen the deterioration process, increase rehabilitation costs and may compromise the stability of the structures.

The paper treats the different disorders from the point of view of their main cause – design, construction or use of the structure, as understood by the author. Restoration proposals for some real cases, such as excessive deflection of slabs, inadequate water drainage of balconies, thermal effects on roof structures without proper insulation, attack from weather conditions, removal of frameworks to early, chloride effects on concrete and steel, change in use of a floor dealing with excessive load, poor development of rehabilitation work of an old church, are presented in this paper.

Solutions proposed for each case reported are also presented and results briefly discussed.

References to the technical reports prepared by the author are made.

Improvement of Bridge Design in Relation to the Action of Water

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Summary

To enhance the quality of bridges, it is necessary to protect them against the action of water. Lessons learned from the pathology of existing bridges provide an excellent way for improving design. Sixteen defects have been identified, and for each of them a sheet has been published to present the description of the defect, the analysis of the causes, the consequence for the bridge and the proposed solution. This paper presents some of these sheets, concentrating on the points which are most important for the designer of a bridge.

Keywords: bridge ; design; water; waterproofing membrane; expansion joint; cornice; drainage; drip; durability; protection.

1. Introduction

Water is the principal enemy of civil engineering structures, being involved in most of the processes which affect their durability. The ingress of water into structures assists the penetration of aggressive agents, of which de-icing salts are among the most harmful. Water is responsible for corroding steel reinforcement and is involved in pathological processes such as carbonation, the alkali-silica reaction, sulphate attack and deterioration caused by freezing and thawing, etc.

To ensure that structures are durable and that maintenance can be performed without undue expense the action of water must be taken into account from the design phase. The experience which has been gained in recent decades from inspections and the pathology of existing bridges provides design engineers with essential information concerning the penetration of water into structures. The French Ministry of Public Works, more precisely the « Service d'Etudes Techniques des Routes et Autoroutes » and the « Laboratoire Central des Ponts et Chaussées », have published a set of 16 information sheets which are intended to improve the design and construction of bridges in relation to the action of water. These sheets are based on a large number of surveys conducted by the Regional Road Research Laboratories and on the views of specialists from the SETRA and the LCPC who are responsible for laying down technical principles. These sheets aim to assist designers, contractors, construction authorities and project managers.

2. Main points for the design of a bridge in relation to the action of water

2.1 The Necessity to install a waterproofing membrane

In the case of all new structures, including prefabricated underground culverts, all parts of the structure must be waterproof. This also applies to high performance or very high performance concrete bridges: although these materials are considerably more compact than ordinary concrete, cracking results when they are laid (in particular shrinkage cracking soon after laying) which provides routes for the penetration of aggressive agents.

2.2 The Necessity to extend the waterproofing membrane under footpaths

It is necessary to provide a waterproofing membrane over the entire deck, continuing under the footpaths and rising into a recess in the longitudinal beam which supports the cornice. In addition it is recommended not to anchor directly into the deck the longitudinal beams which provide support for the footpath kerbs or anchorage for the safety barriers as so doing will impair waterproofing.

2.3 The Selection and quality control of the waterproofing membrane

The waterproofing membrane must also be compatible with the type of structure, in particular one must be selected which adheres to the substrate and the bituminous mix in areas where vehicles brake. When such membranes are laid it is also necessary to comply with the correct procedure: the substrate must be prepared, climatic conditions must be taken into account, the laying procedure must be carefully followed. Lastly, particular care should be paid to all particular features (recesses, points of penetration, grids, downpipes, sealing of tendon anchors, etc.). In particular, waterproofing should terminate at its edges by rising to a recess.

2.4 Others points

When detailing, other important points should be considered :

- suitable waterproofing at expansion joints in pavements and footpaths
- improvement of the waterproofing of cornices
    - -- Preventing leakage between precast units
    - -- Ensuring Watertightness of cornice gutters
- taking care of the waterproofing adjacent to equipments
- installation of drips
- ensuring a good water drainage.

3. Conclusions

The installation of a waterproofing membrane on the entire structure is a fundamental factor in the protection of bridges from water. The collection and removal of water must not be viewed as a secondary detail of design, but as an important aspect of it. When the system of drainage devices is designed the path of water should be followed from when it reaches the structure until it enters the ditches or main drains designed to remove it.

Quality and Durability of Concrete Structures through CPF

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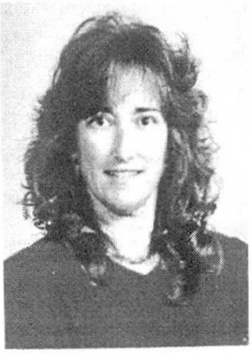

Joana Sousa-Coutinho, born 1956, received her civil engineering degree from the Univ. of Oporto, Portugal, completed her PhD. at the Univ. of Oporto, on "Durability enhancement of concrete through CPF" in July 1998.

Summary

A study of two CPF Controlled Permeability Formwork systems was carried out at Oporto University to ascertain the benefits of CPF compared to traditional Portuguese formwork. Several types of tests were undergone on concrete cast with CPF and traditional formwork, such as surface hardness, absorption, water penetration, mercury porosity tests including assessment of pore size distribution curves, carbonation, abrasion and chloride ion diffusion tests. The study lead to the conclusion that quality and durability of a concrete structure may be drastically enhanced by the use of CPF.

1. Introduction

Quality and consequently durability of concrete structures have turned into outstanding issues in the last 20 years as engineers have become aware of the leaping maintenance and repair costs for concrete structures.

The now classical approach to durability of a concrete structure, is already implied in Standards throughout the world (for example, in Europe - ENV 206) in which durability of a concrete structure depends on a good protective outer concrete cover over the reinforcement able to resist the environmental actions on the structure, throughout its design life. The quality of this cover is mainly achieved by considering simplicity in the design phase, an adequate mix with a low water-binder ratio, adequate binder, and efficient compaction and curing.

Quality and durability may be drastically enhanced by the use of Controlled Permeability Formwork (CPF), which consists of using a textile liner on the formwork. This liner acts as a filter/drain, allowing air bubbles and surplus water to drain out but retaining cement particles which are flushed out of the bulk of the concrete, mainly during compaction (*Fig. 1*).

When concrete is cast, the first particles in contact with the liner cannot proceed their way through and as compaction is carried out, more and more particles are retained further and further to the inside of the surface layer of the concrete. At the same time water and air bubbles manage their way out easily.

All this process leads to a very dense outer concrete layer with a very low water/cement ratio where it is most needed - the outer layer. This is, where concrete is usually of poorer quality. If the CPF formwork is not stripped off too soon, the liner will also induce hydration as the filter makes enough water available at the right time.

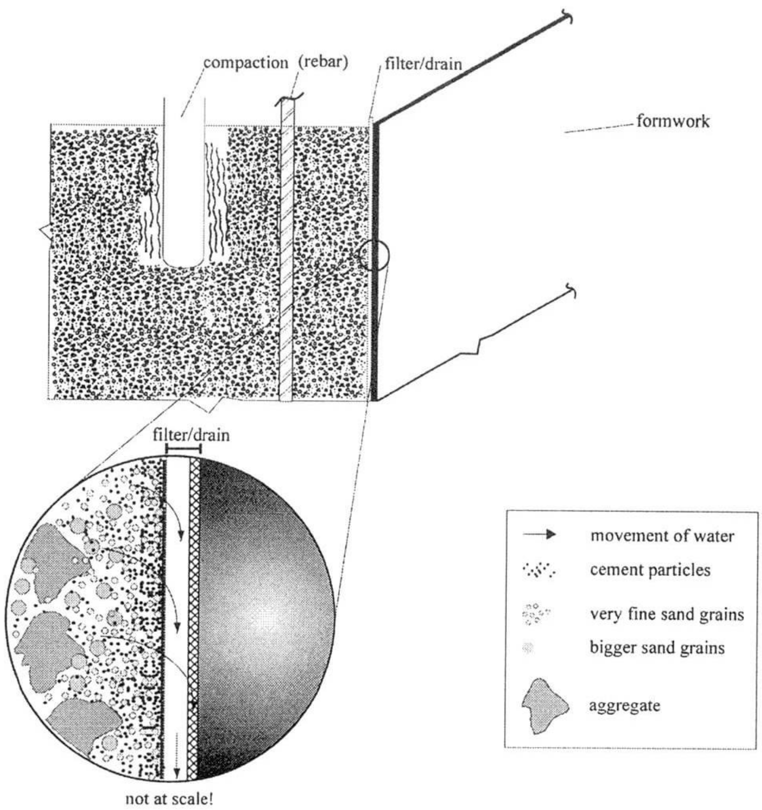

Fig. 1 - Controlled Permeability Formwork - a schematic representation.

2. Research Programme and Results

Test-walls 100 x 100 x 20 cm^3 of two concrete brands were cast using a form made of pine boards attached to each other (traditional Portuguese formwork) previously divided into vertical strips, each corresponding to different formwork types: untreated (control), CPF-T and CPF-Z. CPF-T was obtained from perforated Portuguese formwork over which was tensioned and attached a pp and pe liner, prior to casting. CPF-Z corresponds to Portuguese formwork over which was placed - no tensioning needed, Zemdrain MD1 specially engineered by DuPont for this purpose.

The surface hardness of the concrete was evaluated (EN/ISO 8045), 28 days after casting. Later, cores were drilled out and submitted to several tests. For each experimental test the outcome for cores of the same concrete but corresponding to different formwork types, was compared. This paper presents some of the results, with a special focus on the absorption by cappillarity tests.

The increase on surface hardness in CPF concrete was over 59% compared to the same concrete cast with the control formwork. As to other tests, the absorption coefficient decreased over 62%, permeability (water penetration) decreased over 49%, porosity (Mercury intrusion) decreased over 32%, carbonation decreased over 67% and concerning chloride penetration the Diffusion coefficient decreased over 38%.

3. Discussion

All these results confirm the aim of this study: CPF applied to Portuguese formwork does enhance durability. Although actual results have not been displayed it is interesting to verify that a C12/15 concrete cast with CPF-Z performed better in every test compared to a C30/37 concrete cast traditionally.

Swedish Experiences of Integral Bridges

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Summary

Sweden has a long tradition of the design and construction of integral bridges, i.e. bridges without expansion joints. The experiences of these types of bridges are very good and the cost for maintenance is lower than for other bridge types. The design is based on simple frame models justified by experience. These models are supposed to be conservative so there are many design problems not yet properly solved.

Keywords: bridges; integral bridge; concrete; design; maintenance.

1 Introduction

For over 70 years, the integral reinforced concrete slab frame bridge, according to *Fig. 1*, has been one of the most common bridge type in Sweden. In the latter years, another integral bridge type is increasingly popular with road and railway holders, bridges with integrated abutments according to *Fig. 2*.

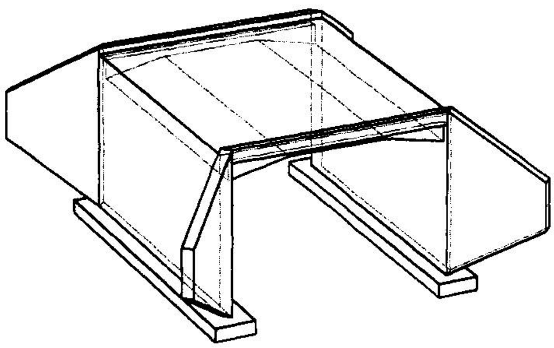

Fig. 1, Integral slab frame bridge.

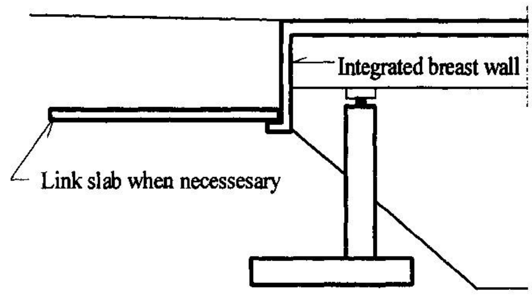

Fig. 2, Integrated breast wall.

In Sweden there are at least 8 000 bridges out of 14 000 bridges owned by the Swedish Road Administration, of the type shown in *Fig. 1*. A great part of medium span bridges up to a total length of 80 m are now designed using integrated breast walls in principle according to *Fig. 2*. Both road and railroad bridges with composite or concrete superstructure are designed with integrated abutments. The experiences of these types of bridges are very good and the cost for maintenance is lower than for other bridge types. There is however many design problems not yet properly solved i.e.:

- Soil pressure against the abutments, especially for load cases with large breaking forces from trains
- Interaction between the structure and soil
- Settlements in the soil behind the abutments due to repeated movements from temperature variations.

2 Design methods

The design methods used for integral slab frame bridges are based on experience and on simple elastic frame models. The bridge is modelled as a frame based on a strip with a width of 3 m. For bridges with larger widths and for bridges with many lanes there are some simple rules transforming the bridge to the width 3m.

Many of the slab frame bridges are skew. An approximate calculation method was developed at the Dept. of Structural Engineering, KTH already 1963 by Uppenberg (1963). The method was based on a large series of elastic models, and the method is based on some correction factors for transforming the skew bridge into a straight model.

Bridges with integrated breast walls, see *Fig. 2*, is nowadays widely used for bridges with a total length of not more than 80 m for concrete bridges and not more than 60 m for steel or composite bridges. The allowed total length is also dependent on where in Sweden the bridges are situated. These bridges have proven to be more economical both in construction and maintenance than bridges with expansion joints.

3 Learning from history

Information from elderly inspections has been collected, and put into a database called BEA (Bridge Element Analysis), Racutanu (1998). The information in the database shows how the structural members have served in time. *Fig. 3* shows an example of the development of total condition class (CC) where condition class 3 means that the bridge must be repaired immediately

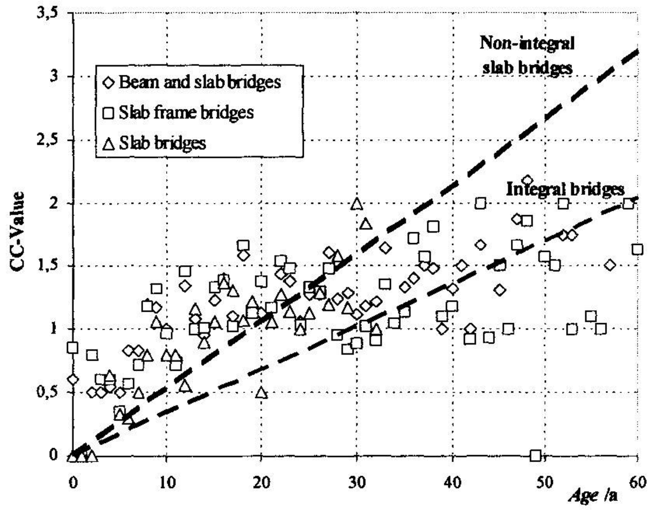

Fig. 3 *Development of damage measured as the equivalent overall condition class for the three compared bridge types. The bridges of integral type slab frame bridges and beam and slab bridges compared with the non-integral type slab bridge. The evaluation shows that the integral type has a slower increase of damage than the non-integral type.*

4 Conclusions

The evaluation of concrete bridges shows that the integral type has proven to be cost-effective and causes lesser maintenance costs than the bridges with expansion joints. The evaluation of the performance of the bridges is done using a detailed database, BEA. This database has proven to be useful to achieve important information about the service life of bridges and their structural members. The historical and empirical information from former bridge inspections can be used to determine the growth of damages in time, certain service environment and bridge generation.

The evaluation of the design methods used, based on simple structural models and experience has proven to be somewhat conservative but adequate.

Progressive Collapse of Multi-Span Bridges – A Case Study

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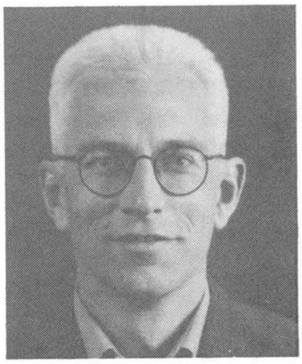

Uwe Starossek, born 1956, received his civil engineering degree from RWTH Aachen, Germany in 1982 and his doctoral degree from the University of Stuttgart, Germany in 1991. He is a director of VSL Korea being responsible for the design of prestressed concrete and composite bridges. He is also a lecturer at Korea University, Seoul.

Summary

The significance of overall structural response to accidental local failure and the possibility of a failure progression throughout the structure are discussed. A progressive collapse study of a multi-span prestressed concrete bridge is presented. An analysis strategy is developed. Analytical results and the ensuing impact on the design of this bridge are discussed.

Keywords: Accidental load; local failure; progressive collapse; robustness; risk theory; response spectrum; dynamic amplification; plastic hinge; conceptual design; detailed design; design code

1. Introduction

Local failure of a structural element may cause failure of another neighboring element. In such a way, failure may progress further throughout the structure. Present design codes give little guidance on how to prevent such a progressive collapse, and designers have to address the problem on a case-by-case basis. The Northumberland Strait Crossing Project, a recently completed multi-span prestressed concrete bridge, is considered as a practical example. A progressive collapse study, performed by the designers of this bridge, is presented.

2. Northumberland Strait Crossing Project

The Northumberland Strait Crossing Project or, as now called, the Confederation Bridge is a prestressed concrete bridge between Prince Edward Island and the mainland of New Brunswick, Canada. The bridge is 12.9 km long. It consists of the main bridge of 43 continuous 250-m spans and approach viaducts on both sides of the main bridge. Possible mechanisms of, and means of design against, progressive collapse of this bridge have been studied.

The conceivable triggers of collapse are manifold. A ship could go astray or an airplane might crash into the bridge; unexpectedly strong ice formations might collide with a pier; etc. In view of the accidental nature of imaginable and unimaginable circumstances, and of the large dimensions of this structure, it would be unrealistic to design against progressive collapse just by preventing local failure at any expense. Instead, the possibility of a local failure must be accepted to the extent that it becomes the starting-point of further investigation.

A collapse triggered by the failure of pier B or pier C should come to a halt, at the latest, at hinge H1 and at pier D (see *Fig. 1*). It is assumed that the bridge girder slides off its bearings at hinges H1 and H2 so that the vertical supports, at these locations, are lost.

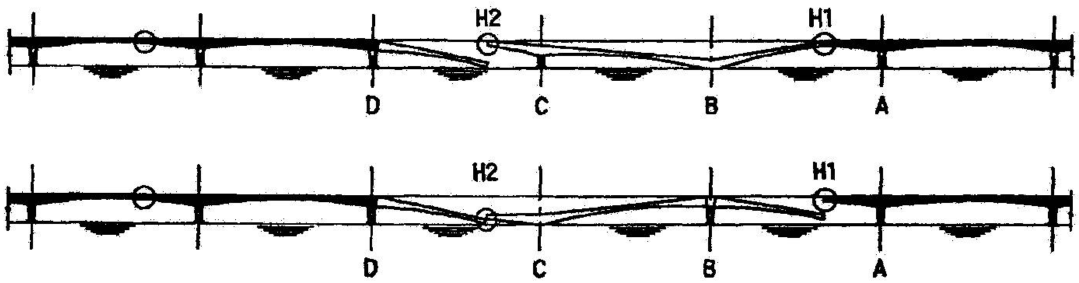

Fig. 1 Working hypothesis on progressive collapse onset

First, the response of the remaining structure to the left of H2, after a sudden loss of this hinge, has been investigated. Analysis indicated that failure of the adjacent span (to the left of D) and, thus, progressive collapse is possible. The structural system has therefore been changed. An additional hinge was inserted into each hinged span (thus having simply supported drop-in girders in every other span). Next, the response of the remaining structure to the right of H1 after a loss of this hinge has been studied. Because of the modification of the structural system, this loss does no longer need to be a sudden event. Instead, sudden loss of the opposite hinge might occur leaving the drop-in girder connected, for some time during its fall, to hinge H1. The vertical hinge force at H1, during this more gradual event, has been analyzed by establishing and solving nonlinear equations of motion (*Fig. 2*). The response of the remaining structure to this force was investigated in a linear time-history analysis and in a quasi-static plastic analysis. The latter method required determination of a dynamic amplification factor. In view of the accidental nature of the considered loading, the formation of plastic hinges was deemed allowable, and the plastic reserves of the structural system were utilized in the detailed design.

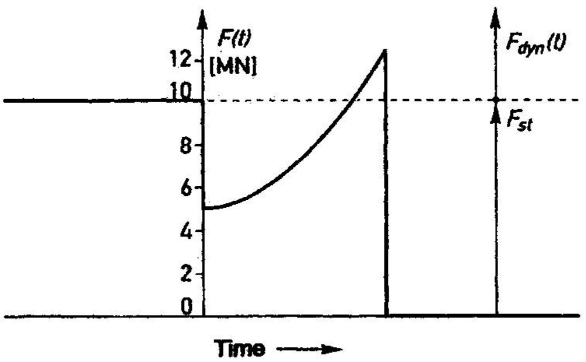

Fig. 2 Vertical force at H1

Further design modifications, due to progressive collapse, range from changes of the post-tensioned and mild steel reinforcement layout, including additional transverse reinforcement at expected plastic hinges, to a change of the superstructure's soffit line.

3. Conclusions

The requirement to avoid progressive collapse in case of local failure is an important design criterion for multi-span bridges. It can have strong impact on both conceptual design, including choice of structural system, and detailed design. Current design codes do not strictly require the prevention of progressive collapse. Recent disasters and theoretical considerations on the basis of risk theory indicate that codes should be improved to more clearly address this problem. In the meantime, owners and engineers should be encouraged to use judgement and discretion to implement the necessary measures even if not yet specifically required by codes.

Performance Validation of Large Seismic Response Modification Devices

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Abstract

In the seismic retrofit design of California's Toll Bridges, seismic isolation is used in several bridges to limit the seismic force input into the superstructure and to avoid costly superstructure retrofit measures which would require partial lane closures and traffic interruptions. Isolation bearings designed for these seismic bridge retrofit projects can have ±1.2 m horizontal design displacements while carrying up to 50 MN of axial load from gravity and seismic overturning effects. Viscous dampers to provide energy absorption and deformation control at movement joints are designed with up to 5 MN capacity. Seismic Response Modification Devices (SRMDs) with these capacities have not been manufactured or tested to date and questions concerning scale-up effects for these response modification devices become critical since the safety of the retrofitted bridge relies on well defined friction and energy absorption characteristics. Only full-scale real-time dynamic testing of these new SRMDs can verify the actual response characteristics and thus validate the structural retrofit concept.

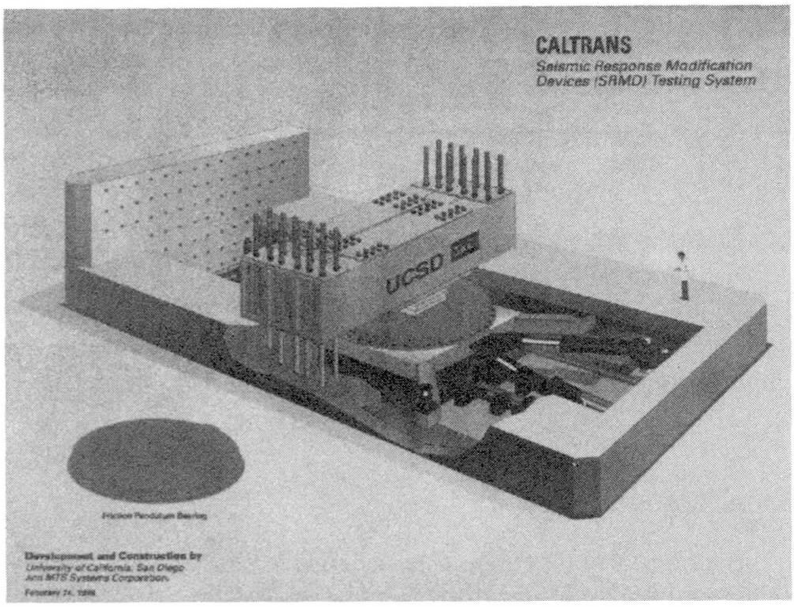

Fig 1 SRMD Test System

Table 1 Technical Specifications

Vertical Force	53,400 kN
Longitudinal Force	8,900 kN
Lateral Force	4,450 kN
Vertical Displacement	±0.127 m
Longitudinal Displacement	±1.22 m
Lateral Displacement	± 0.61 m
Vertical Velocity	±254 mm/s
Long. Velocity	±1,778 mm/s
Lateral Velocity	±762 mm/s
Height of Specimen	Up to 1.52 m
Relative Platen rot.	±2°

The Caltrans Seismic Response Modification Device Test System (SRMD), developed jointly by Caltrans, UCSD, and MTS, requires an exceptional hydraulic power, equal to about 19,000 liters (5,000 gal) of oil storage and pressurization through nitrogen gas up to 34 Mpa (5,000 psi), in order to achieve the technical specifications indicated in Table 1. The machine performance, the different types of isolating devices and dampers to be tested, and the testing objectives and procedures, drove the overall design approach. The isolating devices considered were the Friction Pendulum System (FPS) and high damping rubber or lead core elastomeric isolation bearings, proposed solutions for the Benicia-Martinez and Coronado bridges, respectively, as well as viscous dampers and lock-up devices, common to almost all the toll bridge retrofit projects. Testing objectives ranged from slow speed uni-directional testing for basic performance characterization to high speed, 3-D testing for energy based analysis. The investigation of the effect of wear and aging was also an important issue in the proposed test program, through re-characterization of the performance of the prototype SRMDs already exposed to the actual bridge loads, deformations and environmental conditions. The testing system consists of a horizontal prestressed concrete reaction frame (concrete box), and of a moving platen, connected by four horizontal actuators to the concrete box. The platen slides over four hydraulic hydrostatic low friction bearings attached to the floor of the concrete structure. The platen also extends with four steel outrigger arms that support four low friction-sliding actuators at their tops. The testing system is completed by two additional reaction structures: a steel cross beam, removable and linked to the concrete box through a tie-down rod system, and a heavily prestressed reaction wall on one end of the machine. Due to the large displacements of the test specimens in the longitudinal and transverse directions, the traditional solution of a platen with hinge connection to horizontal and vertical actuators was not practical. This configuration would have required very long stroke vertical actuators, with deep excavation inside the existing building. Space limitations, difficulties of access for tall excavating equipment and the need to maintain the rest of the existing laboratory in working condition during construction made this solution impractical. The adopted solution was the use of 4 vertical hydraulic sliding bearings which support the moving platen, apply the vertical load, and allow horizontal motion and swivel capacity with very low friction (less than 0.2% of vertical force). The four horizontal actuators have a 2.5m stroke, 800 ton capacity and dual $20m^3$/min. servo-valves, and the 4 vertical low friction bearing actuators have 0.25m stroke, 2,000 ton capacity and $11m^3$/min. servo-valves each.

This paper describes the design, construction and performance characterization of a full-scale testing facility (Fig. 1) which will allow the real-time 6-DOF dynamic characterization of these new generation of seismic response modification devices for long span bridges.

Masonry Reinforced with FRP - Walls with Openings

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Abstract

Plain masonry is considered one of the oldest construction materials. It can be traced back to ancient Egyptians who built the pyramids. Since then, a lot of masonry buildings were built with unreinforced masonry all around the world.

Unsatisfactory seismic performance of unreinforced masonry has been observed in earthquake zones around the world. This is attributed to its limited tensile and shear strengths. Unreinforced masonry walls are very stiff and exhibit a brittle response to lateral loads. Innovative methods of repair and seismic upgrading of unreinforced masonry walls are urgently needed. In response to such a need an ongoing comprehensive research program has been initiated at Helwan University to investigate the effectiveness of fiber reinforced plastic (FRP) as small fibers, added to the mortar, or laminates, glued to surface of the wall, to repair and strengthen unreinforced solid masonry walls with openings. The program was divided into two phases as follows:

The first phase was designed to study the improvement in tensile and shear strength of walls with openings when strengthened or repaired by strips of (FRP). The behaviour of model small assemblages, plain and strengthened with FRP, under in-plane joint shear and diagonal tension can represent possible modes of failure for unreinforced shear walls with openings. A total of nine walls are constructed and tested under diagonal splitting tension. The parameters studied are the opening/wall ratio and the method of repair or strengthening with FRP laminates.

The second phase was designed to study the behavior of shear walls with windows and doors under the action of in-plane shear. Thirteen half-scale masonry solid shear walls are constructed to be tested under vertical and lateral loads. The effect of openings, their size and location within the walls will be studied. The strategy of strengthening and repair of these walls is as follows: some of the walls are tested until a certain level of damage is reached then repaired with FRP laminates glued to the walls on both sides at the cracked zones only and retested again. The rest of the walls are strengthened using FRP before testing.

The results of the first phase only are presented in this paper. These results demonstrate that the FRP laminates are very effective in increasing the strength and the deformation ability (ductility) of these walls without significant increase in their stiffness or weight. Although the fiber plastic type is rather cheap and weak, it adds significantly to the wall's strength as the wall is originally very weak in tension and shear. Another advantages of this method of repair are the ease of application of this type of repair (no need for experienced labor) and the fast execution. Hence, this method of repair and strengthening can be considered as an adequate method for seismic upgrading of existing structures and for increasing shear strength and ductility for future structures. The main strength of this method of upgrading is that strips can be glued to zones of weakness or of expected plastic zones without the need of high technology. In the cases that need repair it can be glued to the cracking zones and around opening without the need to evacuate the building.

The New Airship Hangar in Germany

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1. Summary

Currently, a new generation of airships, the CargoLifter CL 160, is developed. Therefore a new hangar for two airships is going to be built. With a span of 210 m, a height of 107 m and a length of 363 m it will be the largest hall in Germany. The central part of the hangar is of a cylindrical shape consisting of five steel arches covered with a textile membrane. At both ends of the building are the doors which consist of two fixed and six moving elements. They form a semicircle in plan and a quarter-segment of a circle in elevation.

2. Introduction

Six decades ago the golden age for the airships - succeeded by air planes- seemed to have died forever. But the global market and ecological needs require the search for new forms of transportation. Currently, a new generation of airships (helium filled, carbon fibre structure, length 260 m), the CargoLifter CL-160 is prepared. This airship allows transporting a pay-load of 160 t over a distance of up to 10.000 km with a speed of 80-120 km/h. For more information see http://www.cargolifter.de.

In the middle of 1997 the CargoLifter AG commissioned the design for an airship hangar in Germany to house two new airships. The site is an airfield in Brand, about 50 km south of Berlin. With a span of 210 m, a height of 107 m and a length of 363 m it will be the largest hall in Germany (Fig.1). Currently, the detailed design is taking place and most of the building structure should be ready in 1999.

3. Description of the airship hangar

The structural concept distinguishes two main parts of the building. The central part is of a cylindrical shape consisting of five steel arches at 35 m centres - each of the four bays being covered with a textile fabric - and at both ends of the building are the doors which consist of two fixed and six moving elements. Both doors form a semi-circle in plan and a quarter-segment of a circle in elevation. The shape of the building is oriented closely on the clearance diagram for two airships. In addition, this solution is suitable to avoid excessive wind turbulence. The structure has been designed using steel grade S355.

The arches of the cylindrical part have a structural height of 8 m and span over 225 m. The top chords (diameter 559 mm) are at 3,441 m centres and the bottom chords (559 mm) at 2,0 m centre. All the chords of the truss-arch are brace-connected to each other by diagonals (355 mm) and posts (273 mm), with the exception of the two bottom-chords; these are connected by only straight members (355 mm) at 4,135 m, forming a Vierendeel-system.

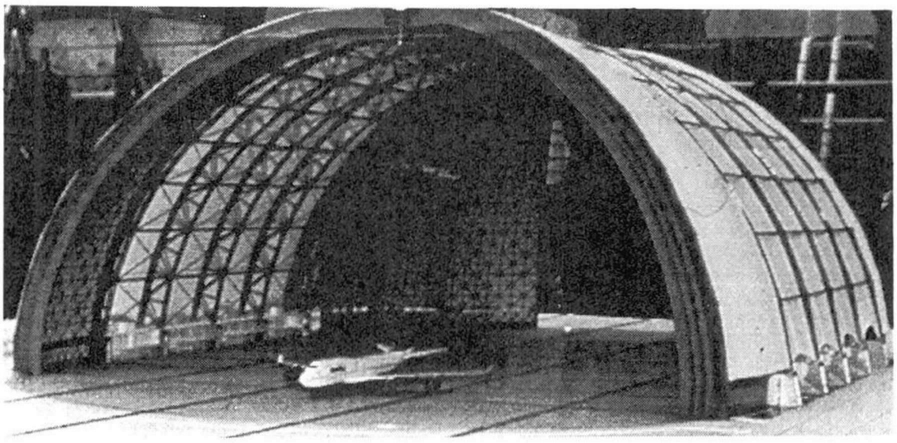

Fig. 1 Perspective of the hangar (model SIAT)

All elements are tubular hollow sections, used because of their high torsional resistance and their good buckling performance. The section thickness varies between 10 and 95 mm. The tube connections have been modelled as fully fixed joints. Currently, analyses are done to investigate the influence of semi-rigid connections on the buckling behaviour of the arches.

The wind bracing is pinned to the bottom chords at each intersection of two arch-polygon segments and pretensioned by 50 kN. At the same intersection between the top chords, external props restrain any torsion in the arches, induced by the eccentrically connected membrane.

At their ridge the arches are longitudinally connected by a four chord, 8 m deep truss, similar to the structure of the arches. This ridge beam enables the connection of the membrane and the valley cable at the top, and takes up the large compression force between the two end arches, generated by the doors.

The hangar entrances are located at both ends of the central part of the building. They consist of a shell structure with a spherical surface. The quarter shells, thus formed, are subdivided into 8 parts. The two fixed bays adjacent to the cylindrical part are more or less continuously connected at one side to the end arches. For these sections, a reticulated structure of 9 by 6 m^2 of HE500A-sections form the structural system. The other six bays of the sphere form the moveable door segments.

The door weight strongly influences directly and indirectly the hangar costs so that minimising the door tonnage was a main aim. The lightest structure was achieved by adopting a shell principle. The inner part of a door segment consists of a spherically shaped frame structure, realised by identical horizontal, vertical and diagonal elements (HE 240A sections), which are rigidly connected in their joints. The cladding spans between the horizontal elements. The side beam dimensions are 3000x800x30x10 [mm], with T90 stiffeners in a 3 by 1 m^2 grid against local buckling. More likely, the edge beams will be manufactured as trapezoidal corrugated web profiles (TWP) with dimensions 3000x800x30-40x5 [mm]. Although the webs are only 5 mm thick due to their folding no additional stiffeners are required. The shell is eccentrically connected to the side beams: at one side to the top flange and at the other side to the lower flange. Hence, a horizontal section through a door shows an approximate Z-shape, allowing for the overlapping of doors underneath each other. At the bottom the shell joins the lower horizontal beam (2300x800x20x15 [mm]) concentrically.

The building enclosure is achieved using a stressed coated polyester fabric type V membrane spanning between the trussed tubular arches in the warp direction and between the ridge truss and the 65 mm edge cable attached to the arch bases in the weft direction By adopting a 65 mm valley cable midway between the arches and by form finding the surface with an increased prestress in the weft direction, the warp span is decreased and the surface stiffened in the weft direction even though the overall curvature is decreased. This way inversion of the outer membrane under wind loading can be controlled.

In addition to the large amount of steel, large amounts concrete are also used for the hangar, about 20.000 m^3 in total. The main concrete elements are the foundations of the arches and the doors, and the floor slab. Between the concrete bases of the arches a two storey high office area is planned with an office height of 3.96 m. Single slab foundations are used for the arches in the cylindrical part.

Detailed information about loading, analysis and design is given in the full-length paper.

Mass-produced Structures of Multistorey Houses

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Summary

The design and structural solution of a multi-storey building system and superstructures using thin-walled three-dimensional steel units is an example of a structurally and technologically advanced type of designing structures for the future. Among its structural advantages there are e.g. a considerable ductility of the system and its resistance to seismic impacts. The load-bearing system of buildings is created by a composition and mutual interaction of three-dimensional steel units with contact bearings through horizontal bonding brackets. Superstructures designed for seismic or undermined regions are characterised by the concentration of the "mass" of the structure close to the core (centre) or by elastic connection of some parts of the bearing system, reducing thus the loading due to seismic effects, the effects caused by the differences in the building's settlement etc.

Keywords: mass-produced structures, thin-walled three-dimensional steel units, horizontal connecting brackets, superstructures, troubled geological and seismic regions, thin-walled gable profiles, loading tests, dynamic characteristics, extreme loading

1. Constructional and structural design of a multi-storey building system and superstructures from thin-walled three-dimensional steel units

According to the structural arrangement of three-dimensional units, a number of variable structural systems of multi-storey buildings or, in combination with other structures, the so called superstructures, may be designed. Horizontal bonding brackets act as bonding elements between individual three-dimensional units. They are "loosely" laid on the corner units' joints forming continuous "surfaces" for mounting upper three-dimensional units. Horizontal brackets stabilise the structure in the mounting horizontal plane, providing, at the same time, distance intervals between the units in both directions.

Supersystems with three-dimensional units are characterised by more distinct structural principles of a superstructure. In high-rise superstructures it is, as a rule, the effects of horizontal loads that have a decisive share in the concept of the loading structure design. Superstructures designed for seismic or undermined regions are characterised by the concentration of the "mass" of the structure close to the core (centre) or by elastic connection of some parts of the bearing system, reducing thus the loading due to seismic effects, the effects caused by the differences in the building's settlement etc.

The application of three-dimensional units within a supersystem allows for their wider and more variable usage. A superstructure creating "construction surfaces" for the multi-storey building unit as a whole, acts, at the same time, as a "compensation member" between the effects and impacts (force, deformation) of the outside environment and the purpose-built structure of three-dimensional units itself. This allows to design and manufacture unified building structures, implementing them in quite varied conditions, to erect buildings in troubled geological and seismic regions, in territories with extreme traffic requirements etc.

2. Load-bearing structure of a three-dimensional unit

The load-bearing structure of a three-dimensional unit consists of thin-walled steel profiles, cold formed into rectangular gables. In its transversal direction, a wall three-dimensional unit section is shaped as an enclosed plate frame with rigid joints. Corner joints are formed by mutual sliding of perpendicular gable profiles onto each other and by their subsequent connection by spot welds along the contact surfaces of profile walls. The vertical load-bearing structure of wall sections is formed by gable section profiles, similarly to the ceiling and floor structures.

Thin-walled gable profiles are mutually interconnected to form a three-dimensional load-bearing structure by spot welds on the production line, without using any other auxiliary or reinforcing profiles. After the thin-walled steel gable profiles have been welded to form a load-bearing structure on a production line, the structure is sprayed with protective anti-corrosion and fire-proof agents and it is gradually fitted with distribution frames and installations, insulation and facing slabs etc.

The sections' dimensions have been designed with regards to the functional and transport requirements (the sections' width being 2,400 mm to 2,700 mm, their depth 5,100 mm to 7,500 mm and their height 2,950 mm to 3,400 mm). The weight of a steel structure of a three-dimensional unit (section) with a net floor area of 14 m^2 is approximately 750 kg - 1050 kg, the weight of three-dimensional column unit approximately 550 kg - 810 kg, depending on used profiles.

The three-dimensional steel structure was exposed to a series of loading tests. The static and dynamic loading tests were aimed at the verification of theoretic computation estimates, the joint's functioning and comparison of theoretically computed and measured values. The test results show a very high level of compliance with theoretic computations. No failure of the structure occurred and not a single case of reaching the plastic deformation zone was recorded during the tests. The results of verification tests and structural computations proved the feasibility of application of three-dimensional steel units in the construction of 8- to 12-storey buildings. The analysis of dynamic behaviour of a multi-storey building composed of three-dimensional steel units has shown satisfactory characteristics of the load-bearing system.

The relative rigidity of elastic bearings at the joints between three-dimensional units, as compared with the relative rigidity of enclosed plate frames, makes it possible for the bearings to absorb a considerable part of deformational energy (seismic effects, gusts, shocks etc.) when subjected to dynamic impacts, reducing thus the stress and deformation of the three-dimensional steel units themselves, so that they do not exceed the elastic impact zone. In this way the failure and loss of mechanical resistance of the load-bearing structure due to effects of extreme loads may be prevented.

Experimental measurements on a testing site have shown excellent characteristics of the structure from the point of view of acoustic comfort and energy demands.

Structural Response of Represtressed Preflex Beams and Box Girder Bridges

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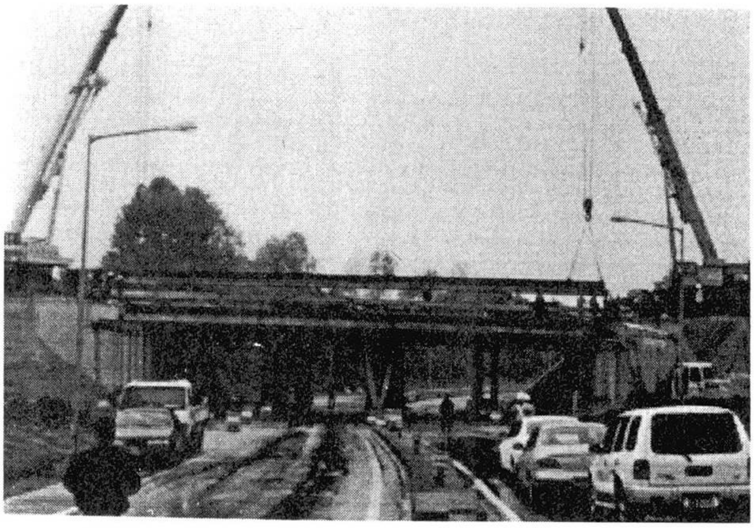
Summary

The conventional preflex beams are based on the method of partial prestressing, and allow the tensile stresses at the lower concrete of beams. As a consequence, many preflex beams experience the tensile cracks under the service loads.

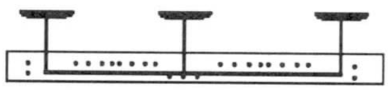
Introducing additional prestressing called 'represtressing' at the lower concrete of beams, there could be accomplished control of crack problem. For analysis and design of experiment and improved beam, Design and Analysis Program of Represtressed Preflexional Composite Beam (DARP), was developed. In the experiment, two actual sizes of represtressed beams were tested under the imitated service loads. The results of test have shown that the performance of the represtressed preflex beams(RPF) are generally excellent for straight simple beams.

Although RPF do not experience the tensile cracks under the service loads, the use of this beam for the bridge structures couldn't apply for skewed bridges.
Therefore, newly invented box-type bridge, Represtressed Preflex - Box Girder Bridges (RP-Box Girder Bridges) are suggested for the skewed or long span continuous bridges. Adding the characteristic of low beam depth, high clearance and economical design of RPF, this suggestion of RP-Box Girder Bridges, is expected to contribute to practical and economic construction of bridges.

Keywords: represtressed preflex box girder bridge, preflex, tensile crack, ratio of displacement, torsion strength, bridge structure



Construction site of RPF, located in Jeonju, Korea

Type of Bridge	Developed Shape	Merits
Preflex 1949, by Lipski (Belgium)		Low depth ratio to 1/40
RPF 1995, by KICT (Korea)		Low depth ratio to 1/40 Full prestressing by strand
RP-Box 1998, by KICT (Korea)		Low depth ratio to 1/40 Full prestressing by strand Large twisting rigidity

The history of development of preflex composite bridges

From Corrosion of Existing to Durability of New Concrete Structures

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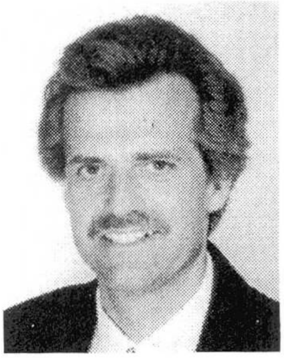

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Summary

The problem of reinforcement corrosion in new concrete structures can be avoided if measures are taken to increase the corrosion initiation time. These measures include providing adequate concrete cover thickness, ensuring a low permeability of the cover concrete, and limiting early age concrete cracking. As adequate concrete cover is normally not a problem, this paper highlights the findings of two studies that focus on the latter two measures. The first study investigated chloride ingress under given climatic conditions and evaluated the feasibility of measuring the permeability of concrete cover in situ. The second study used numerical models to investigate the effects of early age cracking and determine measures to be taken to limit their development during construction. The principal findings of both studies are given herein. It is concluded that permeability measurements should be conducted for quality control of the cover concrete of new structures and that early age cracking due to the hydration of young concrete may be limited by reducing the difference between concrete and ambient temperature.

Introduction

Reinforced concrete structures have generally shown satisfactory performance in terms of strength and durability. Under certain climatic conditions, however, reinforced concrete structures have suffered from corrosion damage of steel reinforcement (rebar) resulting in premature and expensive repair work.

Damage to concrete structures is the result of the interaction between the material and the environment. Gas, liquid and ionic transport mechanisms play a major role in the concrete deterioration. The nature of the damage can be chemical, such as rebar corrosion or sulfate and acid attacks on cement paste, and physical, such as abrasion or freeze-thaw cycles (Fig.1). Both the quality of cover concrete and the presence of cracks (especially those which allow water seepage) have a predominant influence on the durability of a concrete structure. Concrete not only has a load bearing function, but also acts as a protective coating for the steel reinforcement.

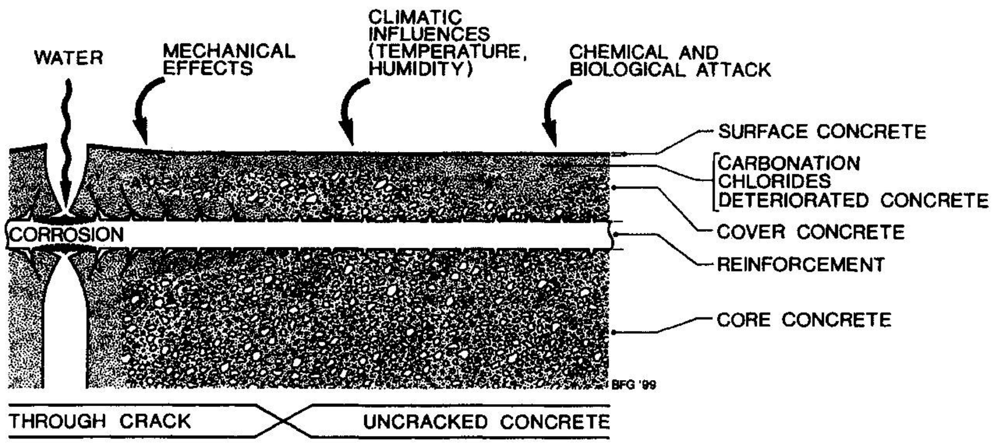

Fig. 1 Vulnerability of reinforced concrete

The majority of research on the deterioration phenomena affecting reinforced concrete has concentrated on material behaviour. The ultimate limit states for structural safety and serviceability, however, are based on structural considerations. These aspects must be combined. With this combination it is important to analyse the deterioration mechanisms considering both the observable steady state and a time evolutionary point of view.

This paper investigates measures to reduce the risk of occurrence of rebar corrosion that may be used in the construction of durable and low maintenance concrete structures for the future. After a review of the relevant aspects of rebar corrosion, this paper investigates:
- the roles of the permeability of cover concrete and
- early age crack formation in delaying the onset of corrosion.

Conclusions

1. The objective of measures to reduce the risk of rebar corrosion should be to increase the corrosion initiation time until it is greater than the service life of the structure. Three methods to reduce this risk are (1) to provide sufficient concrete cover thickness, (2) to provide dense surface and cover concrete of low permeability, and (3) to limit early age cracking of concrete. These measures depend on the climatic exposure of the structural element.

2. Numerical models allow approximate prediction of the initiation phase for rebar corrosion and early-age cracking of concrete elements. They are reliable tools to evaluate time evolutionary issues of concrete structures and therefore investigate the effectiveness of measures to ensure durable new concrete structures.

3. Permeability measurements should be conducted for quality control of new concrete structures. They are reliable with adequate adjustments for moisture.

4. Cracking due to hydration of young concrete increases concrete permeability and thus its vulnerability to rebar corrosion. Early age cracking may be limited or even avoided by reducing the difference between the temperature of new concrete and ambient temperatures.

Effect of Strength of Concrete on Corrosion of Reinforcing Steel

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Abstract

Corrosion is one of the major problem in R.C.C. Construction facing the civil engineers these days. Chloride intrusion and the carbonation are the two extensive deteriorating processes that cause the depassivation of steel embedded in concrete leading to damage to concrete structures and the costs of repair become substantial.

The corrosion process of reinforcing bars in concrete is a function of number of variables such as environmental factors in which the concrete is used, steel surface at the steel concrete interface, and concrete properties etc. The corrosion of metals, especially reinforcing steel, in concrete has received increasing attention in recent years because of its widespread occurrence in various types of structures and high cost of repairs.

The assessment of reinforcement corrosion have been the subject of extensive study and research during recent years. In most of the related literature, the risk of corrosion of reinforcement in concrete is assessed by measuring the half cell potentials, concrete resistivity values, impedance etc. The measurements by these methods only represent the qualitative estimation with certain limitations. The classical measuring method for recording the rate of corrosion is to measure the weight loss, which involves recording that, which is sought, namely metal loss, unlike above methods which usually provide relative values.

The high alkalinity of cement paste offers satisfactory protection against corrosion of steel reinforcement in concrete. This protection is due to existence of self generating protective layer at the steel concrete interface. The protective layer has normally been considered to be a tightly adhering film of γ-Fe2O3. The steel reinforcement is further protected by the physical shielding afforded by the concrete cover, which lowers the steel susceptibility to corrosion. However despite these protective mechanisms, a decline in the durability in reinforced cement

concrete structure as a result of corrosion is not uncommon. This can be readily envisaged in the light of the diverse and often severe conditions of the environment and service encountered by the structures. Cases of particular relevance include, chloride attack and carbonation. These two mechanisms are unusual in that they do not attack the integrity of the concrete. Instead, aggressive chemical species pass through the network of pores, often with substantial interconnectivity, into the concrete and attack the steel.

In this paper an attempt has been made to study the effect of compressive strength of the concrete on the corrosion of three grades of steel, viz., the smooth round Mild Steel bars, HYSD bars of grade Fe 415, and CRS (Corrosion Resistant Steel), in a laboratory accelerated corrosion test by classical mass loss measurement technique which is the only method to assess the corrosion rate quantitatively.

With the help of the study conducted, it can be concluded that the concrete quality and the thickness of concrete cover largely determine the effectiveness of general defensive shield against corrosion. The mass loss of the steel are significantly influenced by strength of the concrete. The rate of decrease of the mass loss for all types of steel are maximum up to concrete strength 35 to 40 MPa. Beyond this strength the mass loss of the embedded steel decreases but at a relatively slower rate. The study also brings out that the higher carbon content of the steel exhibit more mass loss due to corrosion. Also Mild Steel bars are capable to extend the time to initiation of the corrosion process in concrete but exhibit relatively a large amount of pitting type of corrosion, with areas of rusting separated by clean steel, as compared to HYSD steel and CRS bars. The performance index of CRS lies in the range of 1.3 to 1.5 and this index increases as the quality of concrete increases and decreases with the time of exposure in LACT.

The data developed in the proposed study show the value of concrete strength in resisting the corrosion of different types of reinforcing steel. The data will help in the selection of reinforcing steel in conjunction of strength of concrete in chloride corrosion situations.

Performance of Concrete Structures in Argentine Environments

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Introduction

Reinforced concrete. performance throughout its useful life is related to the intrinsic characteristics of its component materials, their interrelationship and the nature and aggressive characteristics of the surrounding environment.

Theoretical knowledge about the causes and mechanisms of portland cement concrete failure, as well as of the techniques to minimise or eliminate their consequences, is available. Laboratory experiments and concrete performance in different environments have been extensively reported in the literature. It should be borne in mind that portland cement concrete has been used for over a century and there are currently constructions that date back to that age.

As regards reinforced concrete, field experience is much more limited since its intensive use is restricted to the past five decades and, therefore, most of the structures in service are not of significant age. Rebar corrosion is the most serious problem.

This paper is part of a research project that has been carried out for many years. Its objective is to study the field performance of structures built in the province of Buenos Aires, Argentina, located in different environments (rural, urban and industrial environments, and urban-marine subenvironments) in order to contribute to the knowledge of the "in situ" performance of the material.

Environmental characteristics

Temperature, rainfall and relative humidity conditions, as well as the number of clear, overcast and rainy days (National Weather Forecast - 1971-1980 period) for rural, urban and urban-marine environments are reported. Mean annual temperatures are in the range 14 - 16°C. Industrial environments are characterised by the presence of contaminants from chemical, petrochemical, and siderurgical industries.

Evaluation methods

Evaluated structures, located in a rural environment and urban-marine subenvironment, were selected at random, considering only the characteristics of the environment to which they were exposed. The evaluations of structures located in urban and industrial environments correspond to studies conducted according to the deterioration found. A visual inspection was made first in all cases. Depth of carbonation and rebar cover thickness were determined in some instances.

Concrete cores were extracted from some structures to study the mineralogical and petrographic characteristics of the aggregates. Tests were performed by spraying uranyl acetate to detect possible signs of the alkali-silica reaction (ASR). These tests were supplemented with analysis under an optical microscope. The physical, chemical and/or mechanical properties of the material were also established. The working scheme used follows the guidelines set by *DURAR Manual de Inspeccion, Evaluacion y diagnostico de Corrosion de Estructuras de Hormigon Armado.* CYTED.

Concluding remarks

From these findings, which will have to be supplemented with new surveys of structures, it can be concluded that:

a) Technological developments over the past years, especially those related to concrete performance in different aggressive environments, are not applied adequately in the field.

b) Most of the structures evaluated in different environments, which were constructed before 1950, exhibit adequate field performance, no rebar corrosion processes or concrete damage being detected.

c) In structures exposed to rural environments (moderately aggressive) signs of rebar corrosion can be detected at ages under 20 years due to lack of or insufficient cover depth that is carbonated. In structures located in aggressive environments (marine or industrial), the problem is more critical.

d) In structures executed with aggregates petrographically classified as non-reactive, no signs of reaction are detected, as expected. In other structures, where aggregates with high strained-quartz contents and undulatory extinction angles greater than 20° were used, external reaction signs (cracks, gels, etc.) appear at greater ages when the alkali content is high. It has been found that some aggregates have satisfactory performance in the structure although in accelerated tests they exhibit high reactivity or fall into a classification where late reacting deleterious aggregates coexist with others of innocuous characteristics.

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High Performance Concrete in the Argentinean West Central Area

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Keywords: HPC, additives, natural pozzolan, pozzolanic activity, mechanical properties.

Abstract

Active mineral additions, as components of concrete, make possible the enhancement of the mechanical resistance of the concrete. Besides, it allows preparing concrete with better durability properties, i.e. resistance against aggressive agents and lower development of hydration heat. Anyway, it is possible to get significant energy savings and design purpose made mixtures for specific structures located in special areas. This preparation of mixtures is what we call in this work HPC.

A lot of research works exist in relation to the use of active mineral additives (micro-silex, fly ashes, metha-kaolin, etc.) in the elaboration of HPC. Physic-chemistry properties of these additions are variable and the action mechanism is controlled by the pozzolan reaction. The mechanical properties of the HPC, also depend on the characteristic of the granular materials used. There are considerable differences on the mechanical behaviour of HPC, inclusive with the same resistance, depending on these variables and on where they are been manufactured [1]. The characterisation of natural pozzolan of volcanic origin and diatomaceous ground extracted from local quarries, of the area of the Andes mountain range (San Juan and Mendoza provinces) in the region of "Cuyo", (the region of Cuyo is located in the Argentinian West Central area) with the purpose of being employed, as active mineral additives, is shown in this work. Preliminary geologic surveys, specimen collection and tests for physical chemical characterisation were also carried out.

The most interesting samples were selected to investigate the effect of fineness and granulometry on the pozzolanic activity. In addition, some concrete samples were made to evaluate traction and compression mechanical properties and their evolution in time. All the results of these samples are also presented. The work presented in this paper is part of a greater research project which aim is the evaluation of the technical and economical possibilities for using this HPC, made with local materials, in building structures of the Argentinian West Central area. This area is, on the other hand, the zone of maximum seismic risk in this country.

Nevertheless, it is necessary to investigate further in the optimisation and study of structural elements under different load patterns, in order to use this HPC in the Cuyo region. As we mention before this is a high seismic region. The following conclusions of the first results can be pointed out:

- The pozzolanic activity of the samples studied has been tested and proved.
- The addition of active raw materials from the geographic area can be considered to be apt to elaborate concrete of especial properties. For instance, durability, i.e. sulphate attack resistance, and to control the hydration heat dissipation.
- Despite of having a lower efficiency than concrete prepared with microsilex, still it is possible to use the Pozzolan to make HPC.
- The cost of the quarry is around $2 per ton. From the economic point of view is very interesting and can be considered to be apt.
- The fresh concrete has a slightly higher viscosity compared with the ones without pozzolan, but nevertheless is apt to be placed through difficult access places.
- Hardened shows increasing resistance up to one year, with a higher increase in the beginning due to the type of cement used.

Aknowlegments

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Durable Watertight Lining System for Tokyo Wan Aqua-Line Shield Tunnel

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Abstract

Tokyo Wan Aqua-Line, a dual highway with two lanes in each direction, was constructed across Tokyo Bay which sits on an active earthquake area. Its total length is 15.1 km, of which 9.4 km is for the tunnel section, 2x0.2 km for the man-made islands, 4.4 km for the bridge and 0.9 km for the land access.

The tunnel section, consisting of two parallel tunnels, was driven through very soft ground 9.4 m to 18 m below the seabed under 0.6-MPa water pressure. Eight world's largest slurry shield machines with a diameter of 14.14 m were used for boring. To shorten the construction period, the shield machines were advanced from the vertical shafts both at the land area and the man-made islands. Underground shield machine docking was carried out below the seabed under high water pressure.

To achieve durability and watertightness of tunnel linings under such a severe condition, special considerations were given to various items from mix proportion of segment concrete to membrane waterproofing.

The authors will present these considerations including the following:

1) Admixture for segment concrete
   Blast-furnace slag powder was mixed to give concrete a lower coefficient of permeability, thereby increasing its anti-corrosion.
2) Backfill grout holes
   The number of holes was limited to four per ring. And to give watertightness, a butyl-based, hydro-expansive rubber seal was set around the grout pipe.
3) Watertight gasket
   A hydro-expansive seal, which would swell when it got wet, was set along each liner segment.
4) Steel materials
   -The covering of iron bar for the segment was set at 5 cm.
   -A semi-permanent anti-rust coating was applied to the segment bolts.
5) Shield machine docking below the seabed
   Skin plates of the machines were connected to one another by welding. Some flexibility was provided in the joint section.
6) Watertight membrane
   A watertight membrane was laid between the primary and secondary liners to lead unexpected water leak down for drainage.

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Indirect Loading and Indirect Supporting in High-Strength Concrete Beams

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Abstract

In concrete structures, most frequently, the loads and reactions are applied on the top and the bottom of the beams, respectively. These are said to be directly loaded and directly supported beams. However, situations are found in which the beams are loaded or supported by intersecting beams so that the load transfer is by shear rather than by bearing on the top or the bottom surface. These are referred to as indirectly loaded or indirectly supported beams.

The experimental investigations conducted during the past decades with normal-strength concrete beams showed that a tie shall be provided at the joint between these beams which is responsible for transferring the mutual reaction from the supported beam to the supporting beam. This transfer is accomplished by a reinforcement made up by stirrups referred to has hanger reinforcement.

This abstract is concerned with indirect loading and indirect supporting in high-strength concrete beams and reports the results of experimental investigation conducted by *Ribeiro* (1), under direction of the author, using horizontal T-shaped specimens. The specimens were B1-L and B1-R with shear span-to-depth ratio a/d = 1.89 and C1-R with shear span-to-depth ratio a/d = 3.92. The beams were designed to fail in flexure. The flexural strength was given by the supporting beam. The predicted ultimate load was $2F_u$ = 140kN and the corresponding mutual reaction was A_u = 1.2 F_u = 84kN. The specimen concrete average compressive strength at the age of the tests was about 80MPa.

The hanger reinforcement in specimen B1-L was made up by one 5.0mm diameter stirrup in the supported beam (beam B) and four 4.2mm diameter stirrups in the supporting beam (beam 1) arranged within h/2 of the center of the intersection of the beams. This reinforcement could hang up 144% of the predicted ultimate mutual reaction: 34% for the stirrup in beam B and 110% for the stirrups in beam 1. The stirrup in the supported beam was responsible to hang 24% of the effective mutual reaction and those in the supporting beam were responsible to hang 76% of the effective mutual reaction. The two stirrups in the common volume could hang 55% of the predicted ultimate mutual reaction and were responsible to hang 38% of the effective mutual reaction. The hanger reinforcements in specimens B1-R and C1-R were made up by one 8.0mm diameter and two 6.3mm diameter stirrups placed at the common volume of the beams. This reinforcement could hang 137% of the predicted ultimate mutual reaction and was responsible to hang the full load transferred.

The tests showed that, initially, the load transfer is by shear over the depth of the beams. After cracking, the cracks produced by the principal tensile stresses in the supported beam follow the principal compressive stress trajectories. The inclined cracks in the supported beam give rise to a compressive stress field which acts as struts in a truss-like system. The stirrups crossing the cracks

play the role of ties in the web The load on the supported beam is transferred to the tensile chord of the supporting beam by the last strut in the supported beam.

The tests showed that the supporting beam is initially uncracked while the load transfer is by shear. After cracking, the cracks produced by the principal tensile stresses give rise to a fan-like crack pattern similar to that known for directly loaded beams. At late load stages, horizontal cracks appear over the web of the supporting beam indicating a tie-like behaviour of the hanger reinforcement.

In specimen B1-L, at ultimate load $2F_u$ = 170kN the mutual reaction was A_u = l02kN. At this load, the four stirrups in the supporting beam were hanging up 79.3kN ($0.77A_u$) and the single stirrup in the supported beam was hanging up 23.6kN ($0.23A_u$). At the same load the two stirrups in the common volume of the beams were hanging up 47.7kN ($0.46A_u$) and the next two were hanging up 31.6kN ($0.31\ A_u$). The two stirrups in the common volume of the beams started yielding at a load 2F = 160kN and one of the next two had started yielding at 2F = 150kN. At these late load stages horizontal cracks appeared on the web of the supporting beam at the intersection of the beams due to stretching of the hanger reinforcement. At the same time a fan-like crack surface started to outline in the web of the supporting beam at the intersection of the beams. This crack surface was bounded by wide inclined cracks due to large yield strains in the hanger reinforcement. At a load 2F = 175kN, just after the compressive chord had initiated crushing, the bottom of the supporting beam was suddenly pushed downward. This fact enhances that this hanger reinforcement did not provide effective support for the bottom of the supporting beam at the intersection of the beams, even tough this reinforcement could hang up 119% of the effective mutual reaction: 28% for the stirrup in the supported beam and 91% for the stirrups in the supporting beam. This is because the hanging stirrups placed farther from the intersection are not as efficient as those in the common volume of the beams. The stirrups in the supporting beam were able to hang up only $0.\ 77A_u$ ($0.46A_u$ for the stirrups in the common volume of the beams and $0.31\ A_u$ for those in outside) and needed the help from the stirrup in the supported beam to hang up the missing $0.23A_u$.

In specimen B1-R, at ultimate load 2Fu = 170kN the mutual reaction was A_u = 102kN. At this load, the three stirrups were hanging up 106.7kN. This load is approximately equal to A_u. These stirrups started yielding at a load 2F = 160kN. At the late load stages, the same horizontal cracks and the same fan-like crack pattern appeared on the web of the supporting beam but no pushing-down failure occurred. The hanger reinforcement placed at the common volume of the beams was efficient up to failure of the specimen.

In specimen C1-R, at ultimate load $2F_u$ = 1 60kN the mutual reaction was A_u = 96kN. At this load the three stirrups were hanging up 100.4kN. This load is approximately equal to A_u. These stirrups reached yielding at failure of the specimen. The hanger reinforcement placed at the common volume of the beams once more was efficient up to failure of the specimen when the supported beam failed in shear tension.

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Seismic Response of the Reconstructed Piers and of the Collapsed Pilz Piers

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Abstract

During the Hanshin Great Earthquake so many types of failure and collapses occurred in different types of structures. For the simulation of behavior of some failed and collapsed RC structures during the Kobe Earthquake, the authors have already developed a computer program using Distinct Element for Fracture Analysis (DEFA). In the developed numerical simulation the structural medium is modeled as an assembly of circular (disk) particles which are connected to each other through two types of springs and dash-pots (Voigt-Kelvin visco-elastic model). The force is transferring due to the concrete and reinforcement between those elements that are within a specified distance from each other. The general idea for using pore springs (regarding the contact springs in original DEM) in continuous media was introduced by Iwashita as the winkler types springs for modeling the cohesion.
The main concept for modeling the reinforcement is to consider the reinforcement as an additional spring-dash-pot between the concrete elements. The average area of the reinforcement can directly be considered as the area of additional (in addition to the concrete pore) pore between the related elements with the same properties of reinforcement. These two pore media between two elements are calculated and checked for failure criteria separately. We named this concept as the double-pore springs-dash-pots.
One of the most complicated and very rare failure cases during the Kobe Earthquake was the collapse of a group of Pilz (German language) piers (pier No.126 to No. 142) in Kobe Route of Hanshin Expressway. Construction method of this group was different from the other piers in their neighborhood. The collapsed segment was of Pilz construction in which the deck, girder and column were built together as a continuous system. In the section of longitudinal bars, amount of the bars reduced for the upper part and new bars were welded to the bars of lower part. There was also a time-gap for concrete placement for the lower and upper parts in this section.
According to these features, we selected the pier No. 126, which was the tallest one and located at the end part of the group from Kobe to Osaka Cities. To have rough estimation of the effect of bar cut-off section and time gap of concrete placement, the cohesion and tension strength are decreased in that section. We assumed two more cases for the analysis, considering the weakness of material and amplification of ground motion as two possible reasons of the collapse. Therefore, we analyzed 3 cases. In case one the concrete properties and one of the most sever acceleration records during the Kobe earthquake are input. In second case it is assumed that the concrete were weaker than the designed one, but the acceleration is the same. In case three we assumed that the concrete is same

as case one but increased the acceleration two times to investigate about the possibility of collapse due to the amplification of ground motion.

For decreasing the analysis time and decreasing number of elements, the pier is modeled from the top of the footing. The analysis is done for unit (cm) width of the section for all cases. We have modeled the pier as an assemblage of disks with 20cm radius and considered the effect of bars and stirrups as additional springs and dash-pots between the related concrete elements. The necessary parameters for analysis are calculated according to data from the site.

In the Pilz case, there are two important points. One is that the deck, girder and the column were constructed with each other as a uniform media. Due to this point, the vibration of the piers will be transmitted through the deck and amplify the response of the structure. The second point in construction is cut-off of the main bars in column and welding them in one section in which the concrete placement also had a time gap. This section has been a weak point during the vibration of the pier. Therefore, referring to the results of 3 cases in analysis of collapsed pier, the main possibilities for collapse are material weakness, amplification of vibration or both of them, which mainly are due to the method of construction. The result of case 3 is in more agreement with that occurred in reality comparing to the case 2. However, some other piers in the collapsed group had a collapse mechanism as the case 2.

The new pier No. 126 after reconstruction has the same height and almost the same width while its length (in transverse direction of deck) is about 6m, which is almost two times of the length of the pervious one. The section of column is almost rectangular shape and the main bars are distributed along the sides of section and mainly in a circular core in the middle of section with a diameter of about 3m. We analyzed it under the same acceleration of pervious pier in 5 cases. In the cases 4 and 5 the material properties same as the site are used under 2 to 3 times of the acceleration, respectively. In the cases 6 to 8, the tension and cohesion of concrete decreased to half amount (considering the concrete possible deterioration) and increasing acceleration from one to three times. There was no cracking in case 4, while in case 5 shear cracks appeared in the base of pier. In case 6 there was no cracks, while in case 7 crashing happened in the middle part of the column. In case 8 concrete crashing and local failure happened in the corner of the connection between column and girder. However, in all cases the pier didn't suffered total failure or collapse.

The main conclusions are:

- The developed program has the capability for fracture analysis and failure monitoring of large RC structures during the earthquake. This method can be used for simulating the existing structures for monitoring their behavior and finding their collapse mechanism during the earthquakes in future.
- The main possibilities for collapse of the old pier were material weakness, amplification of vibration or both of them, which mainly are due to the method of construction. However, good agreement between failure mode in case 3 and the one in the site shows that the amplification of the acceleration was much more possible reason for collapse.
- The new pier unlike the pervious one has shear behavior and may resists to two times acceleration without any damage. However in far future and due to concrete possible deterioration and strong motion up to three times of the maximum one during Kobe earthquake may suffer from some local failure.