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Strengthening of a Bridge with Epoxy Beam and Prestressed Steel

Renforcement d'un pont avec de l'époxi et des fils d'acier précontraints

Instandstellung einer Stahlbrücke mit Epoxyharz und Stahldrahtvorspannung

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Victor L. Engelhardt, born 1936, received his civil and structural engineering degree at the Federal University of Paraná, Brazil. Expert in design, precast bridges and principally in bridge rehabilitation. In 1980 visited "Subway Engineering" JICA, Tokyo, Japan. Nowadays works as consultant in Engelhardt & Graf.

SUMMARY

The historical value of a metallic truss-frame "Warren-type" bridge us required a new process of bridge rehabilitation in 1975. Through the profiles that composed the lower chord, an epoxy beam was constructed with 40 steel wires, which was slightly prestressed with the objective of neutralizing the traction in the lower chord due to the structure's own weight. The epoxy mix that filled the space inside the deteriorated metallic profiles and the prestressed wires formed a new composite girder which had the characteristics required by the rehabilitation project.

RÉSUMÉ

La valeur historique du pont métallique en treillis, du genre "Warren", a encouragé le développement d'un nouveau procédé de réparation, en 1975. Sur la longueur des profilés métalliques, qui composent la longrine inférieure du pont, une poutre d'époxis à été construite. Elle se compose de 40 fils d'acier, lesquels ont été mis en légère précontrainte, avec pour objectif de supprimer la traction engendrée par le poids prope de la structure. Le mélange d'époxis a rempli l'espace vide situé entre les profilés métalliques détériorés et les fils d'acier précontraint ce qui forma une nouvelle poutrelle composite munie des caractéristiques nécessaires au bon achèvement du projet de récupération.

ZUSAMMENFASSUNG

Der historische Wert einer alten Stahlfachwerkbrücke des Typs "Warren" führte 1975 zu einer neuartigen Lösung für deren Instandstellung. Zusammen mit den vorhandenen Profilen des Zuggurtes wurde ein Verbundbalken hergestellt, mit einer Epoxy-Zwischenfüllung und einer Längsvorspannung aus 40 Stahldrähten, welche, leicht vorgespannt, die Zugbeanspruchung infolge Eigengewicht neutralisieren. Der neue Verbundbalken verhielt sich in den vergangenen 13 Betriebsjahren äusserst zufriedenstellend.

1. INTRODUCTION

There is a metalic bridge over Nhundiaquara river on the Paraná coast, Brazil, near the Serra do Mar that was built in 1912. This bridge is used for urban traffic in the historical city of Morretes and is the inicial point of Graciosa road a tourist attaction as it is the pioneer way from to the Curitiba highlands. (900 meters of altitude)

Its superstructure consists of a double metalic truss-frame "Warren" type with a 35.20 meters span and rests on a masonry infrastructure of excellent quality. Due to the proximity of the sea, lack of maintenance and total carelessness the bridge structure was so thoroughly eroded by the corrosion that its crumbling was foreseen even without traffic.

In 1964 an attempt to rehabilitate the bridge was carried out by a contractor using conventional methods but soon the results proved inefficient. Again the bridge had to be closed to the traffic from 1972 to 1975 when the Paraná's Highway State Department, D.E.R/Pr, decided to use its own work force to rehabilitate de bridge.

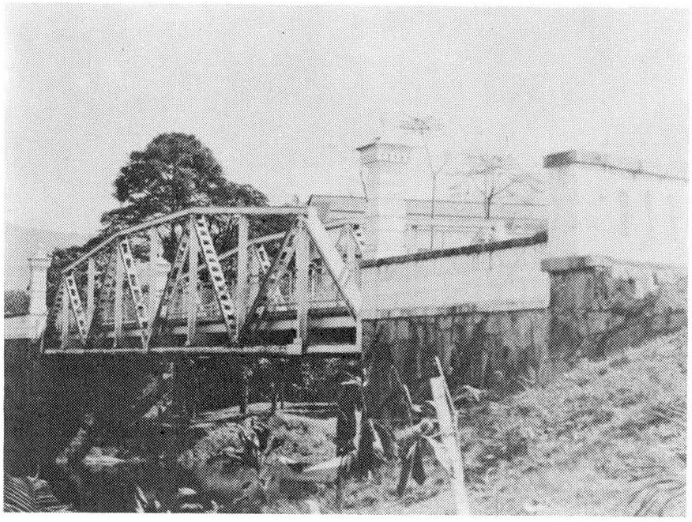


Fig. 1 General View

2. DESIGN AND EXECUTION

The rehabilitation design and execution was carried out in 1975 by engineer Victor L. Engelhardt, following these procedures:

2.1 Careful examination of all the structure showed that

2.2.1 The truss-frame lower chord was almost completely deteriorated due the corrosion.

2.1.2 The truss-frame lower joints were seriously damaged by the corrosion.

2.1.3 The metalic beams which supported the wood road surface were in an advanced stage of corrosion and could not be strenghtened (they were substituted by new parts).

2.1.4 The truss-frame components over deck plate were, in general, in acceptable conditions under structural point of view.

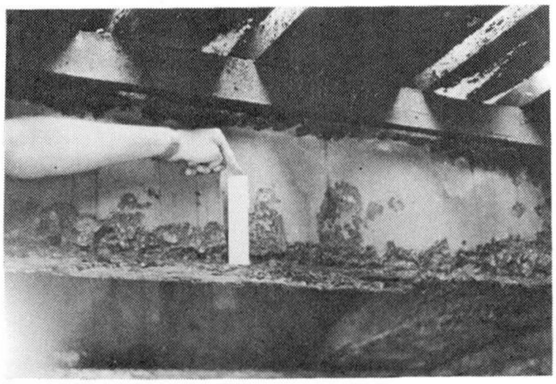


Fig. 2 Corrosion details

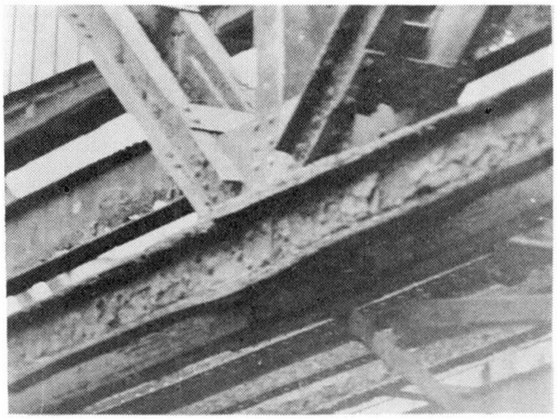


Fig. 3 Lower joints

2.2 Experiments at the DER/PR Laboratory

Several experiments were conducted with resines and epoxy structural adhesives using aggregate in several proportions as well as lightweight aggregate. Several series of samples were tested in different proportions resulting in:

- fcj = average tension of rupture under compression with 'j' days;
- ftj = average tension of rupture to traction with 'j' days;
- τ m = average tension of adherence between steel and epoxy mix;
- γ = specific weight.

2.2.1 Results of the first series of experiments

MISTURE (weight)	fc7 N/mm^2	fc21 N/mm^2	ft N/mm^2	γ g/cm^3
SIKA-DUR 32:SAND 1:0.5	57.4	-	51.7	2.10
SIKA-DUR 32:SAND 1:1	49.3	54.9	40.9	2.22
SIKA-DUR 32:SAND 1:2	28.9	-	-	2.26

Materials utilized:

- Sika-Dur 32
- Sand (from Curitiba)

2.2.2 Results of the second series of experiments

MISTURE SIKA-DUR 32:ISOPROPIL	fc7 N/mm^2	γ g/cm^3
1:3	24.9	1.073
1:4	15.6	1.037
1:5	13.2	0.993

Material utilized:

- Sika-Dur 32
- Isopropil spheres

2.2.3 Results of the third series of experiments

Materials utilized: Sika-Dur 32 - Isopropil Spheres - Steel CP-150 RN
Objective: Obtain the tension of adherence between steel amd mix.
Results: Average tension obtained in the experiments. σ ad = 11.52 N/mm^2

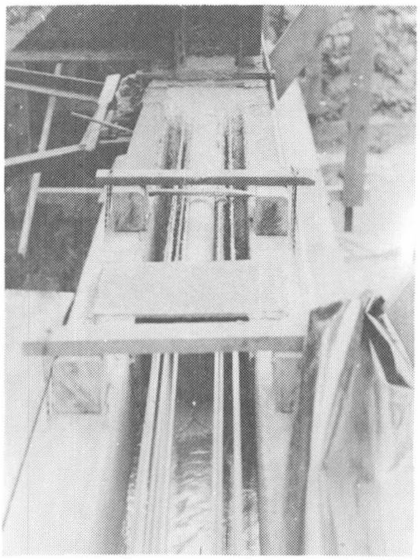
2.3 Solution adopted

- Cleaning with sand blast all the structure.
- Use new metalic reinforcements mainly in knots and superior chord.
- Through the profiles that composed the lower chord an epoxy beam was constructed with 40 steel wires ϕ 5 mm which was slightly prestressed in order to neutralize the traction in the lower chord due the structure's own weight. The epoxy mix that filled the space inside the prestressed wires and the original profiles in addition to its great resistance for compressing and traction effors due ro its high adherence power, worked in order to form an unique block composite by old structure and reinforcements introduced.
- The compression tensions obtained with the samples moulded with the mix utilized in the rehabilitation were greater than ones obtained in the laboratory experiments as shown by the values indicated below:

  Mixed design - 1:4 (Sika-Dur 32:Isopropil Spheres)
  Moulding in 21/11/75 and rupture 28/11/75

Average tension obtained	tc7m	= 19.65 N/mm^2
Standard deviation	δ	= 1.59
Maximum tension	fc7max.	= 21.24 N/mm^2
Minimum tension	fc7min.	= 18.06 N/mm^2
Specific weight	γ	= 0.965 g/cm^3

  The 1:4 proportion mix volume was the first one indicated in view ot its reduced specific weight, high adherence and facility of execution.



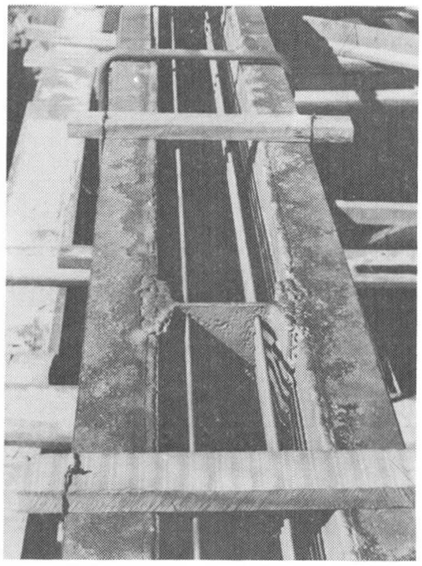
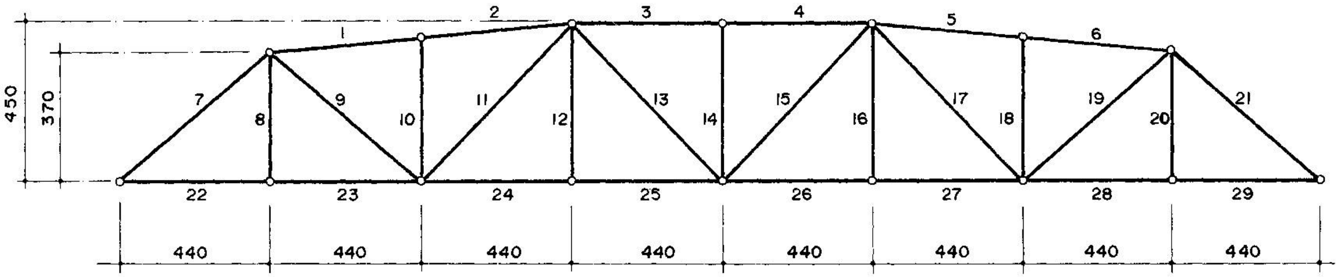
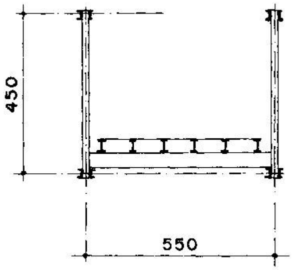
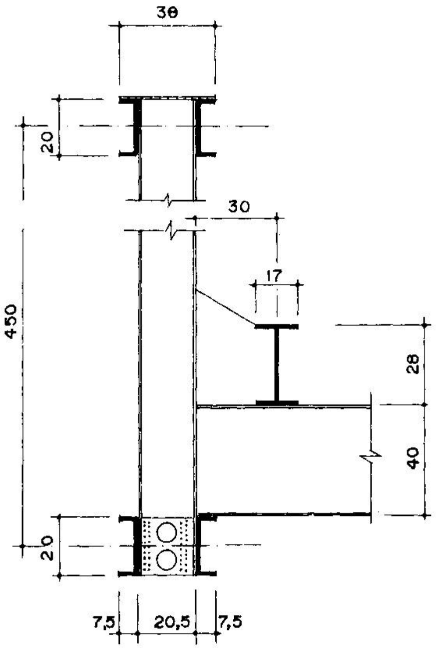


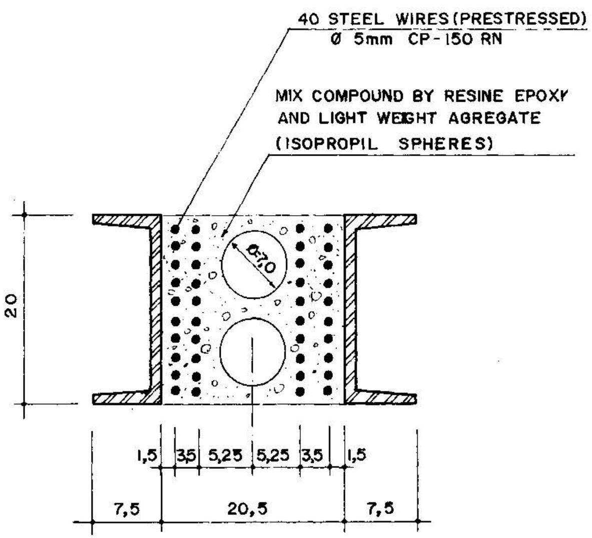
Fig. 4 and 5 Execution details

3. STRUCTURAL PLAN



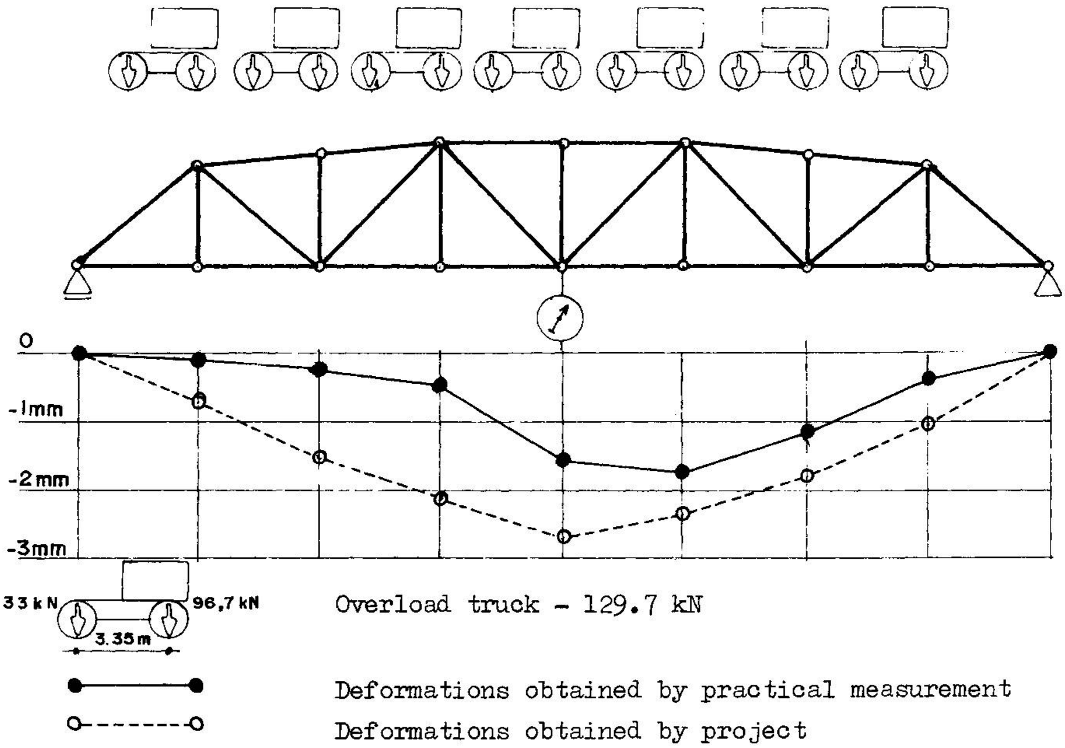






ELEMENTS	ORIGINAL		WITH REINFORCEMENT	
	S	I	S	I
	$M^2 * E\text{-}4$	$M^4 * E\text{-}5$	$M^2 * E\text{-}4$	$M^4 * E\text{-}5$
1 a 6	79,54	3820	119.70	7.852
7 = 21	80.89	3.820	115.10	6.111
8 = 10 = 12	62.55	4.320	62.55	4.320
14 = 16 = 18	"	"	"	"
20	"	"	"	"
9 = 19	75.34	2.797	93.35	2.797
11 = 17	63.87	1.890	79.01	1.890
13 = 15	48.48	1.234	55.64	1.234
22 a 29	183.30	22.770	183.30	22.770
30 a 37	32.20	1.850	32.20	1.850
38 a 45	—	—	7.86	
46 a 53	—	—	315.17	9.33

4. THE STRUCTURE BEHAVIOR



In order to check the structure's behavior another serie of tests were executed with an overload truck weighting 129.7 kN, in january 1989.
The tests were realized with the truck parking in 7 stations, at same time the deformations were indicated through a deflectometer (accuracy 1/100 mm) installed in the middle of the bridge.
With these results were possible to confront the deflections obtained by project and practically. We can observe in figure above that the deformations measured were always smaller than the theoretical deformations, consequently showing the good performance of the structure.

5. CONCLUSION

The structure has been constantly observed for 13 jears, subject to normal loads and its performance has proved excellent in regard to deformation and also to the knots rehabilitated and the adherence between epoxy resine and metalic parts.

Reduction of Plate Girder Depths for Bridges

Réduction de la hauteur de poutres métalliques de ponts

Verminderung der Trägerhöhe von Stahlbrücken

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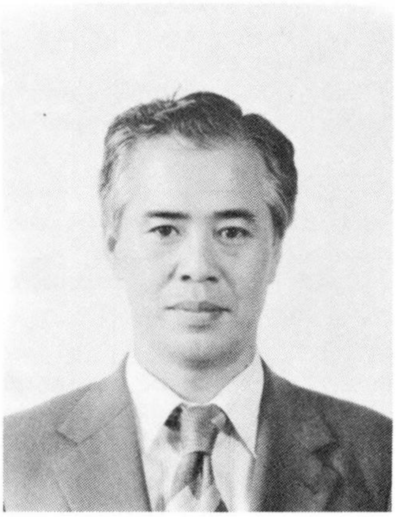


Hidehiko Abe, born 1931, graduated and obtained his doctoral degree, from the University of Tokyo, Japan. For 30 years he worked for the Japanese National Railways, mainly engaged in the design of steel bridges. Since 1984 he has been teaching structural engineering at University of Utsunomiya.

SUMMARY

The paper gives examples of remodelling work for reducing the depth of the plate girders of railway bridges on site, instead of replacement with new bridges, thus prolonging their lives. It describes the preliminary tests to select adequate procedures for construction, especially for field welding, and the method for calculation of the variation of stresses in various parts of the structure during the remodelling work.

RÉSUMÉ

Cet article contient des exemples de travaux de transformation de vieux ponts en acier, réduisant l'épaisseur des poutres en tôle, de manière à prolonger leur durée de vie, au lieu de les remplacer par de nouveaux ponts. On décrit les tests préliminaires qui permettent de sélectionner les procédures adéquates pour leur construction, notamment pour les opération de soudage sur chantier et pour la détermination des variations d'efforts dans les différentes parties de la structure durant les opérations de transformation.

ZUSAMMENFASSUNG

Dieser Beitrag beschreibt die Verlängerung der Lebensdauer alter Stahlbrücken durch Verringerung der Trägerhöhe und der Verstärkung. Er beschreibt auch die Vorversuche zur Bestimmung des geeigneten Vorgehens, speziell in bezug auf die Baustellenschweissungen und die Berechnung der Spannungen in den verschiedenen Bauelementen während der Instandstellungs- und Verstärkungsarbeiten.

1. INTRODUCTION

Many bridges are often replaced, not because of physical deterioration, but because of change of circumstances, such as improvement of the river, widening of the street underneath and necessity of reduction of the girder depth. The bridges, thus, have often unduly short lives.

The Japanese National Railways has many such experiences. There are, however, three cases where the depth of the girders of through type steel railway bridges, 14 spans in total, was reduced, instead of being replaced by new bridges. In two of them the bridges had originally been constructed in marshalling yard. But it was decided afterwards to lower the upper flanges of the outer girders of the bridges, because it was dangerous to the personnels working in the marshalling yards that the upper flanges were situated in a higher level than the rail level.

In the other case, the through girder bridge for Tokaido Shinkansen (the bullet train line between Tokyo and Osaka) adjacent to the platform of Kyoto Station was obstruction to extension of the platform, which was planned for increase of the capacity of the platform. These remodeling works were carefully planned, because in some cases all the work had to be done without interrupting or endangering the train operation.

The paper describes the sequence of work to be chosen for the maximum efficiency of the remodeling work, the calculated stresses in such an integrated girder, the procedure tests to select an adequate field welding method and so on.

2. OUTLINE OF REMODELING WORK

Takakura Viaduct was remodeled for extension of the present platform of Kyoto Station for Bullet Trains. Both Suikawa Bridge and Yamauchi Bridge were remodeled for safety and efficiency of personnels working in marshalling yards. The outline of the works is summerized in Table 1 and their cross-sectional configurations are shown in Fig. 1.

Table 1 Outline of Remodeling Works

Name of Bridge	Takakura	Suikawa	Yamauchi
Type of Structures	Through-type, Single track, Ballast floor, Welded	Through-type of 3-girders, 2-tracks, Open floor, Riveted	Through-type of 5-girders 4-tracks, Open floor, Riveted
Span length (m) and Number	35.0 x 1	16.2 x 1	22.6 x 10 13.2 x 2
Errection Method	Divided and lifted by a crane on the street underneath	New girder was errected on stagings	Divided and lifted by a crane on the bridge floor

3. STRESS CALCULATION

As the remodeling work proceeds, the stresses working in the cross-section of the existing girder and that of the added girder vary complicatedly. The example of stress calculation in Yamauchi Bridge (Span length is 22.6 m) at the center of span is described here.

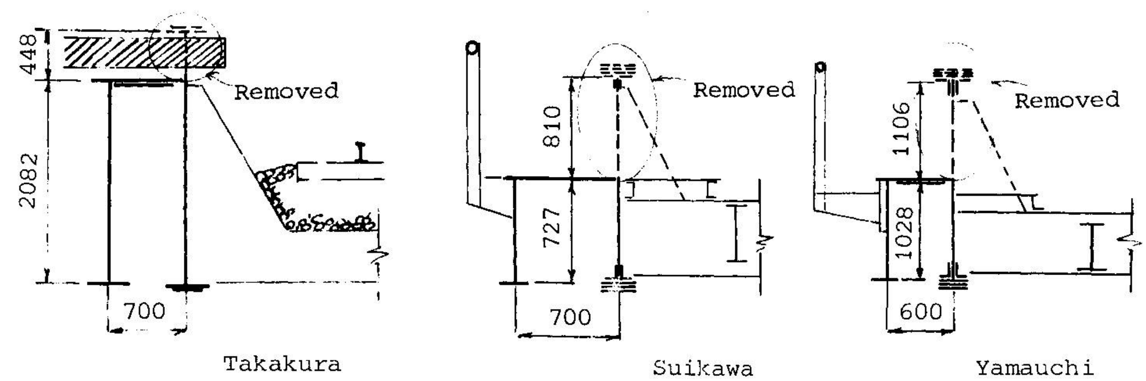


Fig. 1 Configurations of Cross-section of Bridges

The constructional steps chosen for the calculation are as follows;

1) First, the existing girder carries the dead load.
2) A new lower girder is connected to the existing girder, while the former is carrying its own weight only.
3) The portion of the existing girder above the upper flange of the added girder is removed.
4) The train load and the added dead loads such as the track and new side walk are carried by the integrated girder.

The calculated stresses change as shown in Fig. 2. The allowable stresses were reduced by 5 % and 10 % for compression and tension, respectively, for the existing girder, which was fabricated at a certain period between 1901 and 1950, as specified by the current related code.

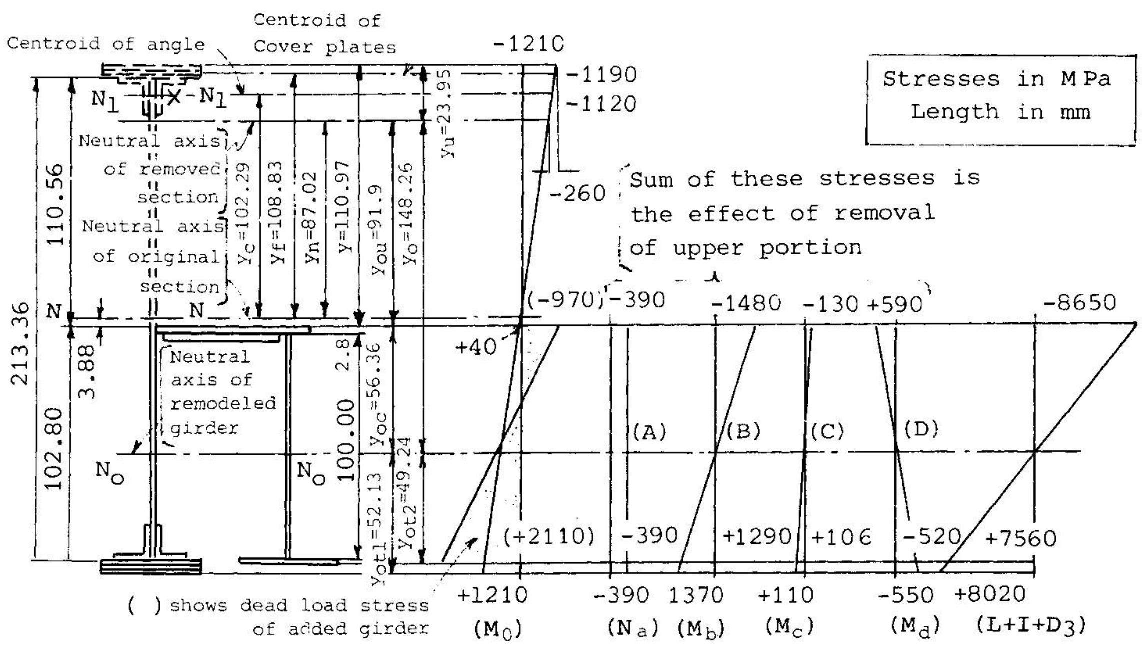


Fig. 2 Calculated Stress Variation at Span Center at Each Constructional Step

The portion of the dead load bending moment, which was carried by the upper portion to be removed from the existing girder, should be transferred to the integrated girder after removal. The stresses in the integrated girder can be calculated, breaking up the effect mentioned in Step 3) above, as follows;

a) The average compressive stresses which are induced in the integrated cross-section due to loss of the compressive stresses in the removed portion are to be evaluated.

b) The bending moment due to the resultant force of the stresses mentioned above with an arm length between the centroid of the removed cross-section and that of the integrated cross-section should be added.

c) The local bending moment which worked in the removed section only should be added.

d) The bending moment due to the weight of the removed portion should be deducted.

4. PRELIMINARY TEST FOR FIELD WELDING

Because this kind of construction was experienced for the first time, various preliminary procedure tests were conducted, in order to find proper conditions, such as edge preparation for welding, root gap, backing strip and welding sequence. In result solid flux backing of a special shape shown in Fig. 3, instead of backing strips of steel was chosen.

A possibility of considerable variation in the root gap for the welding between the flange edge of the added girder and the web of the existing girder was anticipated and a procedure to enable a sound welding with wide variation of root gap was finally developed.

As the result of many series of test, it was decided that the standard root gap should be 6 mm and a deviation of ± 2 mm should be tolerated without any modification and that if it happened to be smaller than 4 mm, the gap should be enlarged by an arc-air gouging before welding.

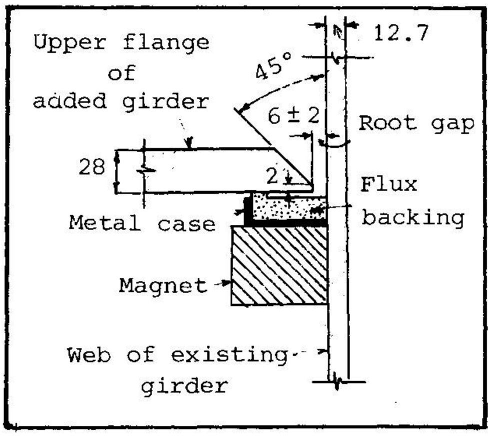


Fig. 3 Detail of Weld Preparation

Fig. 4 shows the cross-sectional shapes of test specimens with the first pass of weld deposited in groove of various root gap width. The root gap was varied from 2 mm to 8 mm. It was found that the weld with root gap of 6 mm was the best, with respect to penetration, back side of the first pass, uniformity of deposited bead. With root gap of 2 mm, a slight lack of penetration was recognized.

5. CONSTRUCTION

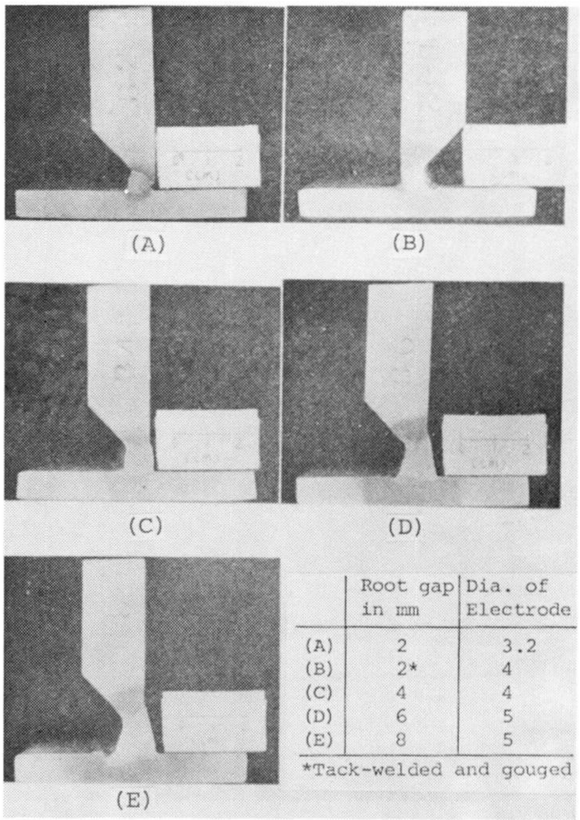
The existing girder was accurately measured and the girder to be added was so proportioned as to be well fitted to the existing girder which involved inaccuracy to some extent originally in fabrication. Prior to the field welding between the existing girder and the added girder, the diaphragms connected the both girders together first. In order to facilitate the adjustment of difference of deflections between them, the bolt holes on one side of the diaphragms were bored in slotted shape, while those on the other side were regular circular holes, as shown in Fig. 5. The deflection at each step during the construction was checked.

The surface of the web plate of 80 mm in breadth along the weld line was sufficiently cleaned, prior to welding. The part to be welded and its neighbourhood were preheated in 50c° to 100 C°. Care was taken to dry sufficiently the electrodes and the flux backings to be used in a controlled hearth.

In a rainy or windy weather and in humidity beyond 80%, the welding work was suspended. The sequence of welding was selected as shown in Fig. 6, in order to minmize the shrinkage deformation due to welding.

The back-side welding was unnecessary as a rule. But if any defect was detected by inspection from the back side, it was removed and rewelded.

The web of the existing girder was cut by a flame-cutting machine, which was automatically moved, guided on rails placed on the flange of the added girder, and the cut edge was ground smoothly.



	Root gap in mm	Dia. of Electrode
(A)	2	3.2
(B)	2*	4
(C)	4	4
(D)	6	5
(E)	8	5

*Tack-welded and gouged

Fig. 4 Preliminary Test to determine an adequate root gap length

6. CONCLUDING REMARKS

Japanese National Railways successfully lowered the depth of main plate girders of no less than fourteen spans of through-type bridges. The remodelled bridges have been in use without any trouble for more than ten years since then, and their lives have, thus, been prolonged.

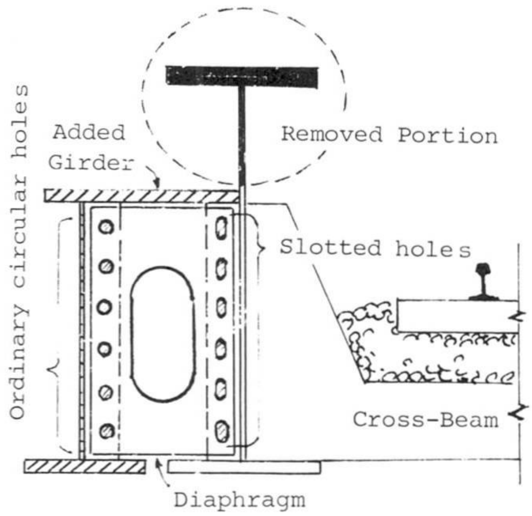


Fig. 5 Bolt Holes in Diaphragm

In these examples, the girders to be added carried their own weight only, when they were connected with the existing girders. But there may be cases where it is more efficient to use steel material of considerably high strength for the added girder and a proper portion of the dead weight of the existing girder is carried by the added girder, to make the best use of their respective allowable stresses. It will, however, require such a device as shown in Fig. 7, for adjustment of the ratio of carried load between the existing girder and that to be added.

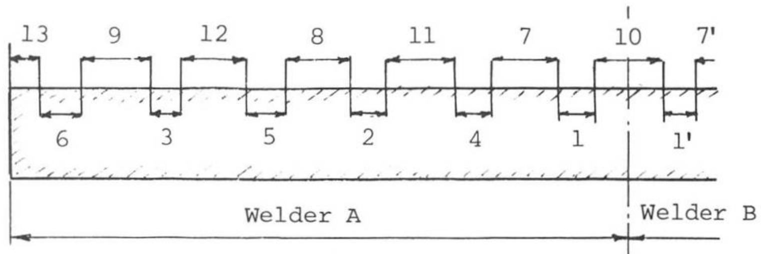
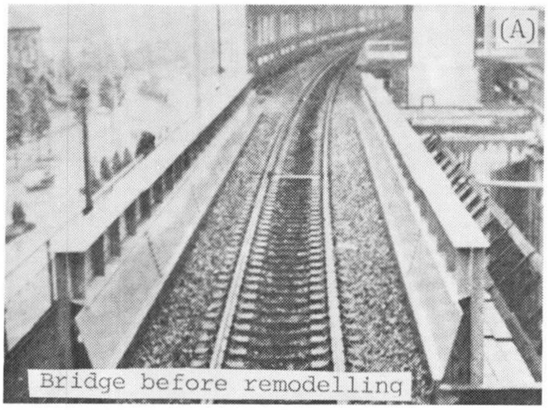


Fig. 6 Sequence of first pass in welding



Bridge before remodelling

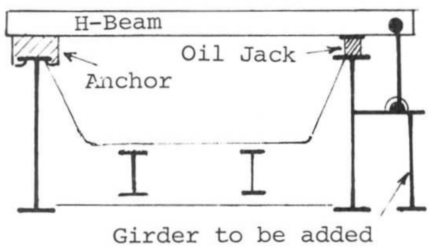
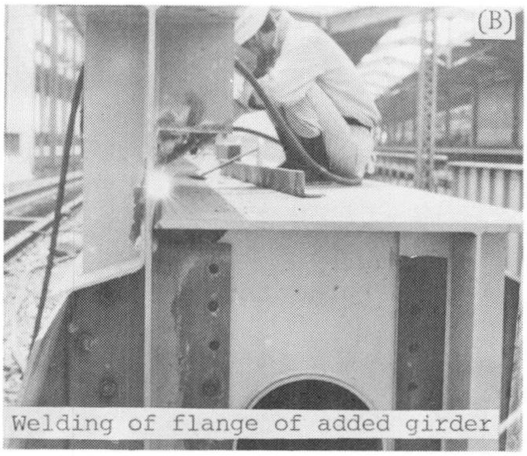
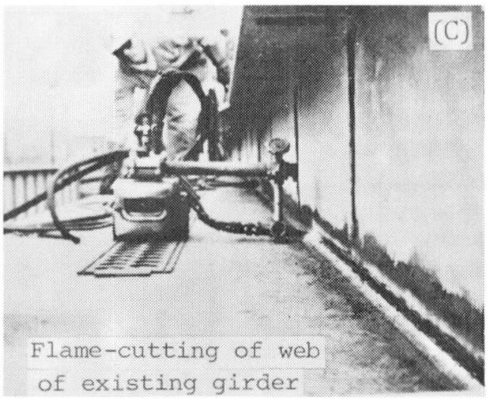


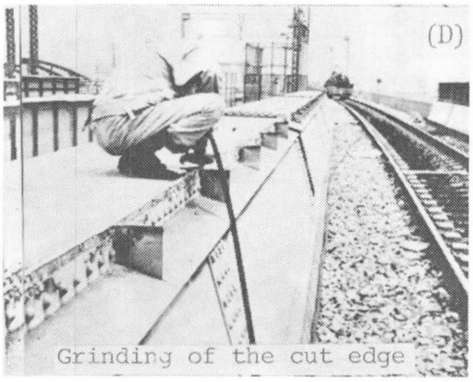
Fig. 7 Device for adjustment of carrying load



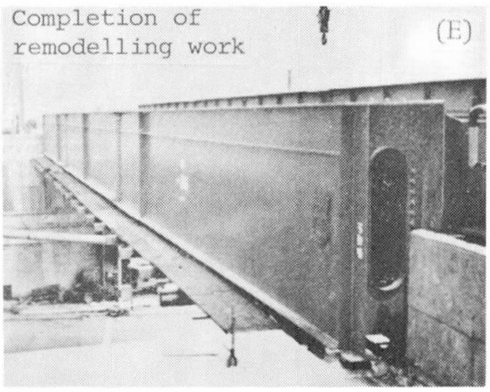
Welding of flange of added girder



Flame-cutting of web of existing girder



Grinding of the cut edge



Completion of remodelling work

Photos 1 to 5 Remodeling work of Takakura Bridge, Kyoto

Rehabilitation and Maintenance Design for Durability

Réparation et maintenance pour une meilleure durabilité

Instandstellung und Unterhaltung für eine bessere Dauerhaftigkeit

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Barbara ZOOK
Historical Architect
National Park Service
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Barbara Zook has a Bachelor of Architectures degree from the University of Houston in Houston, Texas, and has attended the architectural conservation course at ICCROM. She has performed field investigations of historic adobe structures as historical architect with the State of New Mexico. Barbara Zook now with the National Park Service, is conducting historic structures condition assessments and historic materials analysis.

SUMMARY

The durability of a historic structure is affected by the choice of rehabilitation and maintenance materials and techniques. Interventions which will improve durability are based on a thorough investigation, compatiblity with the existing materials, their weathering characteristics, their use in the building. As illustrated in this paper, both traditional methods and modern technology are used in the repair of adobe and sandstone structures.

RÉSUMÉ

La durabilité d'une structure est fonction des réparations et de la maintanance, au niveau des matériaux et des techniques utilisés. Les interventions, desquelles dépendent la durabilité, sont basées sur une investigation exhaustive, en adéquation avec les matériaux existants et les différentes actions des agents atmosphériques liés à la construction. Comme décrit dans cet article, les méthodes traditionnelles et la technologie moderne sont utilisées dans la réparation des structures en briques et en grès.

ZUSAMMENFASSUNG

Die Dauerhaftigkeit historischer Bauwerke wird durch die Wahl der Instandstellungsmaterialien und -techniken beeinflusst. Massnahmen zur Verbesserung der Dauerhaftigkeit sind aufgrund einer gründlichen Prüfung, der Verträglichkeit mit den vorhandenen Materialien, ihrem Witterungsverhalten und ihrer Verwendung im Bauwerk festzulegen. Wie dieser Beitrag zeigt, werden sowohl traditionelle als auch moderne Methoden zur Reparatur von Sandstein- und Lehmziegelbauten angewendet.

1. INTRODUCTION

The primary aim of conservation of a historic structure is defined in the Venice Charter [1] as the preservation of the authentic and historical values of the structure. Preservation of these values is best achieved by the minimal degree of intervention with the minimal loss of historic fabric. To improve the durability of a historic structure the intervention must be guided by sound conservation principles.

2. GUIDELINES FOR REHABILITATION AND MAINTENANCE

2.1 Building Investigation

A well documented inspection and evaluation of the building should be performed periodically and before major restoration. This investigation should include:

- Review of existing documentation, including original drawings and specifications, previous reports and assessments, maintenance records, and historic photographs.
- A visual assessment and documentation of existing conditions with photographs and drawings. Identification of materials and deterioration problems.
- Field testing. Collection of samples and laboratory analysis. Structural analysis.

2.2 Selection of Repair Materials

Two factors to consider in the selection of repair materials are the properties of the repair material itself and the function of the repaired element in the structure. The repair will be most durable when the repair material has mechanical, thermal and weathering characteristics compatible with the adjacent materials. Structural retrofits must be compatible with the existing load carrying system of the building.

2.3 Duplicating Historic Building Techniques

Existing structures have survived because of the durability of the original construction techniques. The durability of each specific feature of a building can be analyzed and the most successful techniques utilized in rehabilitation and maintenance work. In addition, today's knowledge and technology allows for improvements in design and materials.

Traditional methods and materials are appropriate, for example, in repointing and patching sandstone. However, modern technology can be integrated into the consolidation and structural strengthening of a sandstone building required to meet earthquake standards.

Use of a traditional method and material may be appropriate in a rural New Mexico community where the church and the triannual replastering of the church with mud plaster have historic value. However, improvements in the mud plaster mix by using the optimum particle size distribution, a nonexpansive clay and an improved mixing process will increase the durability of the plaster.

3. CASE STUDY: SANDSTONE REHABILITATION AND MAINTENANCE

Sandstone has been a popular building material in the United States, where it was widely used in the east for 19th Century "brownstone" row houses. On the west coast many high-rise buildings were clad with local sandstone, especially following the 1906 San Francisco earthquake and fire. Sandstone is easily worked but notorious for its tendency to decay, with signs of deterioration sometimes appearing within 20 years of the date of construction. [2] "Replacement in kind" of deteriorated sandstone is often not a viable option, as most sandstone quarries in the United States are now closed. The continued durability of these structures, therefore, rely on a variety of maintenance and rehabilitation techniques.

3.1 Investigation

Sandstone rehabilitation begins with a thorough investigation of the stone material in the context of the building as a whole in order to determine the cause of decay before any restoration is attempted.

3.2 Cleaning

The building is usually cleaned first in order to expose deterioration and prepare the surfaces for repair. Future durability of a building can be profoundly affected by cleaning techniques. Cleaning methods should be tested prior to use, starting with the gentlest and progressing to harsher methods until a satisfactory level of cleaning is achieved.

Sandblasting and other abrasive cleaning techniques can irreparably damage the stone surface and destroy delicate details. Because sandstone deterioration is often water-related, prolonged contact with water is not recommended. Chemical cleaning with a weak acid solution, or with a combination of alkaline prewash and acid afterwash is often effective.

3.3 Repointing

Use of a modern, high strength "durable" material for repointing mortar joints often results in lessened durability for the building as a whole. In general, repointing should be done with a "soft" mortar, lower in strength than the adjacent masonry so that stresses from expansion, moisture migration or settlement can be accommodated within the joints rather than the stones. Use of a hard portland cement mortar can result in accelerated deterioration of the stone, while the mortar joints remain intact, as shown in Fig. 1.

Periodic maintenance of mortar joints will greatly increase the durability of the building by reducing water infiltration into the stone.

3.4 Composite Patching

Mortar material is used to rebuild deteriorated sections of stone. The mortar is placed by hand or cast into forms to reproduce original details, as shown in Fig. 2. The durability of the patch improves when the patching material is similar to the stone in strength, porosity, and thermal expansion characteristics. The surface preparation, application method, reinforcement, and curing

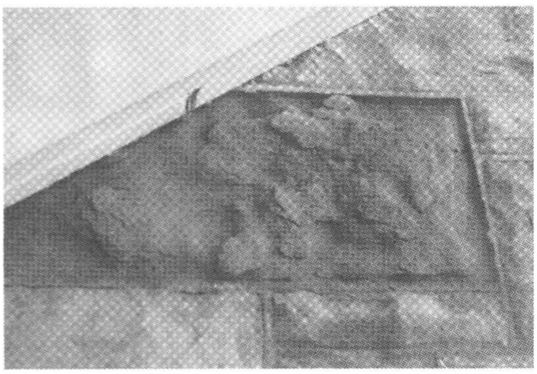

Fig. 1 Stone deterioration at repointed joint

also control the success of the patch. Failure of the repair shown in Fig. 3 occurred because the patch was placed across a mortar joint with no provision for movement of the stones on either side of the joint.

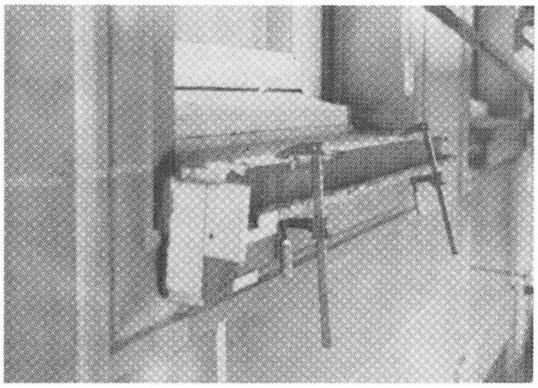

Fig. 2 Stone window sill rebuilt with mortar

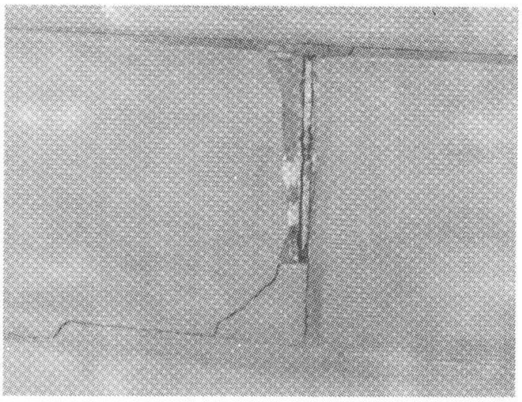

Fig. 3 Failure of patch installed across mortar joint

3.5 Coatings and Consolidants

Painting a sandstone building will, in many cases, decrease durability. Moisture may become trapped and the stone will decay behind the paint film. In isolated cases, coating with a "breathable" paint or water repellent may be appropriate, but such a treatment should be thoroughly tested before general application.

Consolidants are liquids brushed or sprayed onto the stone in order to reinstate cohesion between adjacent grains of stone when the natural cementing material has been lost by weathering. When successfully applied to fairly sound stone, consolidants can increase the durability. However, if proper depth of penetration is not achieved, a hardened surface skin may form which will later spall. Careful testing should be performed before applying a consolidant since the process is irreversable.

3.6 Protective Systems and Maintenance

Durability of a sandstone building depends to a great extent upon the ability to design and maintain details which will keep water away from the building. [3] Rehabilitation and maintenance should include repairs to defective roofs and gutters, installation of flashings, caulking or repointing joints, and solving rising damp and humidity problems. A regular maintenance program will prolong the durability of the stone and help avoid costly large scale restoration.

4. CASE STUDY: ADOBE (MUDBRICK) REHABILITATION AND MAINTENANCE

Adobe, or mudbrick, composed of sand, silt and clay binder, is one of man's oldest building materials, and is still used by an estimated thirty percent of the worlds population [4]. Mudbrick is primarily used in hot dry arid climates because of the deleterious effects of moisture. Water entering the mudbrick physically reacts with the clay reducing the cohesive properties and compressive strength. Consequently, an objective of rehabilitation and maintenance programs for mudbrick is to keep moisture away from the structure.

4.1 Investigation

Early detection and prevention of moisture related deterioration by frequent inspection and early repair is the best way to increase the durability of the mudbrick structure. Typical areas of moisture penetration are the bases of walls, cracked window sills, cracked surface renderings, roof and building joints, and penetrations through the roofs and walls. Typical sources of moisture are improper site drainage, snow or water collection at the wall base, broken sewer and water pipes, high or fluctuating groundwater tables, and deposition of roof runoff near the building base. Water enters the base of walls by capillary action, particularly where the adjacent grade is higher than the foundation.

Often, a detailed investigation, including destructive and nondestructive testing is required when visual inspection is prevented by the presence of a hard rendering. Nondestructive testing using a neutron moisture probe at a two foot grid will provide a map of the moisture content. Destructive tests by sampling with a split tube sampler or removing one square foot sections of the exterior and interior renderings may be necessary when high moisture content is suspected. Changes in the structure and the effect of previous repairs should be evaluated by research of historical data, weather data, and previous repair records.

4.2 Surface Coatings

The primary maintenance procedure for mudbrick structures is periodic applications of a coating as a protection against water and wind erosion. Traditionally, mudplaster has been applied every three years and has served as an easily renewable sacrificial layer. Mudplaster is a porous coating with properties similar to mudbrick units and it allows moisture trapped within the wall to evaporate. Deterioration of the mudbrick is easily detected since the mudplaster will crack in the same location as the substrate and if the wall becomes wet, dark spots will appear on the mudplaster.

A practice in New Mexico is to apply hard nonporous renderings such as cement plaster, soil cement plaster and silicone sprays to mudbrick structures. These renderings trap moisture inside the wall resulting in slumping failure of the mudbrick. Because cement plaster is a brittle coating, movement of a mudbrick wall supported on a drylaid rubble stone foundation causes cracking in the plaster and opens new avenues for moisture penetration.

4.3 Patching

Basal erosion caused by the presence of moisture and freeze thaw action, requires immediate repair because it reduces the load bearing capacity of the wall. The west wall of the San Jose Church in Upper Rociada, New Mexico, shown in Fig. 4, slumped recently after the removal of interior wood furring and removal of adjacent exterior grade. The base of the wall had a moisture content of 17 to 24 percent and the imposition of additional loads on the moist mudbrick base caused the wall failure. Basal erosion is repaired by removing loose material and adding new mudbrick, using

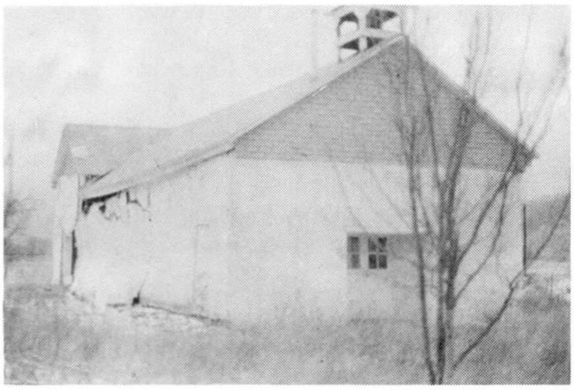

Fig. 4 Wall failure resulting from high moisture content

matching mud mortar. Specifying a good particle size distribution, a nonexpansive clay, and using correct manufacturing procedures for the replacement mudbrick will increase the durability of the repair. [5]

Erosion and cracks at the top of walls are repaired by removing loose material, keeping as much historic fabric as possible, wetting the wall and relaying new mudbricks in mud mortar. Holes and cracks in the wall provide access for water and should be patched using mud mortar applied to a moistened surface in 1/2-inch layers.

4.4 Structural Repairs

Early mudbrick structures in New Mexico were often constructed with a minimal understanding of structural design. Consequently, structural details such as unkeyed corners and large height to width ratios result in wall instability and dangerous movement.

The Pueblo of Acoma in central New Mexico consists of three-story contiguous southfacing mudbrick structures on a high sandstone mesa built by Indians in the eighteenth century. Moisture deterioration has not been a problem in these structures but construction with unkeyed corners and narrow widths of three-story high mudbrick walls caused collapse of a section of an Acoma structure, as shown in Fig. 5. The collapsed wall was rebuilt with additional thickness of adobe and with keyed in corners.

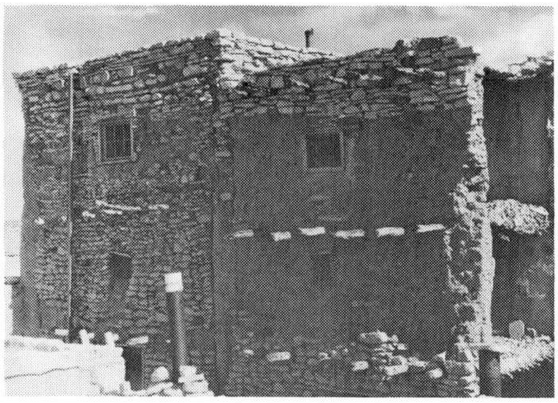

Fig. 5 Mudbrick wall collapse

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Advanced Repair Techniques for Reinforced Concrete Structures

Techniques de réparation évoluées pour les structures en béton armé

Fortgeschrittene Technologien zur Reparatur von Stahlbetonbauten

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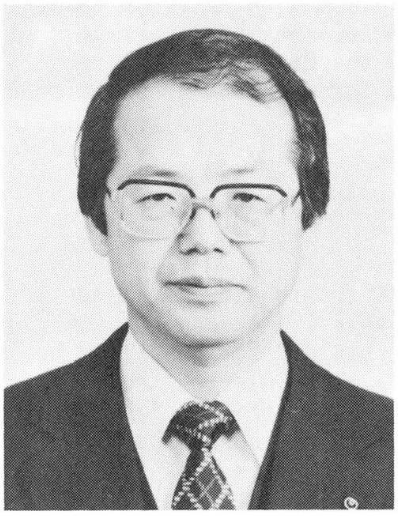

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SUMMARY

NTT has approximately 30,000 buildings with a total floor area of about 21,000,000 square meters. The majority of these buildings are reinforced concrete, many of which are in various stages of deterioration. This paper introduces three techniques: the crack filling method for exterior walls, the repairing method for basement walls, and the rebar corrosion-restriction method. With these methods good results have been obtained on many buildings.

RÉSUMÉ

NTT possède environ 30 000 bâtiments, occupant une surface au sol totale d'approximativement 21 000 000 mètres carrés. La majorité de ces bâtiments sont en béton armé, et beaucoup d'entre eux sont dans divers états de détérioration. Cet article présente trois techniques mises au point: la méthode de remplissage des fissures pour murs extérieurs, la méthode de réparation des défectuosités pour murs de soubassement, et la méthode de protection anti-corrosion pour barres d'armature. Ces méthodes ont permis d'obtenir de bons résultats dans un grand nombre de bâtiments.

ZUSAMMENFASSUNG

Die NTT ist im Besitz von ungefähr 30'000 Gebäuden mit einer Gesamtfläche von etwa 21'000'000 Quadratmetern. Die Mehrzahl dieser Gebäude besteht aus Stahlbeton, von denen viele mehr oder weniger baufällig sind. Dieser Bericht beschreibt drei Methoden: die Mauerriss-Füllmethode für Aussenwände, die Ausbesserungsmethode für Grundmauern und die Betonstahl-Korrosionsschutzmethode. Mit diesen Methoden wurden bei vielen Gebäuden gute Resultate erzielt.

1. INTRODUCTION

Presently, NTT has approximately 30,000 buildings with a total floor area of about 21,000,000 square meters. The majority of these buildings are reinforced concrete (RC) ones. Their average age is about 23 years, and many of them are in various stages of deterioration. The main cause of deterioration for RC buildings is rebar corrosion. Next is rain and underground water leakage, which cause cracking, peeling of concrete. NTT has been carrying out studies to find an effective method for diagnosing the durability and repair techniques for RC buildings [1]. This paper presents three techniques developed by NTT's Building Engineering Department for repairing deteriorating buildings.

2. CRACK-FILLING METHOD FOR EXTERIOR WALLS

2.1 Downward flow

Until recently, a low viscosity resin had been used to fill minute concrete cracks in exterior walls due to rain leakage. However, NTT discovered in the buildings repaired that the resin did not coagulate but instead flowed downwards. To confirm this experimentally, NTT injected the low viscosity resin into a crack in an existing RC wall. Test samples of the crack were taken from the RC wall after the resin coagulated. The relationship between the crack width and the filling ratio for the samples taken from the highest point along the crack is shown in Fig.1. As indicated in Fig.1, the injected low viscosity resin flowed downward in cracks more than 0.1 mm in width.

2.2 High viscosity resin

The resin was improved to satisfy the following conditions. Resin injected into a 1.5 mm width crack does not flow downwards at 5°-35°C. The resin can be used to fill cracks as narrow as 0.05 mm in width, at 5°-35°C. The improved resin, a high viscosity, is compared with the low viscosity one in Table 1.

Filling tests using both resins were performed on crack models. The models used in the test and the results are shown in Fig.2. Each model was made of two clear acrylic panels that were placed together to form a specified crack width. Resin was injected into three holes spaced at 25 cm apart along the seam of each model on the right side as shown in Fig.2. In the case of the 0.1 mm in width crack, both types of resin did not flow downwards. For the cracks 0.3 mm and 1.0 mm in width, the low viscosity resin flowed downwards, but the high viscosity one did not.

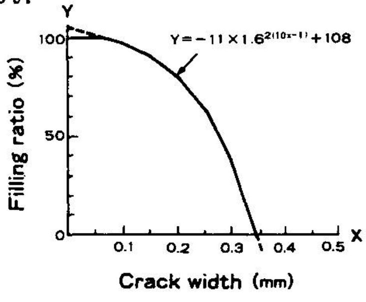


Fig.1 Crack width and filling ratio

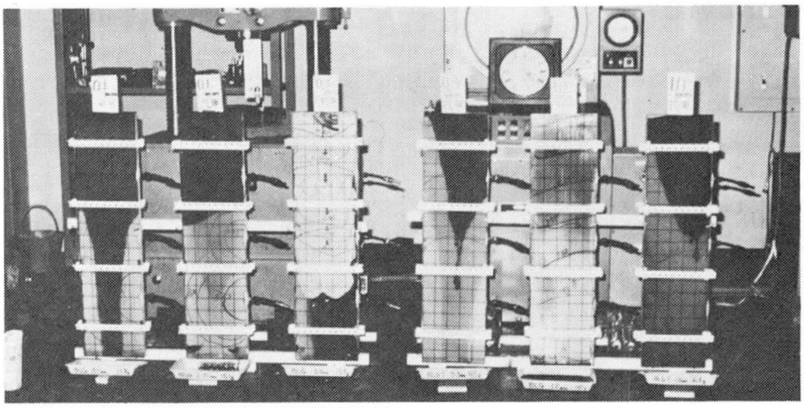
Table 1 Quality of usual and improved resin

Comparison Items	Usual resin	Improved resin
Specific gravity	1.15	1.12
Viscosity	510 cp	8,000 cp
Thixotropy index	—	4.8
Compressive yield strength	642kg/cm²	650kg/cm²

NOTE) Thixotropy index (T.I.value): Expressed as the ratio of the 2rpm viscosity to the 20rpm viscosity, both of which are measured by means of the B-type rotary viscometer using the same rotor.

2.3 Injection method

The injector adopted is a cartridge type and the part containing the resin is a synthetic rubber ballon, as shown in Fig.3 and Fig.4. The resin is slowly injected using the balloon's contractile pressure (2.5 kg/cm² for 25mm in diameter), and the amount of resin packed can be controlled easily. After the injectors are inserted into the drilled holes along the crack, the crack is filled with resin. This injection method greatly simplifies the work and improves work safety [2]. A picture of the injection work is shown in Fig.4.



Crack width 0.1 mm 0.3 mm 1.0 mm
Viscosity resin low high low high low high

Fig.2 Test results for crack model

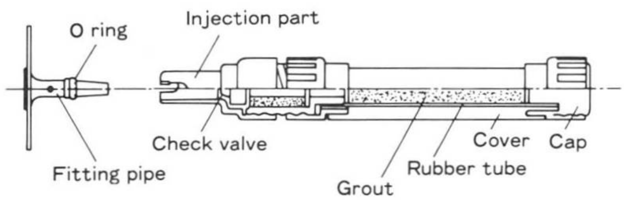


Fig.3 Injector (cartridge type)

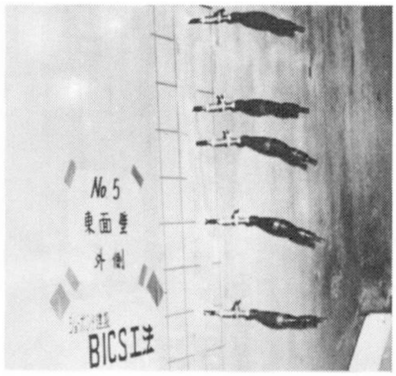


Fig.4 View of injection

3. METHOD FOR ELIMINATING LEAKS IN BASEMENT WALLS

3.1 Characteristics

Since the groundwater level in Japan is shallow, basements sometimes experience water leakage. The leading source of leaks in basements is from cracking, honeycombing or construction joints.

Presently, all repair work is carried out from inside the basement. However, in a newly developed method by NTT all repair work is done from outside the basement. It is particularly effective in stopping leaks in walls with numerous cracks or from an unknown source [3].

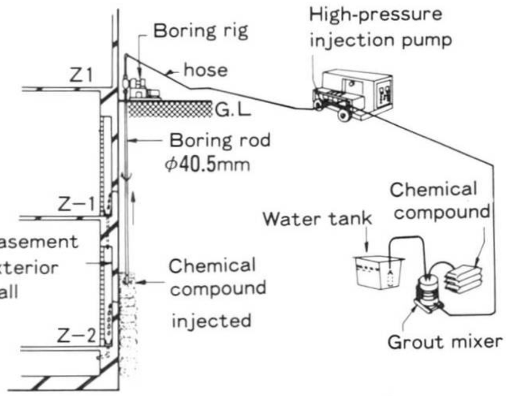


Fig.5 Filling system

The new method uses a chemical compound to stop underground water leakage. The chemical compound is pumped into the ground outside the basement walls at high pressure. The activated silica in the chemical compound reacts to the calcium hydroxide produced by the reaction between water and cement. A silicic acid calcium produced by the reaction enters with water into the concrete defects, becomes implanted, grows and finally fills up the defect. This method is shown in Fig.5.

3.2 Experimental testing

Six models in the shape of boxes and constructed of concrete panels with defects such as cracking and honeycombing were used in the testing. The inside of a box was regarded as the outside of a basement wall and the outside was considered the inside of a basement wall. Model configuration and the test method are shown in Fig.6. Chemical compounds with soil samples were mixed and placed into the models. Water was then pumped into them at 3 ton/m^2 pressure from a water tank placed three meters above the models. Leakage from the models was measured continuously. Chemical compound mixings are shown in Table 2. The effects of the various compound mixings on water leakage are compared and shown in Fig.7. In the case of a fat mix agent (cement 300 kg/m^3, chemical compound 18 kg/m^3), water leakage was reduced by 96.1 - 99.7% by the 30th day of testing. In the case of a lean mix agent (cement 150 kg/m^3, chemical compound 15 kg/m^3), leakage only decreased 30.2% by the 30th day of testing.

3.3 Testing in an existing building

The test was performed on the external perimeter walls of the third basement (see Fig.8). The mixing of the chemical compound used was exactly the same as that of No.6 in Table 2. Boring rods were driven in 25 cm from the outer surface of the exterior walls to the required depth. The chemical compound was injected into the rods, 23 in total. During the injection, the rods were pulled up at 10 cm/step.

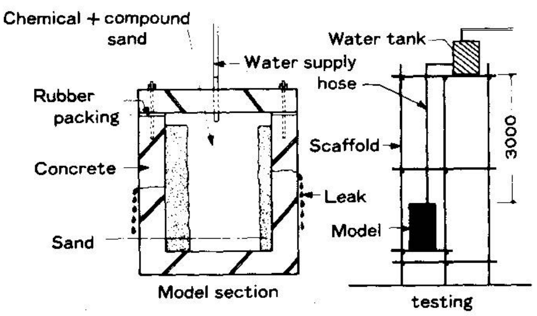


Fig.6 Test model

Table 2 Chemical compound mixing and soil (Unit of dimension : kg/m^3)

No.	Cement	Admix-ture	Water	Soil
1	600	36	724	sand
2	300	30	857	sand
3	150	15	928	sand
4	600	36	724	Silt sand
5	300	18	862	Silt sand
6	450	27	792	Gravel & sand

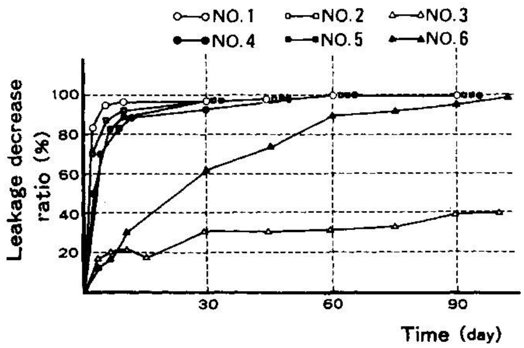


Fig.7 Effect of various chemical compound

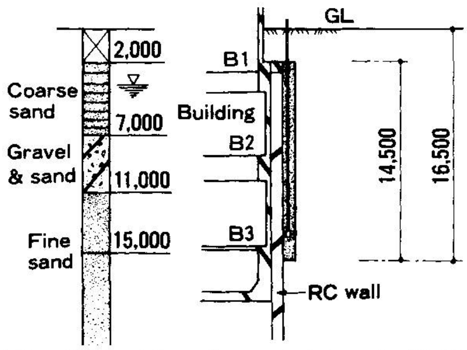


Fig.8 Testing in existing building

The number of leaks was 88, and all of them were observed carefuly by eye. The amount of leakage for six of these leaks was measured. Eye observations are shown in Fig.9. The number of leaks decreased 52% by the 30th day of testing and 86% by the 90th day. The measurements for six leaks are shown in Fig.10. The number of leaks by the 110th day of the testing was one. As each day passed, the effect of this method was noticeable.

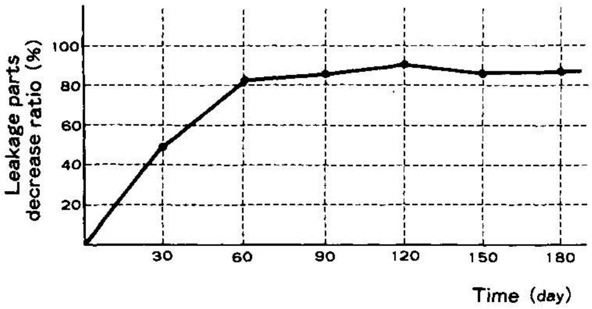


Fig.9 Eye observation

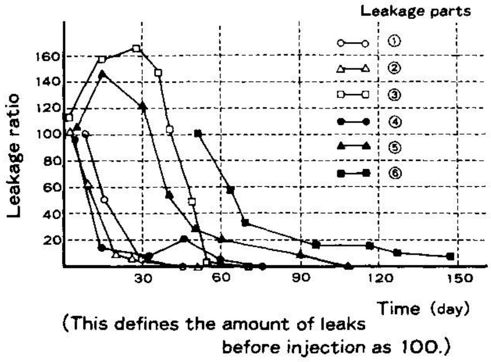


Fig.10 Amount of leakage

4. REBAR CORROSION-RESTRICTION METHOD

4.1 Concept

The method of restricting concrete carbonation has generally been adopted in Japan to improve the durability of concrete structures. However, concrete carbonation and the coverage concrete thickness for rebar vary widely. This means that the concrete around some rebar in existing buildings has probably carbonated, even if a building is not old. Therefore, NTT developed the method of restricting rebar corrosion in the carbonated concrete [4], as shown in Fig.11.

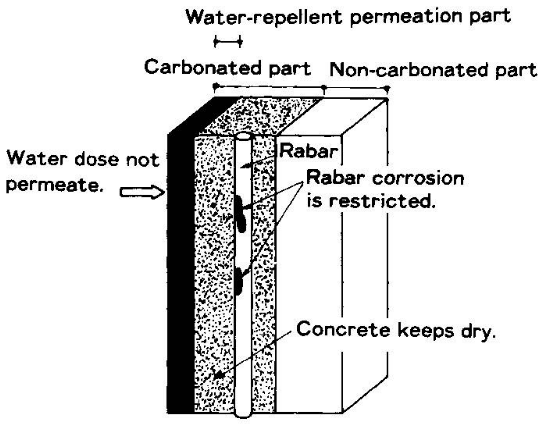


Fig.11 Concept of the method

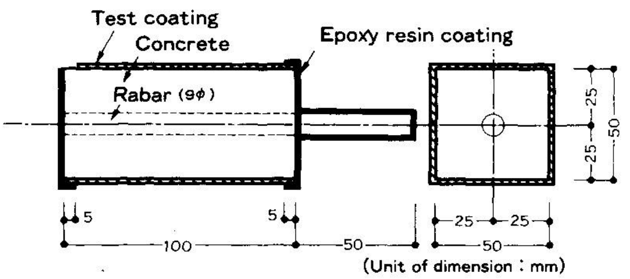


Fig.12 Test sample

Table 3 Code No. for various coatings samples

No.	Various coatings	
1	——	Exposed concrete
2	Water-repellent	Acrile polymer
3		Silane monomer
4		Methyl-silicon polymer
5	Water-proof	Acrile emulsion
6		High elastic acrile rubber
7		High elastic polymer-cement
25	Water-repellent + Water-proof	No.2 + No.5
26		No.2 + No.6
27		No.2 + No.7
35		No.3 + No.5
36		No.3 + No.6
37		No.3 + No.7
45		No.4 + No.5
46		No.4 + No.6
47		No.4 + No.7

4.2 Effect of restriction

An accelerated aging test was performed to confirm the effect of restriction. A reinforced concrete sample used in the testing is shown in Fig.12. After the concrete samples were forcibly carbonated to the rebar position, the surface of the samples were painted with various coatings. Next, the samples were alternately placed in wet or dry environment every seven days. This procedure accelerated rebar corrosion. The test extended over 110 days. The code No. for the various coatings samples used are listed in Table 3.

The ratio of the corrosion area to the entire surface area of the rebar and the average weight of permeated water are shown in Fig.13. The weight of permeated water is the weight of a sample in the wet environment minus its weight in the dry environment. Samples of the rebar corrosion are shown in Fig.14. As indicated in Fig.13 and Fig.14, impermeability to water was satisfied in the case of the samples coated with the water-repellent silane coatings (No.3), waterproof coatings (No.5,6,7), and the water-repellent and waterproof complex coatings (No.25-47).

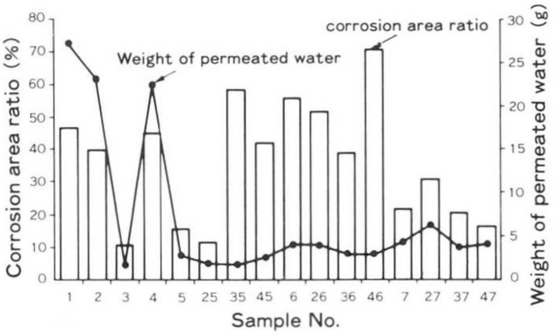


Fig.13 Test results (corrosion area ratio)

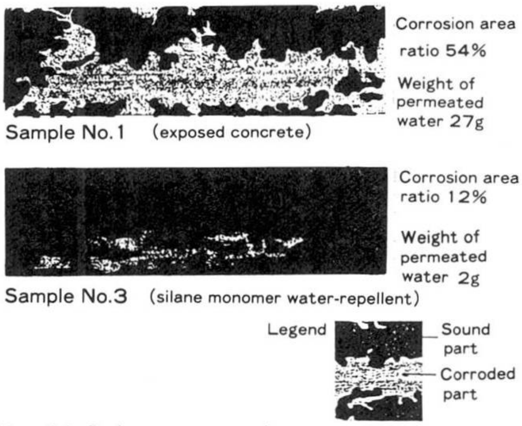


Fig.14 Rebar corrosion

<u>4.3 Requirement standards for restrictive coatings</u>

As the restrictive performance of the coatings varied in the test, NTT set standards to select the most suitable coatings. One of the some requirements is that permeability after accelerated age testing shall not be more than 1.5 times compared with that before testing. Also, in the case of the water-repellent coatings, the permeated water-repellent depth in concrete shall be 5 mm and over in model testing. In the case of waterproof coatings, it must be confirmed by model testing that corrosion is not caused by an ingredient in them.

5. CONCLUSIONS

Three repair methods for concrete structures were introduced. NTT has employed these methods in repairing many of its buildings, and has obtained good results.

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In-situ Assessment of Reinforcement Corrosion in Concrete Structures

Evaluation in situ de corrosion des armatures dans les structures en béton

Bestimmung der Bewehrungskorrosion an Betonbauwerken

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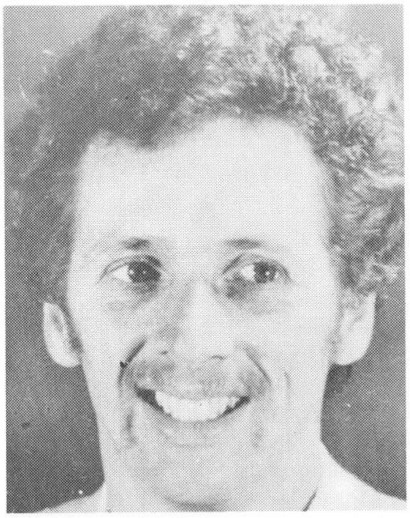

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SUMMARY

This paper reports on the preliminary results of three corrosion durability studies on reinforced concrete structures recently constructed in high risk, chloride rich environments. Various permeability and electro-chemical methods have been used to monitor the corrosion performence of each structure. The results to date suggest that the use of pozzolanic cement replacements and of hydrophobic pore blocking admixtures, together with high quality concrete, will allow the risk of corrosion to be reduced to an acceptable level.

RÉSUMÉ

Cet article rapporte les résultats préliminaires de trois études sur la durabilité face à la corrosion des structures en béton armé récemment construites dans des milieux à grand risque et riches en chlorure. Diverses méthodes de perméabilité et d'électrochimie ont été utilisées afin de saisir l'évolution de la corrosion de chaque structure. Les résultats à ce jour suggèrent que l'utilisation des ciments pouzzolaniques, d'adjuvants à pores bloqués hydrophobes, conjugués à un béton de haute qualité, permettront de ramener le risque de corrosion à un niveau acceptable.

ZUSAMMENFASSUNG

Dieser Beitrag beschreibt die Ergebnisse dreier Korrosionsuntersuchungen an Stahlbetonbauwerken in chloridreicher Umgebung. Verschiedene Durchlässigkeits- und elektrochemische Methoden wurden angewendet, um das Korrosionsverhalten jedes Bauwerks zu überwachen. Die bisherigen Resultate deuten an, dass durch die Verwendung von pozzolanhaltigem Zement und von hydrophoben Dichtungszusätzen, zusammen mit qualitativ hochwertigem Beton, das Risiko der Korrosion auf ein annehmbares Mass vermindert werden kann.

1. INTRODUCTION

The use of reinforcing steel or prestressing steel in concrete structures liable to chloride attack can lead to high risk of failure due to corrosion of the steel. The risk of chloride induced corrosion is particularly large when the chlorides have ingressed through the hardened concrete rather than being present as contaminants or additives in the original concrete mix. The majority of chlorides ingressing through the concrete cover are available to promote corrosion, rather than being chemically bound into the cement matrix [1]. Although there may be a considerable durability risk from using steel reinforcement in a chloride rich environment, its use leads to lightweight, efficient structures. Furthermore, if accidental load damage occurs at any time, the inclusion of reinforcing steel means that the structure may be more readily repaired than if unreinforced.

Corrosion of reinforcing steel is a two phase process [2]. During the first phase chlorides diffuse or permeate through the concrete cover until the passive oxide layer on the surface of the steel is disrupted and corrosion commences. During the second phase anodic and cathodic sites are established and active corrosion leads to loss of section of the reinforcement and spalling of the cover. The corrosion rate is controlled by oxygen diffusing through the concrete cover and by the flow of hydroxyl ions from the cathode to the anode. Methods of enhancing the durability performance of concrete must either restrict the flow of chlorides into the concrete during the first phase or limit some part of the corrosion cell once it has become established.

This paper discusses the effects of using a pozzolanic cement replacement and also of using a durability enhancing admixture on the durability performance of a number of structures located in chloride rich environments.

2. ASSESSMENT OF CORROSION IN REINFORCED CONCRETE

A number of techniques were used in these studies to assess corrosion performance. Some of these were aimed at determining if corrosion had been or was likely to be initiated. Other methods assessed actual or probable corrosion rates, once corrosion had commenced.

The permeability of concrete will be related to the ease of ingress of chlorides, moisture and oxygen through the cover. Two simple tests were used to evaluate the permeability of the cover concrete. The Initial Surface Absorbtion Test [3] (I.S.A.T.) was adopted to measure the rate at which water is absorbed into the concrete. The Figg gas permeability test [4] was also used to assess the ease with which gas can flow inside the concrete. Both tests are highly sensitive to the existing moisture content of the concrete and hence are best conducted on moisture conditioned cubes or other laboratory specimens.

Chemical analysis of concrete samples taken at incremental depths can also be used to assess directly the ingress of chlorides, causing depassivation of the reinforcement.

The initiation of a corrosion pit in the steel reinforcement is characterised by the establishement of an anodic corroding region, surrounded by a cathodic region. The electrical potentials can be mapped using the half-cell technique [5] and, with skill, potential maps can be used to locate where corrosion has been initiated.

Assessing the rate of active corrosion in reinforced concrete structures is rather more difficult. Concrete resistivity measurements have been related [6] to probable corrosion rates if used in conjunction with potential mapping. Alternatively the rate of corrosion on steel samples cast into the concrete during construction can be measured using the a.c. impedance method [7].

However this is a difficult and expensive technique and is only suitable for laboratory or special site studies.

3. MONITORING OF HIGH RISK STRUCTURES

The techniques described have been used in conjunction with regular visual and photographic surveys to monitor the corrosion durability performance of three different types of structure, constructed over the past five years in aggressive chloride rich environments.

The first site comprises some 2500 reinforced concrete coastal armour units, located on man-made offshore reefs and on shoreline wave absorbtion slopes (Fig. 1). Approximately half of these units were cast using a 30% pulverised fuel ash cement replacement. This was done to reduce cracking due to the heat of hydration and also for economic reasons, but it provides an excellent case study on the durability effects of using pozzolanic cement replacement. A concrete cube strength of 55 N/mm^2 and cover of 60 mm were specified.

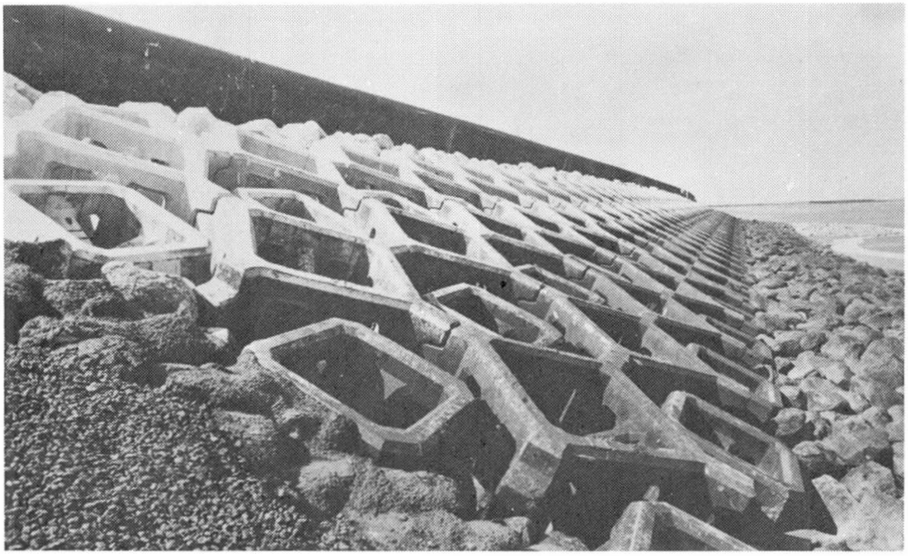

Fig. 1 View of Diode unit revetment

The second site comprised a sewage pumping station located on the shoreline. A line of 20 tonne reinforced concrete Mermade units were positioned to make a 4.5 m high retaining wall, that also acted to absorb wave energy (Fig. 2). Concrete cube strengths were 55 N/mm^2 and cover of 55 mm was used. A trial hydrophobic pore blocking (h.p.b.) admixture was used in three of the units to study the effect on corrosion durability. Trial exposure beams with varying cover, cracking and water-cement ratio were also cast with a.c. impedance inserts to measure direct corrosion rates. A number of wall units were also cast with similar inserts.

The third site was a storage barn for road deicing salt in Nottinghamshire (Fig. 3). Prestressed concrete wall panels, 110 mm thick were used in the construction of the barn. Concrete cube strengths of over 80 N/mm^2 were achieved at 28 days, with a cover to the prestressing tendons of 30 mm. The same h.p.b. admixture was used in all of the panels, with the exception of two control panels which were cast from an unmodified concrete mix.

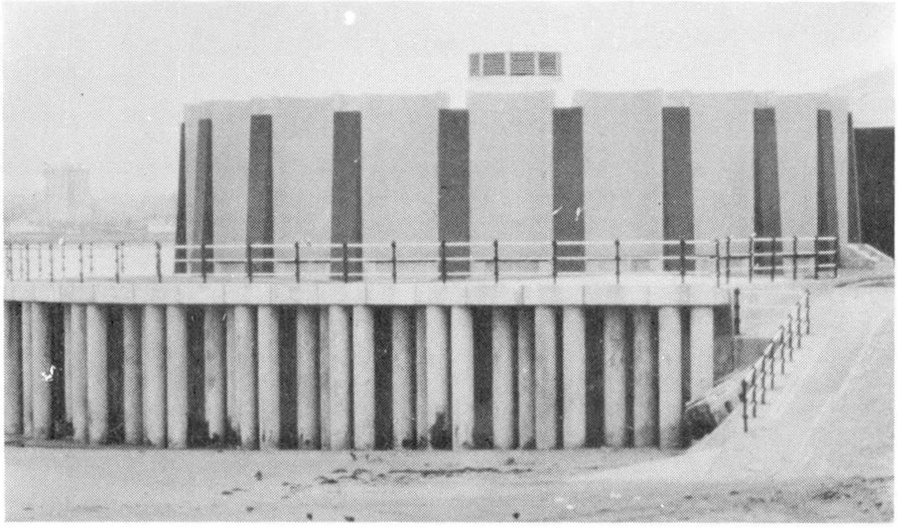

Fig. 2 View of Mermade wall and sewage pumping station

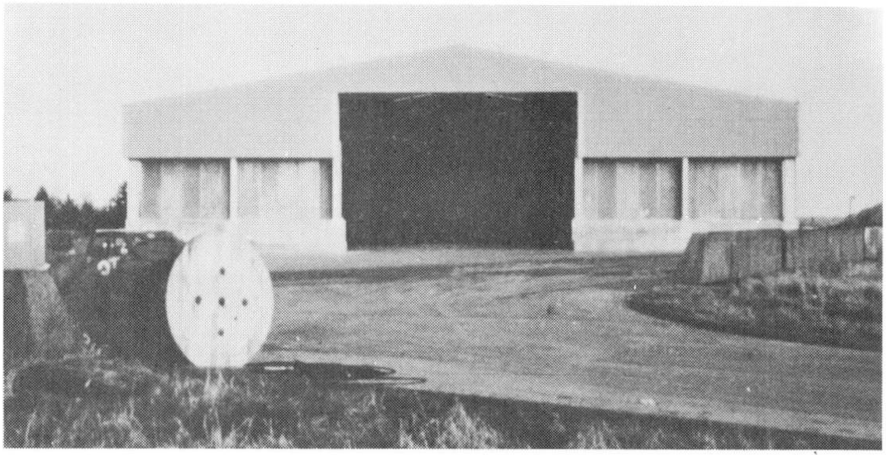

Fig. 3 View of Nottinghamshire salt barn

4. PRELIMINARY RESULTS

A durability project will inevitably be a medium to long term study. The results given below are only those obtained in the first two to five years after construction and may not necessarily reflect the long term trends. However, some initial findings are:

- Almost all of the coastal armour units sampled have potential results in the range 0 to -200 mV, with respect to a copper/copper sulphate half-cell. There are no significant potential differences or large potential gradients. This would imply that chlorides have not yet depassified the steel reinforcement.

- The only exceptions to the above are the cases of three units that have suffered moderate impact damage, exposing the steel. High potential values of -400 mV to -500 mV and steep potential gradients were recorded in the vicinity of visibly corroding steel. One year after repair with a polymer mortar, these units had become repassivated and displayed no significant potential results.

- The apparent resistivity of the ordinary concrete armour units is 5 kΩ.cm to 20 kΩ.cm, which would place them in a high risk category [6]. The units using a pozzolanic cement replacement had significantly higher resistivities in the range of 10 kΩ.cm to 100 kΩ.cm. This places them in the medium to low risk category.

- The retaining wall units would not be expected to be showing signs of corrosion activity just 18 months after construction. However, a.c. impedance studies on the exposure beams cast at the same time are showing significant corrosion activity in the ordinary concrete specimens that are either cracked, have low cover or have a high water-cement ratio. Each of these features may be expected to increase the risk of depassivation. A high water-cement ratio will lead to high permeability whilst low cover will decrease the time for chlorides to reach the reinforcement generally. Cracks will provide easy localised access to the steel for both chlorides and carbonation. There is no activity in any of the specimens using the h.p.b. admixture.

- Preliminary potential mapping of the salt barn panels have shown no anodic regions to date on either the unmodified concrete control panels (01 and 02) or on the panels containing h.p.b. admixture (C1 and C2). The results of I.S.A.T. and Figg permeability tests together with resistivity measurements on dried test cubes cast with each panel are shown in Table 1.

Concrete Specimen	Mean Figg result (secs.)	Mean I.S.A.T. result ($ml/m^2/sec$)			Mean Resistivity (kΩ.cm)
		@ 10 mins	@ 30 mins	@ 60 mins	
01	4471	0.190	0.125	0.095	29.5
02	2200	0.165	0.100	0.075	30.6
C1	7188	0.023	0.018	0.001	39.7
C2	6026	0.006	0.003	0.002	48.2

Table 1 Test results from salt barn panels

Both the permeability tests indicate that the concrete with h.p.b. admixture is several times less permeable than the unmodified control concrete. The I.S.A.T. water permeability test is considerably more sensitive to presence of the admixture. This was attributed to the hydrophobic properties of the concrete provided by the admixture. The electrical resistivity of the concrete with the admixture was also some 50% higher than that of the control concrete. The measurements indicate that the concrete containing h.p.b. admixture will be better able to resist the ingress of chlorides. Furthermore, if depassivation of the reinforcing steel does occur then the higher resistivity of the modified concrete should result in a significantly lower rate of corrosion due to the restricted flow of hydroxyl ions from cathode to anode regions. The lower permeability will also restrict oxygen passing through the concrete cover and fuelling the cathodic reaction.

5. CONCLUSIONS

Using steel reinforced concrete in chloride rich environments will often place the structure at high risk from corrosion attack. To minimise this risk use must be made of high strength concretes with large covers to the reinforcement. High quality workmanship must also be used to ensure an impermeable, uncracked concrete is obtained. The preliminary experimental results also suggest that using a modified concrete mix with a pozzolanic cement replacement or a h.p.b. admixture will further reduce the corrosion risk. In the case of pozzolanic cement replacements the rate of corrosion activity is likely to be reduced as the result of the higher resistivity. Although the resistivity increase with h.p.b. admixtures is more modest, the accompanying reduction in permeability should significantly reduce ingress of chlorides, carbonation penetration, and oxygen flow. However more longer term results are awaited before these initial trends can be confirmed.

6. ACKNOWLEDGEMENTS

The authors are grateful for the use of site facilities provided by the Metropolitan Borough of Wirral and by the Nottingham County Council in conjunction with these studies. Thanks are also due to Costain Concrete Co. Ltd., for their assistance in setting up the test requirements.

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Identification of Structural Properties Using Dynamic Tests

Surveillance des structures à l'aide d'essais dynamiques

Bestimmung der Struktureigenschaften mit dynamischen Tests

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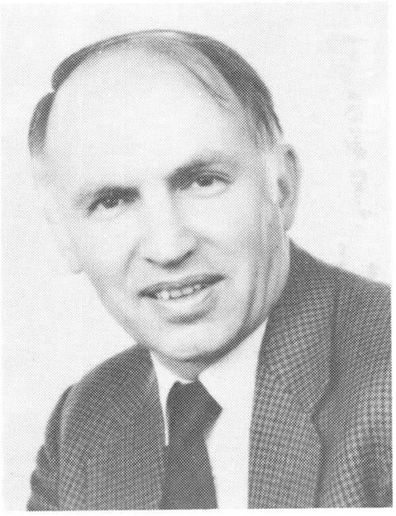

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SUMMARY

Since the dynamic properties of structures are very susceptible even to small changes of local stiffness and mass, dynamic testing promises to be a suitable tool for the integrity monitoring of building structures. For this purpose a procedure was developed, a suitable test equipment was built and tested. Tests have been performed on a test beam as well as several times on a bridge as part of a complete inspection program.

RÉSUMÉ

Comme les caractéristiques dynamiques des structures sont très sensibles même aux petits changements de la rigidité locales et de la masse, l'application des essais dynamiques semble être un moyen approprié à la surveillance des structures. Un procédé a été développé, un équipement réalisé et testé. Des tests ont été faits avec une poutre ainsi que sur un pont dans le cadre d'un programme d'inspection.

ZUSAMMENFASSUNG

Da die dynamischen Strukturparameter sehr empfindlich auch gegenüber kleinen Änderungen der lokalen Steifigkeit und Masse sind, verspricht die Anwendung dynamischer Tests ein geeignetes Hilfsmittel für die Überwachung von Bauwerken zu sein. Zu diesem Zweck wurde ein Verfahren entwickelt, eine passende Versuchseinrichtung zusammengestellt und getestet. Versuche wurden sowohl an einem Testbalken als auch an einem Brückenbauwerk im Rahmen eines Inspektionsprogrammes durchgeführt.

1. INTRODUCTION

In the FRG the major part of bridges has been constructed in the sixties and seventies of this century. In the last years damages on many bridges became obvious and reached such an extent, that they had to be repaired. Therefore the costs for rehabilitation of bridges increased to a value of about 500 millions DM per year. In the same time the density of traffic grew up, so that bridge structures are subjected to higher stresses. Additionally the restrictions of traffic caused by repair of bridges have to be minimized.

It is obvious, that the repair of a damage in a stage just occured will be less consumable in time and money. But therefore it is necessary to have techniques available, which enable an early detection and localization of damages and their sources. Several methods actually are under development for this purpose. Because of their different physical basis they can handle different problems of damage detection. So a bundle of procedures has to be defined, which may be used in the particular steps of investigation.

The main objective of the integrity monitoring of concrete structures is to identify and to quantify changes in the load carrying capability. Since cracks may significantly impact the stiffness as well as the durability of a structure the attention is primarily directed towards monitoring of cracks and detection of their sources. In the past this was mainly performed by visual inspection. But this method may fail, when the cracked area is not accessible. For example, cracks may remain undetected, if they are in top of the sections on supports in multi-span bridge systems.

In this paper a method shall be presented, which provides the examination of bridges with dynamic tests and the detection and rough localization of damages.

2. THE DYNAMIC TEST METHOD

Systematic vibration monitoring aiming at the identification of dynamic parameters such as natural frequency, damping and mode shapes offers substantial means to quantify the stiffness changes and to localize their sources. If events like overloading, restraint or loss of prestressing result in cracks or high stress concentrations, stiffness and damping properties of the vibrating system change. With increasing damage the natural frequencies decrease whereas the damping values generally increase. Furthermore, modes which have large relative rotations in damaged zones, especially the higher modes, exhibit a higher relative change in their eigenfrequencies and thus give indications on potential damage zones. Since many mode shapes can be excited and checked with regard to their properties, for the detection of damages dynamic monitoring provides a much larger data basis than static displacement measurements.

Vibrations can be induced either by ambient vibrations or by artificial excitation (impulse, eccentric mass exciter or hydraulic actuator, Fig. 1). In any case system identification tests are conducted at very low-level excitations since testing may not cause any damage to the structure or activate other sources of non-linear behaviour. Hence, sensitive transducers are needed to capture the dynamic response of the structure.

Fig.1 Hydraulic actuator and transducers on Bridge Ulenbergstrasse

Having identified the dynamic properties of a bridge shortly after construction, a representative mathematical model can be developed by fitting parameters to those gained from the experimental data. Evaluating the data from periodically performed measurements the eigenfrequencies, damping values and mode shapes can be determined and compared with the reference values of the first measurement.

If changes are identified within the further measurements, adequate methods must be used to fit the mathematical model to the new data. This step is necessary to determine the change of structural properties such as stiffness, mass and stresses which are essential to draw conclusions concerning the state of the structure. For the interpretation a profound knowledge on nonlinear force-deflection characteristics of R/C bridges with emphasis on cracking and its effects on stiffness and damping properties must be available.

Considering the appreciable size of a bridge structure, the diversity and inhomogenity of the materials and the various environmental effects such as traffic flow, temperature, humidity, etc., it can be easily recognized that monitoring of a concrete structure is associated with serious limitations and uncertainties. In addition special phenomena like the interaction of the superstructure with nonstructural and substructural elements may pose further serious questions regarding the interpretation of the measured data.

3. LABORATORY TEST

In order to check the method on a structure with well known properties and boundary conditions a prestressed concrete beam with a span of 20 m was tested in the laboratory. This test enabled the comparison of the results with data gained from the measurement of deflections, concrete strains and steel stresses.

The beam was tested dynamically in the uncracked stage and then statically loaded until cracks appeared. After applying an alternating loading with about 28000 cycles as a simulation of the influence of traffic the beam was tested again without any load. In Fig. 2 transfer functions of the uncracked and cracked beam are plotted.

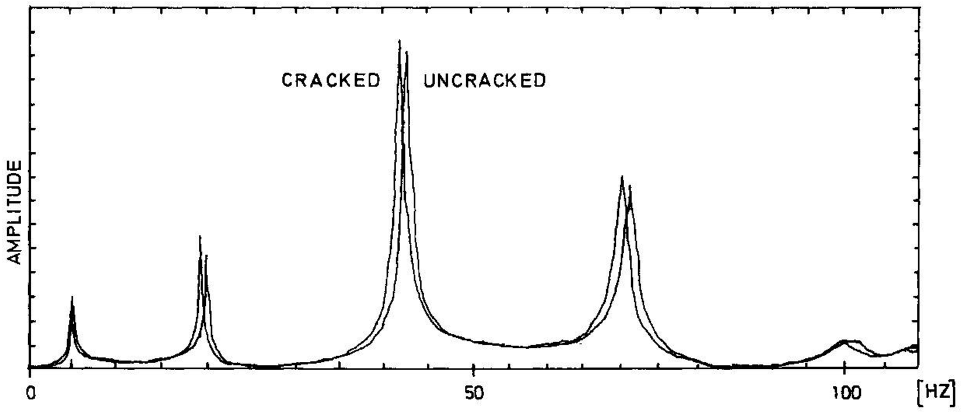

Fig. 2 Transfer functions of the test beam

The results clearly show, that the decrease of stiffness due to cracking can be recognized by dynamic testing.

4. INTEGRITY MONITORING OF BRIDGE ULENBERGSTRASSE

4.1 Structural system and test equipment

Bridge Ulenbergstrasse is a two-span roadbridge in Duesseldorf designed for the load class 60/30 (tons) according to DIN 1072 (Fig. 3). In the longitudinal direction the bridge is prestressed with 59 glassfibre tendons providing a tensile working force capacity of 600 kN per unit.

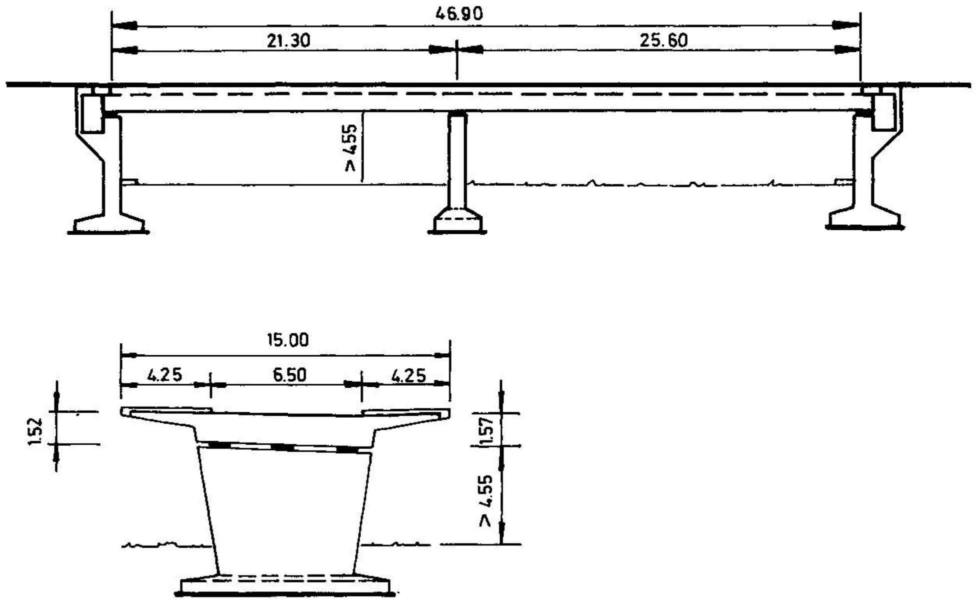

Fig. 3 Longitudinal section and cross section of Bridge Ulenbergstrasse

Prior to the application of the first dynamic test various analytical models for the bridge superstructure have been studied with regard to natural frequencies and mode shapes. A simple beam model has been quite sufficient to reflect the bending modes but because of the high width/span ratio of the superstructure it has been necessary to develop a 3D-model which captures the geometry entirely.

Based on these analytical estimates the optimal locations for artificial excitation as well as measurement points have been established. The bridge has been excited dynamically using a hydraulic actuator mounted on a concrete block and placed on the superstructure (Fig. 2). Accelerating the mass (460 kg) on the jack with a random noise in the range of 0 - 64 Hz, vertical forces have been applied to the structure. In order to avoid any damage to the structure the force level has been limited to 15 kN.

Some of the basic criteria which led to this type of excitation are:

- This excitation technique is capable to excite a wide range of frequencies at the same time and can reliably be reproduced multiple times.
- The applied force can be kept almost constant over the entire frequency range and be controlled easily.

The dynamic response of the bridge has been recorded using sensitive seismometers whereby in each configuration three measuring points and one reference point (top of exciting mass) have been recorded. The signals have been amplified and transferred to a 4-channel FFT analysator. Autospectra and transfer functions have been calculated and stored. Using a modal analysis software package the modal values of the structure have been obtained.

4.2 Results

On this bridge 4 tests have been performed in steps of 3 months, 6 months and twice one year. The first 6 natural frequencies of the first three measurements are listed in Tab. 1. Test 4 isn't still evaluated completely. The comparison of the results clearly shows a nearly equal increase of the natural frequencies, which is caused by the hardening of the concrete.

MODE	1ST TEST	2ND TEST	3RD TEST
1	4.97	5.13	5.14
2	7.14	7.37	7.47
3	9.87	10.1	10.2
4	11.9	12.2	12.1
5	16.6	17.8	17.5
6	21.7	22.4	22.3

Tab. 1 Natural frequencies of Bridge Ulenbergstrasse [Hz]

Comparing the results derived from the measurements and those obtained by analytical model discrepancies could be observed. Therefore the model was improved capturing the bearing characteristics. Studying the interaction between the main bridge structure and the sidewalk plates lower and upper limits of interaction could be defined. The comparison with the test results shows, that these lie between these limits. Nevertheless, it has to be noticed, that not all influences on the dynamic properties could be well identified in the analytical model.

5. ACKNOWLEDGEMENT

The authors want to return thanks to the European Community, which is sponsoring the work in the framework of the BRITE proposal 13534-85 "Testing of Structural Integrity of Building Structures and Measurement of the Position of th Reinforcement".

Measures against Deterioration of Concrete Bridges due to Chloride Ions

Mesures afin d'empêcher la détérioration des ponts en béton sous l'action des ions de chlorure

Massnahmen gegen die von Chloridionen verursachten Schäden an Betonbrücken

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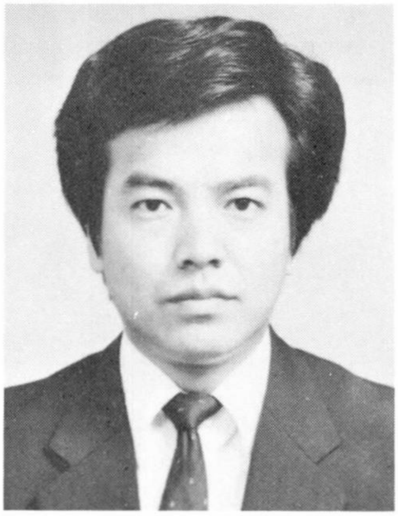


Koichi Minosaku, born 1956, received M. Eng. at Tottori University in 1982. He has been engaged in bridge engineering works at the Ministry of Construction since 1982.

SUMMARY

This report presents an outline of "The Design and Construction Guidelines for Coastal Structures (Draft)" based on a survey on the chloride ion and the corrosion of steel in the existing concrete bridges, and on measurement of airborne chloride ions in coastal areas all over Japan.

RÉSUMÉ

Cet exposé présente une description des "Directives de projet et de construction relatives aux structures côtières (projet)", sur la base des résultats d'une enquête portant sur les ponts existants et relative à la relation entre la teneur en ions de chlorure et la corrosion de l'acier, prenant en compte les mesures des ions de chlorure transportés dans l''air des zones côtières du Japon.

ZUSAMMENFASSUNG

Der Beitrag gibt einen Überblick über "Entwurfs- und Konstruktionsrichtlinien für Küstenbauwerke (Entwurf)", welche auf einer Reihenuntersuchung bestehender Brücken und der Beziehung zwischen Chloridionengehalt und Stahlkorrosion sowie einer landesweiten Messung der Chloridionen der Küstenluft basieren.

1. Introduction

In recent years, some concrete bridges mainly in coastal areas where the effects of splash and sea breeze are strong have exhibited cracking and delamination of concrete and corrosion of steel materials at early stages after the construction due to chloride ions penetrating into the concrete. In response to this situation, the Ministry of Construction carried out a macroscopic survey in 1982 for 920 concrete bridges located in areas within approximately 500m from the shoreline with respect to deterioration due to chloride ions. As a result, the following points were obtained.

(1) Deterioration due to chloride ions is intensive in coastal areas of Hokkaido, Tohoku and Hokuriku Regions along the Sea of Japan where the north-west seasonal wind is strong, and in Okinawa Prefecture which belongs to the subtropical region and is regularly attacked by Typhoons, as shown in Figure 1.

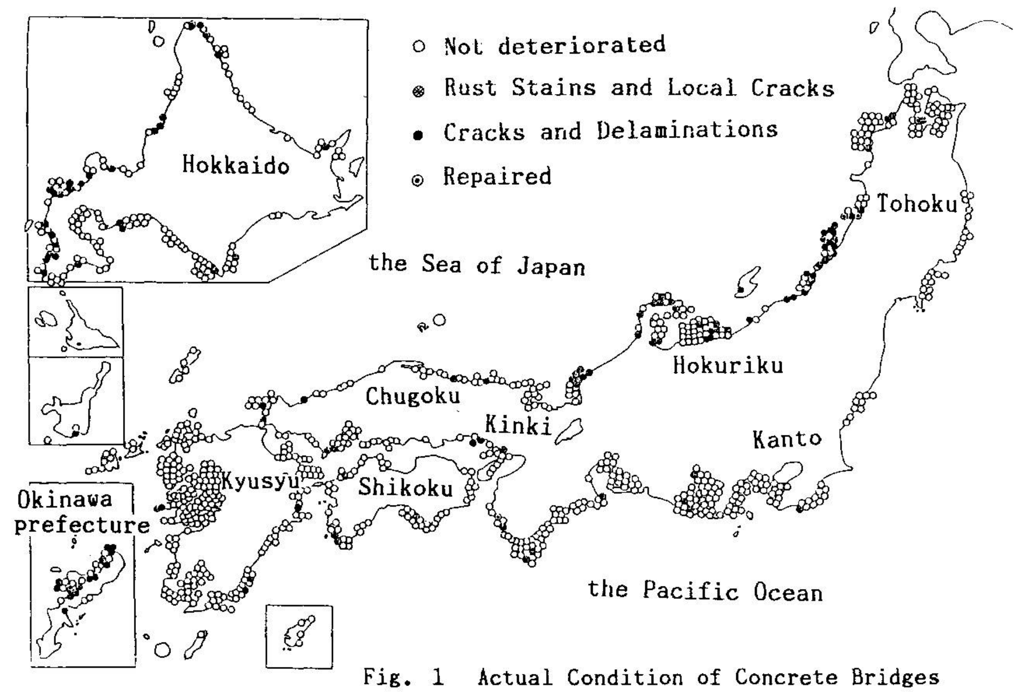


Fig. 1 Actual Condition of Concrete Bridges

(2) The majority of deteriorated bridges are located in areas within 300m of the shoreline, and those within 100m show a higher rate of deterioration.

The Ministry of Construction conducted measurements of airborne chloride ions in coastal areas all over the country as well as a survey concerning the chloride ion content in concrete and the corrosion of reinforcement of two concrete bridges which were built more than 10 years ago. This report describes an outline of "The Design and Construction Guidelines for Coastal Structures (Draft)" prepared based on these results.

2. Outline of "The Design and Construction Guidelines for Coastal Structures (Draft)"

2.1 Areas requiring measures against deterioration

The Guidelines (Draft) defines the areas requiring measures to prevent deterioration of concrete bridges due to chloride ions using the distance from the shoreline as an indicater, as shown in Figure 2. These areas are designated based on the results of a survey on airborne chloride ions as well as the above mentioned survey on the actual condition of concrete bridges.

Airborne chloride ions were measured for 3 years from 1984 to 1987 at 266 points in coastal areas all over the country. Figure 3 shows the distribution of airborne chloride ions in the country based on the survey results. The Sea of Japan coastal areas in the Hokkaido, Tohoku and Hokuriku Regions and Okinawa Prefecture, in which a number of deteriorated bridges were found, recorded a higher airborne chloride ion content. Of these areas, the Pacific coast areas in Okinawa Prefecture and the Sea of Japan coastal area in the Tohoku Region recorded the airborne chloride ion content of 0.1mg/cm² (NaCl weight) on the daily average for the 3 year period even at points of 300m and 100m away from the shoreline, respectively. On the contrary to these areas, the airborne chloride ion content is low in areas along the Seto Inland Sea in which the daily average for the 3 year period is less than 0.01mg/cm² (NaCl weight) even at points within 100m. Althouth the airborne chloride ion content in other areas such as the Pacific coastal areas and the Sea of Japan coastal areas in the Chugoku and Kyushu Regions scattered widely, it tends to become smaller at points more than 300m away from the shoreline.

2.2 Measures at the design stage

The Guidelines (Draft) indicates

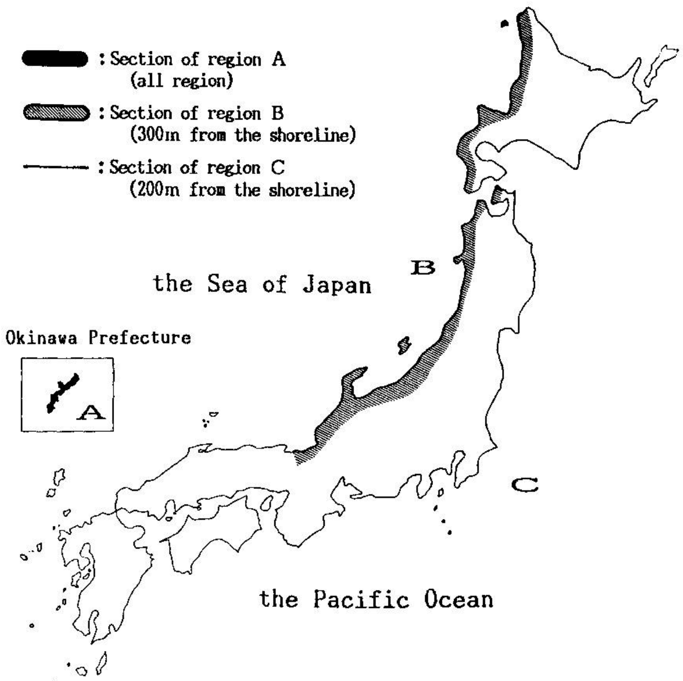


Fig. 2 Areas requiring Measures against Deterioration (Ref. Table 3)

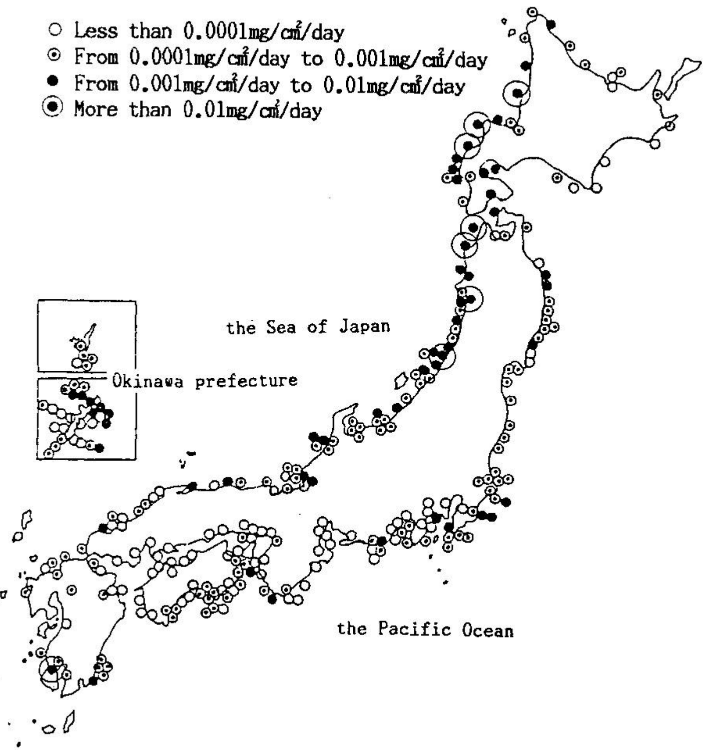


Fig. 3 Distribution of Amount of Airborne Chloride Ions (NaCl Weight)

measures for the design stage such as the shape of bridge members, the cover of reinforcement, and the materials and mixing of concrete. The basic concepts of these measures are as follows.

(1)The shape of bridge members: The chloride ions attaching to the surface of bridge members is reduced by choosing an appropriate shape.
(2)The cover of reinforcement: The chloride ions reaching the reinforcement position is reduced by ensuring appropriate cover.
(3)Concrete material: The chloride ions mixed in concrete is reduced by choosing proper materials.
(4)Concrete mixing: The quantity of chloride ions reaching the reinforcement position is reduced by using dense concrete with proper composition.

The Guidelines (Draft) also indicates that the use of coated reinforcement or the coating of concrete surfaces is effective in place of increasing the cover of reinforcement. The former measure is made to prevent directly the corrosion of reinforcement, while the latter measure is made to prevent the penetration of chloride ions into concrete.

2.2.1 The shape of bridge members

Many deteriorated bridges show that deterioration due to chloride ions concentrates on corners of members. This is because concrete placement and compaction tend to be insufficient at corners so that density of concrete is difficult to assure, and chloride ions from both the lateral and vertical directions lead to accumulate more than those in other parts. For these reasons, The Guidelines (Draft) points out that bridge members should be made to the shape with fewer corners.

2.2.2 The cover of reinforcement

Two bridges of the old Itagai and the old Dokawa located in the Sea of Japan coastal area in Hokuriku region were investigated on the cover of reinforcement, the chloride ion content, and the corrosion of reinforcement.

The old Itagai bridge built in 1978 is a reinforced concrete T-shaped girder bridge with 13.6m in length, 6.5m in width and 3 main girders, and located at the point of about 10m away from the shoreline and 5m above from the sea level. This bridge is exposed to north or north-west winds from the sea throughout the year, and to direct splash when the sea is stormy. Delamination of concrete and exposure of reinforcement were observed around the bottom of the main girders, and cracks were observed in the bridge axis on the sides of the main girders.

The old Dokawa bridge constructed in 1935 is a reinforced concrete T-shaped girder bridge with 46.6m in length, 6.2m in width and 4 main girders, and located at a point of about 40m away from the shoreline. This bridge is in an environment where north and north-west winds from the sea are predominant in winter. Concrete cracking, delamination and rust drops caused by corrosion of reinforcement were observed on the surface of this bridge.

The relation between the corrosion degree of

Table 1 Criteria for degree of Corrosion

Degree of corrosion	
A	Not corroded
B	Slight corrosion
C	Corrosion with little pittings
D	Severe corrosion with pittings

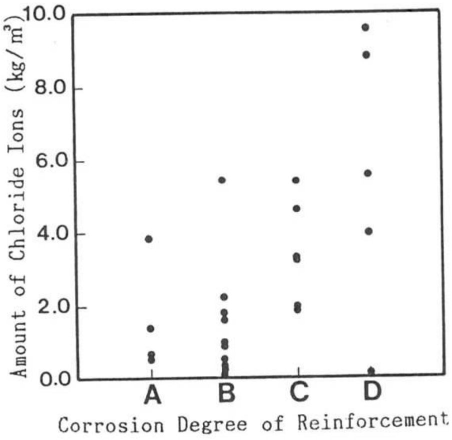


Fig. 4 Relation between Amount of Chloride Ions and the Corrosion Degree of Reinforcement (See, Table 1)

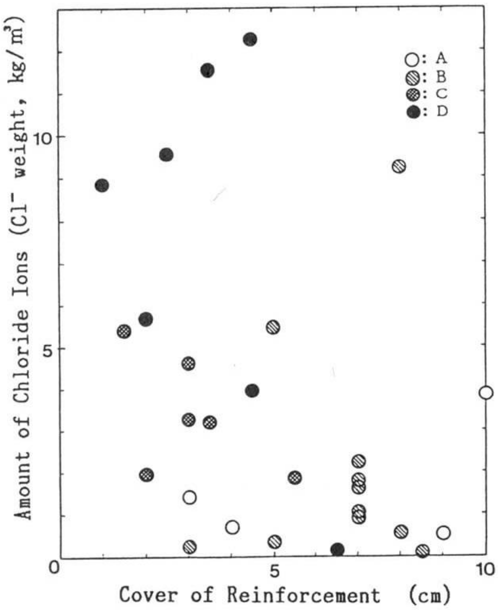


Fig. 5 Relation between the Corrosion Degree of Reinforcement (See, Table 1) and Cover of Reinforcement

reinforcement and the chloride ion content is shown in Figure 4. The corrosion degree of reinforcement was judged in accordance with the criteria shown in Table 1. The average chloride ion content at the corrosion degree C,which indicates development of corrosion, was 3.4kg/m³ with the standard deviation of 1.4kg/m³. Figure 5 shows the relation between the corrosion degree and the cover of reinforcement. The majority of reinforcement of which corrosion degree was C and D are in the range of the cover less than 5cm.

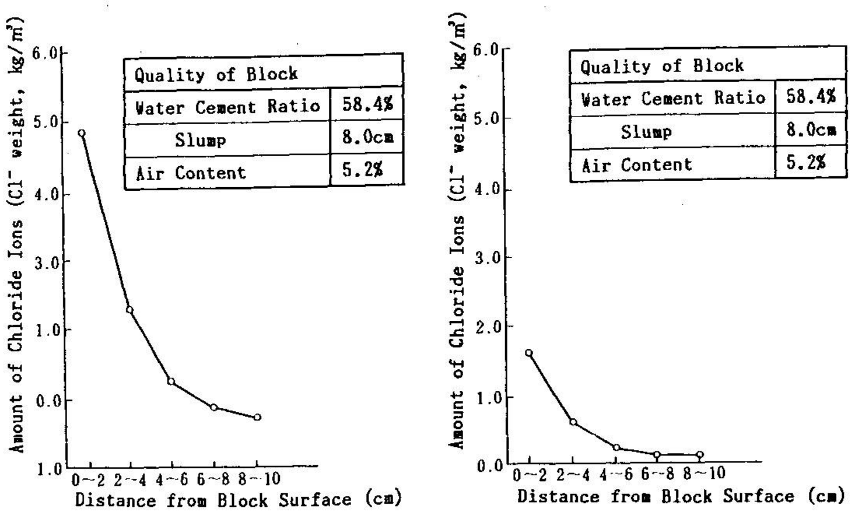
In addition, two concrete blocks and one mortar block with the dimension of 10×10×10cm were placed at 76 points, which were selected from the airborne chloride measuring points in consideration of the distance from the shoreline and other factors. Table 2 summarizes the mix proportion for each block.

Table 2 Mixing Condition for Exposing Blocks

	Water cement ratio W/C	Unit Weight (kg/m³)				Slump (cm)	Air content (%)
		Water W	Cement C	Fine aggregate	Coarse aggregate		
Mixing 1	58.4	171.2	293.0	874	959	8.0	5.2
Mixing 2	39.0	175.7	450.0	705	987	8.0	4.9
Mixing 3	57.8	203.0	351.0	1,681	–	12.5[1)]	2.3

1) Flow value

Figure 6 illustrates results obtained from this exposure test for 2 typical points. As to the block exposed at the point in the Sea of Japan coastal area in Tohoku Region where the average airborne chloride ion amounts to 0.19mg/cm² (NaCl weight), the penetrating chloride ion of approximately 2.0kg/m³ (Cl^- weigt) leading to the corrosion development deposits at 2-4cm depth from the surface during the 3 year period. On the other hand, as to the block exposed at the point in the Pacific coastal area in Kanto Region where the average daily airborne chloride ion is 0.02mg/cm² (NaCl weight), the quantity of chloride ions deposited at 2-4cm depth from the surface during the 3 year period is about 0.60kg/m³ (Cl^- weigt).Thus, the chloride ions penetrating into concrete is proportional to the concentration of airborne chloride ions. Based on these results,The Guidelines (Draft) sets the standard minimum cover of reinforcement as shown in Table 3.



(a) Exposing Point; the Sea of Japan Side of Tohoku Region, Exposing; 3years

(b) Exposing Point; Kanto Region, Exposing; 3years

Fig. 6 Distribution of Chloride Ions in Concrete

2.2.3 Concrete material and mixes

The Guidelines (Draft) indicates that cement to be used should be portland cement conforming JIS(Japan Industrial Standards) excluding ultra high early strength portland cement, or blast furance slag cement.

With respect to fresh concrete, The Guidelines (Draft) indicates that the quantity of chloride ions should be 0.6kg/m³ (Cl^- weight) or less for reinforced concrete members and 0.3 kg/m³ (Cl^- weight) or less for prestressed concrete members. And the following measures are indicated as to concrete mixes.

Table 3 Mixiмum Cover of Reinforcement of the members in the areas requiring measures

(unit : c m)

Section of region[1]	Distance from the shoreline	Kind of members		
		Slab	Girder	Column
A	Marine parts and from 0m to 100m	5.0	7.0	7.0
	Parts other than mentioned above	4.0	5.0	5.0
B	Marine parts and from 0m to 100m	5.0	7.0	7.0
	100m to 200m	4.0	5.0	5.0
	200m to 300m	3.0	3.5	4.0
C	Marine parts	5.0	7.0	7.0
	0m to 100m	4.0	5.0	5.0
	100m to 200m	3.0	3.5	4.0

1) See Fig. 2

(1) The standard water-cement ratio for concrete should be 0.55 or less.
(2) The standard slump of concrete should be 8cm or less.
(3) The standard unit cement quantity should be 300kg/m³ or more.

The item (1) is set to ensure the density of concrete. The item (2) is set for preventing concrete cracking associated with drying and shrinkage. The item (3) is set to ensure the plasticity necessary to spread the concrete uniformly around reinforcements.

In-situ Strength Assessment of Lightweight Concrete

Evaluation in situ de la résistance du béton léger

Festigkeitsbestimmung an Bauwerken aus Leichtbeton

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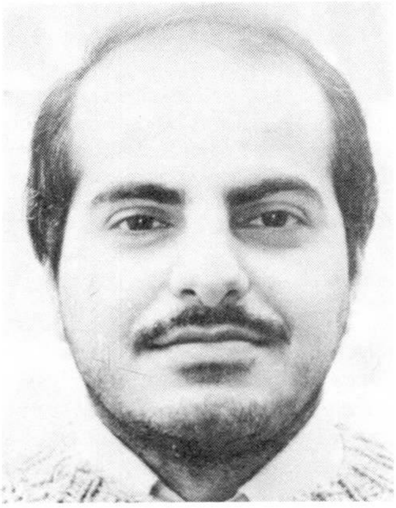

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SUMMARY

A range of non-destructive and partially-destructive test methods have been examined in terms of their reliability when used for in-situ strength assessment of lightweight concrete. These may be used with confidence provided that specially developed correlation curves are available. Testing variability has been found to be generally lower than for concrete with normal weight aggregates, possibly due to differences in failure mechanisms associated with the use of relatively weak aggregate particles.

RÉSUMÉ

Plusieurs méthodes d'essai non-destructives et partiellement destructives ont été examinées selon leur fiabilité lorsqu'elles sont utilisées pour l'évaluation in-situ de la résistance du béton léger. Celles-ci peuvent être utilisées avec confiance pourvu que des courbes de corrélation spécialement développées soient disponibles. Les variations obtenues lors des essais sont généralement de moindre importance que celles obtenues avec du béton normal, cela peut être lié aux différences dans les mécanismes de rupture associés à l'utilisation de particules d'agrégats relativement faibles.

ZUSAMMENFASSUNG

Untersucht wurden verschiedene nicht-zerstörende und teilweise-zerstörende Testmethoden im Hinblick auf ihre Verlässlichkeit bei der Bestimmung der Festigkeit von Ortbeton und Leichtbeton. Die Methoden können ohne weiteres angewendet werden, sofern für diesen Zweck entwickelte Korrelationskurven zur Verfügung stehen. Es wurde festgestellt, dass die Testabweichungen insgesamt niedriger sind als für Beton mit normalen Gewichts-Zuschlagstoffen, möglicherweise aufgrund der Unterschiede bei Fehlermechanismen, die mit der Verwendung relativ schwacher Zuschlagkörner einhergehen.

1. INTRODUCTION

It is now recognized that insitu strength evaluation of concrete by means of non-destructive and partially destructive methods has an important role to play in the building and civil engineering industries. These techniques have a wide range of applications when evaluating structural deficiencies and details of their use are given elsewhere [1].

Assessment of the strength of the concrete in structures has received considerable attention relating to natural dense aggregates, whilst concrete made of lightweight aggregates has received only limited attention. Lightweight concrete has proved itself to be a useful structural material, and applications are becoming more numerous as Engineers gain confidence. Most lightweight aggregates are artificially manufactured, and in the UK the most widely available material suitable for structural concrete is Lytag. This is produced from pulverised fuel ash (Pfa), by a sintering process [2].

A comprehensive experimental programme is being undertaken to examine the reliability and mechanisms of different methods applied to a range of lightweight concretes. In this paper the most important results obtained by six different test methods applied to fully lightweight concrete are presented.

2. EXPERIMENTAL PROGRAMME

An Ordinary Portland cement together with coarse and fine Lytag satisfying the relevant British Standards were used for all the mixes. The 24 hour water absorptions (based on oven-dried condition) for coarse and fine Lytag were 12% and 15% respectively. Four different mixes were designed with 28-day cube strengths between about 23 - 47 N/mm^2. For each mix, the following specimens were cast in four batches; 650 x 225 x 120 mm beams for 50 mm cores, 225 mm cubes for pull-out, 150 mm cubes for internal fracture and pull-off, and 100 mm cubes for pulse velocity testing.

All specimens were compacted on a vibrating table and left in the laboratory. Two curing regimes were adopted, wet and dry. Tests were carried out at ages of 7 and 28 days, except for the core tests which were performed at 28 days only.

Pull-out tests were performed on 25 mm diameter cast-in inserts using commercially available Lok test apparatus with procedures following the manufacturer's recommendations, whilst through transmission pulse velocity measurements were taken with widely used 'Pundit' equipment. The internal fracture tests using 6 mm diameter expanding wedge anchor bolts were carried out by using torquemeter apparatus (B.R.E.) as well as a modified form based on a direct pull. Pull-off tests were performed by gluing a 50 mm diameter aluminium disk to the surface of concrete followed by loading with commercially available Limpet apparatus. The 50 mm nominal diameter cores were cut vertically from the specified beams at the age of 28 days followed by trimming and capping to give overall length/diameter (L/D) ratios of 1.0, 1.4, 1.6 and 2.0. Detailed test procedures for all these methods are given elsewhere [1].

3. TEST RESULTS AND DISCUSSION

3.1 General

Table 1 summarises the average test results based on three readings for cores, pulse velocities and cube crushing strengths, and on six readings for the remaining methods. The cube compressive strengths have also been plotted against test results in figures 1 to 4. In all cases the relationship was found to be dependent on the age and curing conditions. With the exception of pulse velocities this dependency is small, and a single relationship could be adopted for practical purposes.

Mix	Age	100mm Cube Strength N/mm²		Core Strength L/D=2.0 N/mm²		Pull-Out Force kN		Internal Fracture B.R.E. N-m		Internal Fracture Direct Pull kN		Pull-Off Stress N/mm²	Pulse Vel. km/sec	
		Wet	Dry	Wet	Dry	Wet	Dry	Wet	Dry	Wet	Dry	Dry	Wet	Dry
1	7	15.5	17.3	-	-	9.2	9.5	2.05	2.10	3.22	3.70	2.77	3.39	3.39
	28	23.9	29.4	25.0	28.0	14.9	17.4	2.33	2.56	4.40	5.07	3.11	3.53	3.47
2	7	19.7	21.7	-	-	10.5	10.6	2.33	2.45	3.90	4.05	2.86	3.53	3.51
	28	32.0	34.2	26.1	30.3	15.4	17.7	2.73	3.00	5.18	5.31	3.38	3.64	3.59
3	7	23.1	27.5	-	-	13.5	14.1	2.55	2.80	4.52	4.64	3.51	3.57	3.57
	28	35.0	39.7	28.8	37.2	19.4	22.5	3.40	3.70	5.70	6.11	3.88	3.68	3.60
4	7	29.0	33.3	-	-	16.4	16.9	2.88	3.03	5.26	5.33	3.54	3.56	3.60
	28	41.5	46.9	39.1	42.7	22.9	23.5	3.48	3.75	6.64	6.73	4.15	3.68	3.61

Table 1 Summary of test results on fully lightweight concrete

Test Method	Coefficient of Variation % Test Result	Coefficient of Variation % Normal Concrete	Correlation Coefficient	95% Confidence Limit on Estimated Strength
Core	4.3	8.8	0.985	±12%
Pull-Out	5.6	7.0	0.968	±17%
Internal Fracture				
B.R.E.	9.0	15.9	0.978	±34%
Direct Pull	9.8	15.6	0.987	±16%
Pull-Off	5.7	8.0	0.986	±24%

Table 2 Statistical evaluation for partially destructive tests

Statistical analyses based on the coefficient of variation have been summarized in table 2. It can be seen that these values are significantly less than those anticipated for normal weight concrete [1], however there are indications that within member material variability may be higher due to compaction differentials. Correlation coefficients given in table 2 based on single practical curves show that in general each test method applied to lightweight concrete gives a better correlation to cube strength than expected for normal weight concrete [1]. The accuracies of strength estimations based on 95% confidence limit for strength level of 30 N/mm^2 are also given in table 2. It is clearly seen that of the six insitu testing methods, the core test along with pull-out and direct pull internal fracture tests demonstrate the best ability to assess the insitu equivalent cube strength.

3.2 Core tests

As expected, core strengths were generally found to increase with decreasing length/diameter (L/D) ratio, although for dry cores the effect was not as large and not always as consistent as anticipated. This may be due to lack of uniformity in moisture content resulting from air drying, and emphasizes the importance of use of standardised specimens soaked for at least 48 hours. Correction factors to obtain the equivalent strength of a core with L/D = 2.0 are given in table 3. Comparison with the data for small cores of normal weight concrete reported by Bungey [1] suggests that considerably less correction is required for fully lightweight concrete. A similar finding has been obtained by Swamy [3] for semi-lightweight concrete. Recommended correction factors according to A.S.T.M. [4] and British Standards [5] are also included in table 3 and it can be seen that widely accepted British Standard values overestimate those required, even for wet specimens of lightweight concrete. Analysis of correction factors related to strength level also suggests that some dependency is present, as for normal weight concrete [1]. From the limited number of results at present available this relationship is however not clearly defined and it would be prudent to keep the L/D ratio as close to 2.0 as possible. The correlation between crushing strength of lightweight cores of this ratio and cube compressive strength agrees closely with that anticipated for comparable normal weight concrete cores.

L/D Ratio	Core L/D Correction Factor				
	Lightweight Test Results		Bungey [1]	ASTM C42-82 [4]	BS 1881 pt 120 [5]
	Wet	Dry			
2.0	1.00	1.00	1.00	1.00	1.00
1.6	0.97	0.98	0.91	0.97	0.94
1.4	0.94	0.96	0.86	0.95	0.90
1.0	0.86	0.90	0.77	0.87	0.80

Table 3 Comparison of core correction factors

3.3 Ultrasonic Pulse Velocities

It is well known that correlation between pulse velocity and compressive strength will be influenced considerably by factors such as mix proportions, aggregate type and curing regime. A relationship may however be developed for a particular concrete of specific proportions under defined conditions of age, moisture and curing. It can be noted from Table 1 that pulse velocities are significantly lower than expected with normal weight concrete of comparable strengths. Table 1 also shows that the influence of curing is less significant at early ages, possibly due to the large reservoir of water absorbed in the aggregate. It is thus considered inappropriate to use a strength/pulse velocity relationship developed during early stages for longer term strength assessment since the drying out effects may be misleading. Nevertheless, insitu pulse velocity measurements may provide valuable information concerning concrete uniformity within structural members.

3.4 Pull-out, Internal fracture and Pull-off tests

Fig. 1 shows that, although of the same general form, the relationship between pullout strength and compressive strength for lightweight concrete is significantly different to that for normal weight concrete. To permit inspection of the failure mechanism some truncated cones of concrete were completely extracted following testing, and visual examination of the failure surface showed that this mostly passed through the relatively weak aggregate particles. Behaviour of the overall system is thus more homogeneous than normal weight concrete with strong aggregates and may explain the lower variability of testing. The reduced pull-out force achieved at a given strength level may also be explained by the differences in failure mechanism, with no aggregate interlock occurring [6]. It is clear from Fig. 2 and Fig. 3 that the failure force for both internal fracture loading methods applied to lightweight concrete is also reduced. This feature, coupled with the much

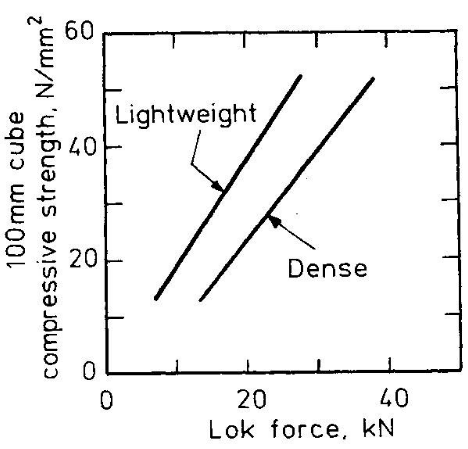

Fig. 1 Correlations between compressive strength and pull-out force

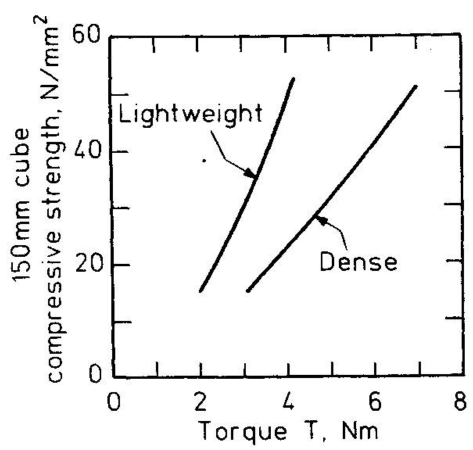

Fig. 2 Correlations between compressive strength and B.R.E. Internal fracture torque

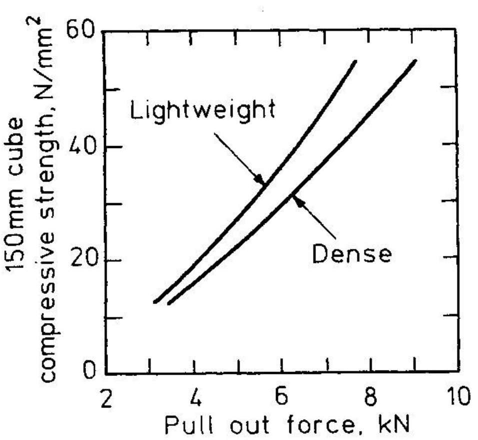

Fig. 3 Correlations between compressive strength and direct-pull internal fracture force

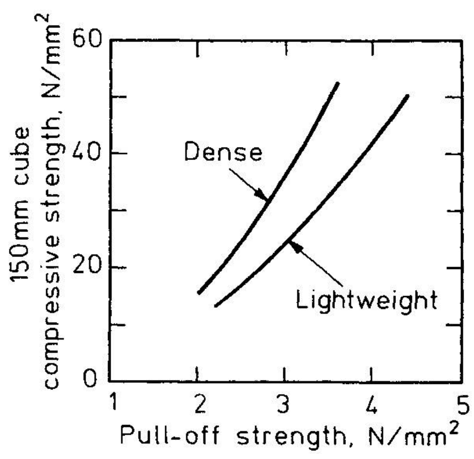

Fig. 4 Correlations between compressive strength and pull-off strength

reduced test variability, is likely to be for similar reasons. It can be noted from table 3 that the accuracy of strength estimation is improved significantly by use of the direct pull method, as also found with normal weight concrete [1].

For the pull-off tests a higher force was achieved at a given compressive strength level (Fig. 4). The reason for this is unclear at present but it is suspected that greater surface porosity may permit deeper adhesive penetration below the concrete surface, and hence increased pull-off strength. Possible differences in relationships between tensile and compressive strength may also be a contributory factor.

4. CONCLUSIONS

From the data presented in this paper it can be seen that all insitu tests, with the exception of cores, showed dependency upon the type of concrete under investigation. All also demonstrated lower testing variability for fully lightweight concrete than for that made with natural dense aggregates, possibly as a result of improved homogeneity due to the absence of strong aggregate particles. Correction factors for core length/diameter ratio were also found to be considerably reduced.

Good correlation was found to exist between compressive strength and results of each test, and accuracies of strength estimation by core, pull-out and direct-pull internal fracture methods were marginally better than assumed for normal weight concrete. Practical usage will however depend upon the aesthetic acceptability of surface damage and consequent repairs, as well as the availability of relevant correlations for the materials used.

It is recommended that ultrasonic pulse velocity measurements be confined to comparative situations, whilst any of the partially-destructive tests may be used as an alternative to cores although providing strength estimates of lower accuracy.

5. ACKNOWLEDGEMENTS

The authors express their thanks to Boral Lytag Ltd. for their generous supply of lightweight aggregates used in this research.

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Monitoring of Load Bearing Structures with Optical Fiber Sensors

Surveillance des constructions à l'aide de capteurs à fibres optiques

Bauwerksüberwachung mit Lichtwellenleitersensoren

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SUMMARY

Optical fibers, known as signal transmitters in the field of communications, are also used as sensors for the monitoring of structural elements. Optical fiber strain sensors, which are embedded directly in the concrete, provide at any time information, e. g. on nascent cracks in the monitored structural element and any change in these. Optical fiber sensors integrated into prestressing tendons made of composite fiber materials provide information on the functionality of these prestressing tendons.

RÉSUMÉ

Les capteurs à fibres optiques, connus comme signal transporteur en technique de communication, sont aussi utilisés comme détecteurs pour la surveillance des structures porteuses. Des détecteurs d'allongement à fibre optique sont directement incorporés au béton de l'ouvrage, indiquant en permanence l'état de fissuration et son évolution dans le temps. Les capteurs à fibres optiques, intégrés aux unités de précontrainte de matériaux composites, contrôlent le bon fonctionnement du système de précontrainte.

ZUSAMMENFASSUNG

Lichtwellenleiter, aus der Nachrichtentechnik als Signalübermittler bekannt, werden auch als Sensoren für die Überwachung von Bauteilen verwendet. Lichtwellenleiterdehnungssensoren, die direkt in den Beton eingebettet werden, geben zu jeder Zeit z. B. Auskunft über entstehende Risse im überwachten Bauteil und deren Veränderung. In Spannglieder aus Faserverbundwerkstoffen integrierte Lichtwellenleitersensoren geben Auskunft über die Funktionsfähigkeit dieser Spannglieder.

1. THE OPTICAL FIBER STRAIN SENSOR

In the field of communications the optical fiber is primarily and principally a signal transmitter. A prerequisite for its great significance is the excellent light transmitting capacity achieved. The attenuation of light undesired in the field of communications, dependent upon the mechanical load on the optical fiber, is utilized as a sensor effect for the monitoring of structures. In contrast to the field of communications, efforts in the development of the optical fiber sensor are directed towards obtaining as large a measuring signal as possible as a consequence of mechanical changes in the optical fiber.

Gradient fiber with a refraction coefficient diminishing away from the radius of the core, is employed as a rule for such sensor applications. The inner core is enclosed by external sheathing upon which the reflexion of light takes places. However, it is also capable of transmitting light with a lower refraction coefficient. If a ray of light is transmitted by an optical fibre, leakages occur in the range of micro-deflexions. The light leakage incurred is reported by a measuring technique as a change in attenuation in dB (decibels).

By fitting the optical fiber with a thin spiral wire (fig.1) the finding is put to use that micro-deflexions are also able to be generated as a consequence of radial pressure. Upwards of a certain length laid, the spiral wire, when pulled longitudinally, presses radially upon the optical fiber and creates micro-deflexions on it, which then cause corresponding changes in attenuation and turn the optical fiber into an optical fiber strain sensor.

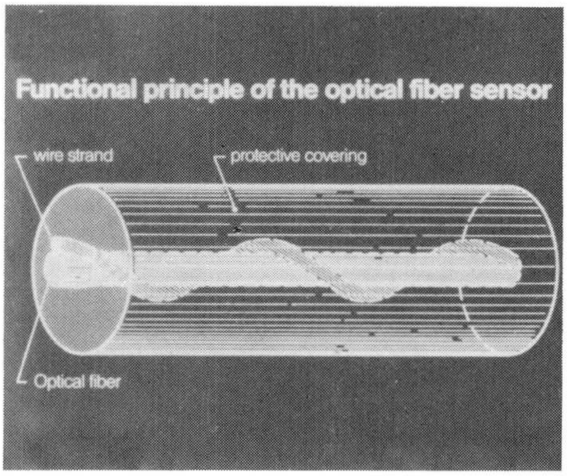


Fig. 1: Functional principle of the optical fiber sensor

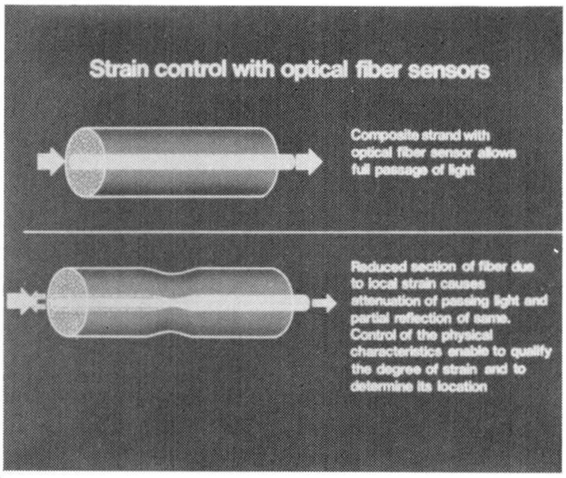


Fig. 2: Measurement principle of the optical fiber sensor

The optical fiber strain sensor displays changes in attunuation as a function of the strain producted. Since loads on concrete structures do not occur evenly, but can take place due to localized inconstancies, e.g. due to cracks, in addition to the integral attenuation measuring process, the localized fault ought also to be identifiable. For this purpose the attenuation signal which occurs more strongly at the fault's location is recordable using the backscatter measuring technique well-known in communication engineering (fig.2). The type of fault and over the localized change in strain are detectable by superimposing the attenuation curves of sensor when subjected to a load or no load.

2. MODEL EXPERIMENTS FOR THE TESTING OF OPTICAL FIBER SENSORS

Nowadays two measuring principles are available for integral monitoring, depending on whether both ends or just one end of the sensor are accessible (fig.3 and 4). The optical reference fiber serves to compensate the temperature and to correct any changes in transmitter power or in receptor sensitivity.

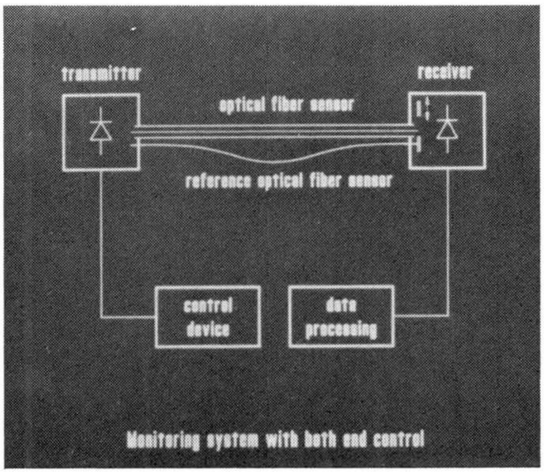


Fig. 3: Monitoring system with both end control

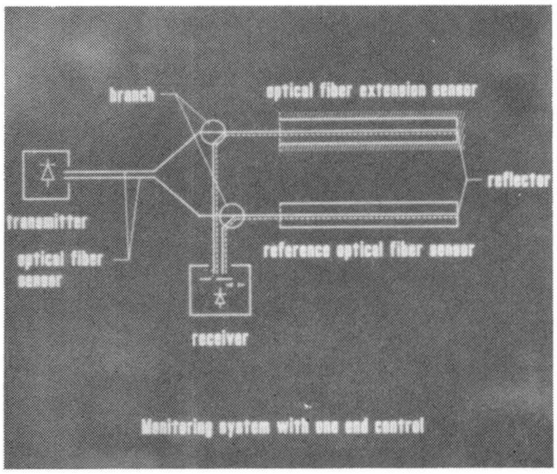


Fig. 4: Monitoring system with one end control

There are two fundamentally different possible applications in the monitoring of structures using optical fiber sensors. Firstly, the monitoring of concrete structures, e.g. during the formation of cracks and during their continued development, and secondly, the monitoring of prestressing tendons made of composite fiber materials. In the case of crack monitoring in concrete structures where these are new structures, sensors specially designed for this application are placed directly in the concrete in those positions in the structure known from static principles to be subject to a heavy load, in order to obtain evidence of any requisite repair measures at a very early stage, and hence, by this means to reduce this work considerably. In the case of existing structures, particularly in the case of structures which are already damaged, these optical fiber sensors are retro-applied to the structural elements to be monitored, in order to continue observation of the cracks which have been incurred by them, or in order to monitor repair measures which have already been performed with respect to the loadbearing capability of the structural element. In the case of composite fiber prestressing tendons, optical fiber sensors are integrated directly into the composite fiber bar, in order to indicate its undamaged condition.

In order to record evidence that these optical fiber sensors comply with the requirements made of them, initially, extensive experiments were performed in the laboratory, with the object of determining the fundamental suitability. The next stage was experiments with small prestressed beams at the University of Gent, with a span of 2,0 mtrs and an overall height of 60 cm (fig.5). In these experiments (prestressing with and without bond) all sensor applications developed to date were checked in the prestressing tendon, inside the concrete and with retro-attachment. The final field suitability test was an experiment with a 20 mtr long prestressed beam (overall height 1 mtr),

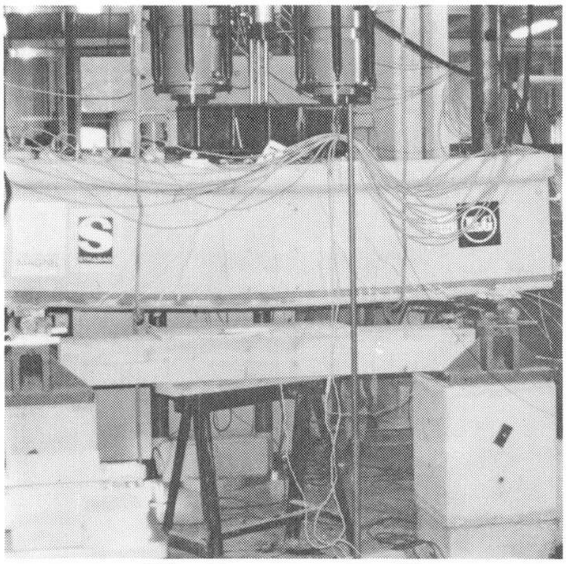


Fig. 5: 2 m-beam

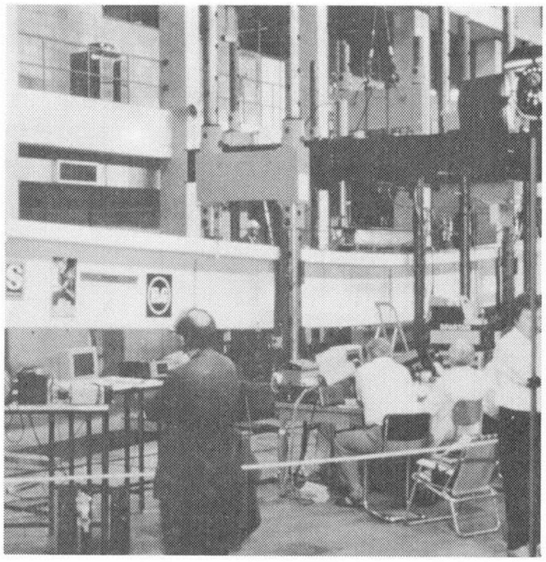


Fig. 6: 20 m-beam

likewise at the University of Gent (fig.6). In this case the suitability of these sensors for the tasks set and the selected application techniques was able to be proven impressively. With the aid of optical fiber both the transition from non-crack to cracked state and the exceeding of the steel reinforcement's yield limit were able to be determined (fig.7). These changes were equally detectable by means of optical fiber sensors in the prestressing tendon up to the failure of the beam (fig.8).

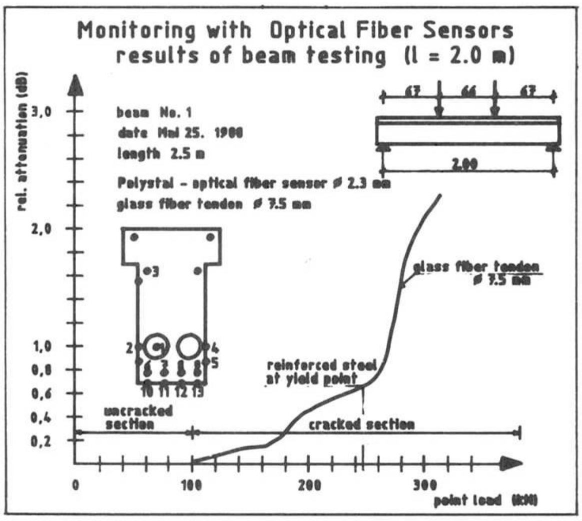


Fig. 7: Results of beam testing with optical fiber sensor in the tendon

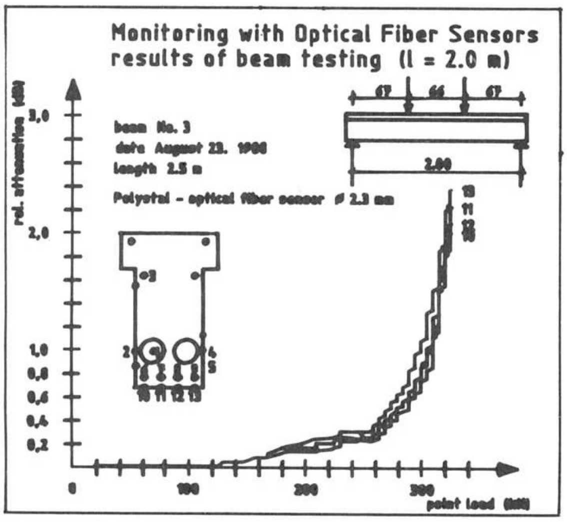
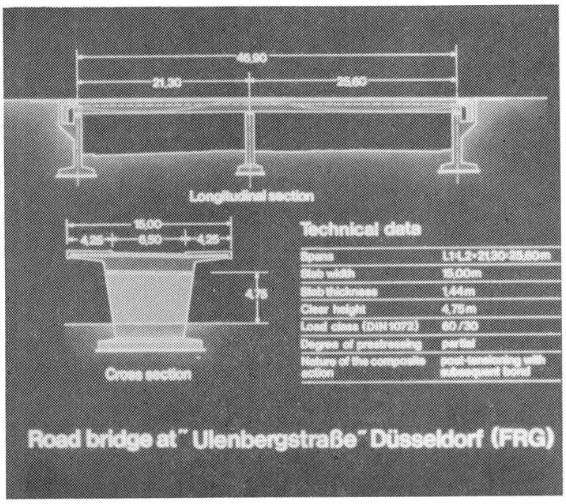


Fig. 8: Result of beam testing with optical fiber sensor in the concrete

3. THE APPLICATION

On the Ulenbergstrasse Bridge [4], the first bridge worldwide for extremely heavy road traffic loads (bridge classification 60/30) to be completed with glass fiber prestressing tendons) besides the direct embedding of optical fiber sensors in the concrete, several prestressing tendons were also equipped with optical fiber sensors. The measurements which have been carried out to date for almost 3 years now, show no change whatsoever in the structure of the bridge (fig.9).



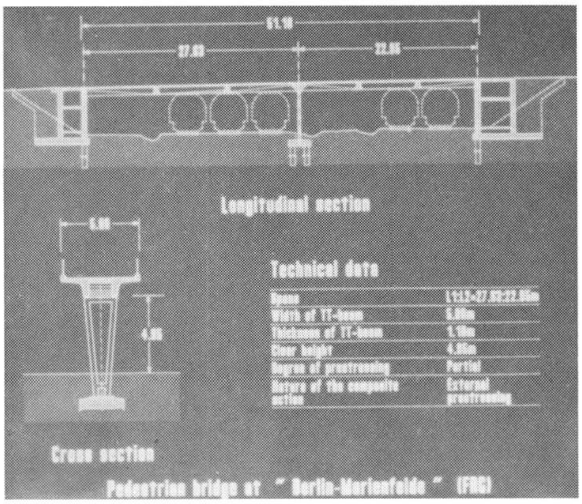


Fig. 9: Technical data
Bridge Ulenbergstrasse

Fig. 10: Technical data
Bridge Marienfelde

The design of the Berlin-Marienfelde pedestrian bridge (fig.10), a two span slab/beam bridge, is an example of the first fully monitored bridge structure. The prestressing of this bridge, designed as external prestressing withon bond, is generated by seven 19-bar composite fiber glass prestressing tendons. In addition to all those prestressing tendons in which sensors are integrated, a whole series of optical fiber sensors have been embedded directly inside the concrete or have been retro-applied. During a trial loading carried out in November using 250 concrete slabs (weight per slab 1 to.) this trial loading was able to be monitored with the aid of sensors. By using particulary highly sensitive sensors, evidence was even able to be recorded on the dynamic behaviour of this bridge.

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Checking the Reliability and Durability for Corrosion

Evaluation de la fiabilité et de la durabilité d'élements soumis à la fatigue par corrosion

Kontrolle der Zuverlässigkeit und Dauerhaftigkeit gegen Ermüdung bei Korrosion

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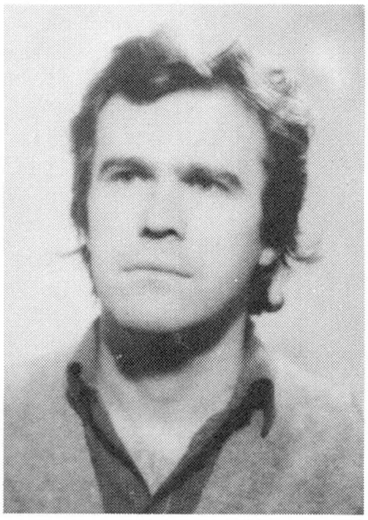

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SUMMARY

This paper deals with reliability and durability assessment procedure for engineering components with respect to corrosion fatigue. The problem of safety concept is treated as the probability of bearing capacity failure, expressed with its operative value safety index. The objective of these results, where fatigue is analysed phenomenologically, is to use the known facts as a basis for a quantitative estimate of the risk level at corrosion fatigue.

RÉSUMÉ

L'article traite l'évaluation de la fiabilité et de la durabilité d'éléments structuraux soumis à la fatigue par corrosion. On a assimilé le problème de sécurité à la probabilité de renoncement de la capacité portante exprimée par la valeur de l'indice de fiabilité. Le but de cette étude où la fatigue est analysée de façon phénoménologique, est d'appliquer ses résultats pour l'estimation quantitative du niveau de risque de fatigue par corrosion.

ZUSAMMENFASSUNG

Im vorliegenden Bericht werden Zuverlässigkeits- und Dauerhaftigkeits-Bewertungsverfahren von Konstruktionselementen bei Korrosionsermüdung behandelt. Das Sicherheitskonzept wird als Versagenswahrscheinlichkeit gegeben und durch ihre operative Grösse, den Sicherheitsindex, ausgedrückt. Die Absicht dieses Vorgehens besteht darin, bekannte Tatsachen als Grundlage für quantitative Schätzungen des Risikoniveaus bei Korrosionsermüdung zu verwenden.

1. INTRODUCTION

Causes that bring about failure due to fatigue in steel structures take place on micro-structure level and are subject to local defects due to fabrication and erection process ,and are also subject to corrosion during service.Already small changes in local properties of material have been found to influence the results even in strictly controlled conditions of laboratory sample testing,so the problem of fatigue safety can be adequately treated only with mathematical statistic models by applying fracture mechanics.

This paper concentrates on steel railway bridge structures with fatigue due to corrosion.Numerous inspections have detected corrosion process on steel bridge parts. Corrosion on steel bridge structures is mainly due to two reasons.The former is underestimation of corrosion fatigue,and the latter is lack of access to different parts of the structure for repeated corrosion protection.

The objective of this paper is to use the facts known so far as a basis for estimating the influence of corrosion fatigue by means of an operating engineering procedure where safety is treated as probability of bearing capacity failure.

2.INFLUENCE OF CORROSION ON FATIGUE

The fatigue damage increases with the duration of exposure to corrosion or factors causing it.The decreased fatigue strength depends on the kind of stress,category of the part, chemical composition and mechanical properties of the steel, aggressiveness of the corrosion medium,and corrosion protection system.If fatigue strength is analysed phenomenologically by means of the Wohler's line,it can be concluded that,in the case of corrosion fatigue,there is no area of indefinite amplitudes that the element can bear,but the $\Delta\sigma$-N line continues to zero as seen in Fig.1.

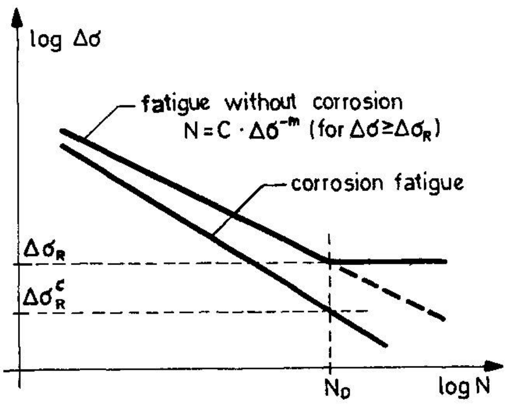

Fig.1 Phenomenological analyses of corrosion fatigue

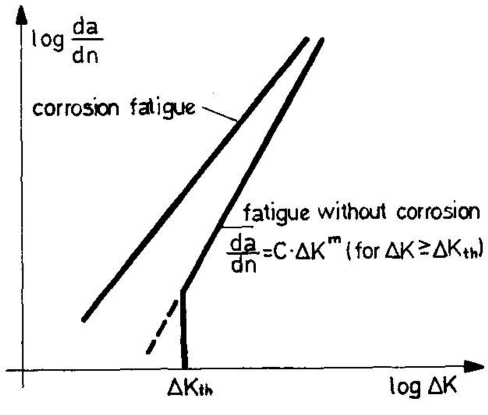

Fig.2 Corrosion fatigue expressed by fracture mechanics

Similar considerations can be applied in fatigue analysis by applying fracture mechanics,where in diagram $\frac{da}{dn}-\Delta K$ in the case of corrosion fatigue one can notice a decreased critical stress intensity ΔK_{th} (Fig.2.)Test results [1]in steels with the strength of 400 to 700 N/mm2 have shown an average drop in strength at corrosion fatigue.

3.DURABILITY AND RELIABILITY OF ELEMENTS SUBJECTED SIMULTANEOUSLY TO BOTH FATIGUE AND CORROSION

Durability and reliability at fatigue can be estimated probabilistically with the Hasofer-Lind method,where some statistical values of loads and strength are known.Density functions of these values change with time,which means that reliability ,i.e. its operational value(safety index β), also changes with time.This explanation is shown in Fig.3.

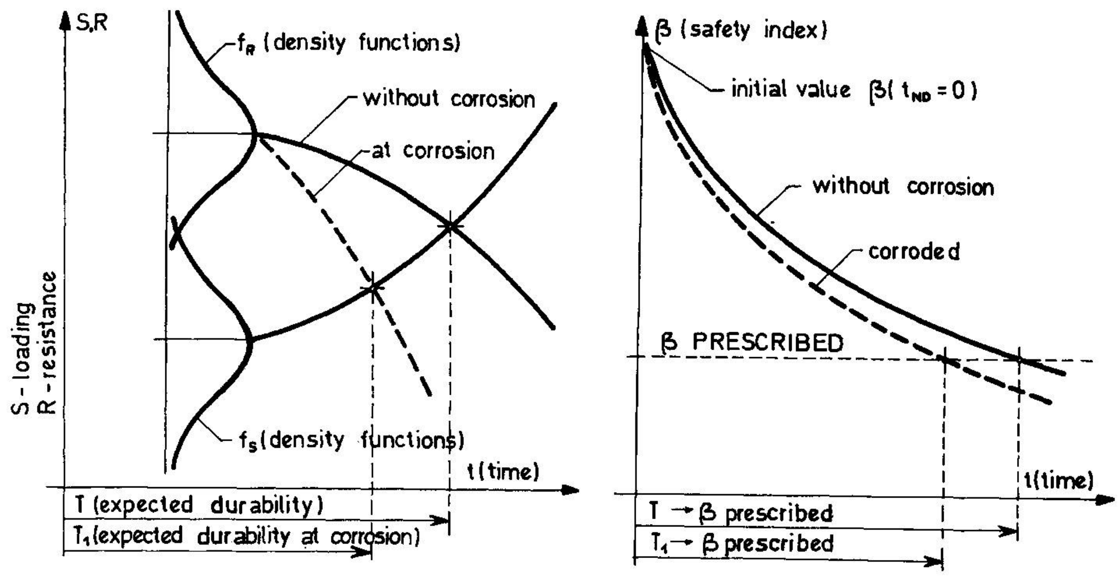

Fig.3 Relation of reliability and durability at corrosion fatigue

A general expression for the limit state equation for an n-dimensional vector space can be written in the following form:

$$Z=G[\vec{X}(x1,x2.....xn);\ \vec{K}(k1,k2....kn)] \quad /1/$$

where
Z - safety margin
G - bearing capacity value function(mechanical model)
$\vec{X}$(x1,x2.......xn) - basic variables vector and its components
$\vec{K}$(k1,k2 kn) - deterministic parameters vector and its components
n - vector space dimension

Expression /1/ in the case of fatigue can be written in the following form:

$$Z = D(t) - \int_{r_d}^{r_g} \int_{\Delta\sigma_d}^{\Delta\sigma_g} \frac{n(t,\Delta\sigma,r)}{N(\Delta\sigma,r)}\, d\Delta\sigma \cdot dr = 0 \quad /2/$$

where
D(t) - total damage in time t
rd,rg, $\Delta\sigma_d, \Delta\sigma_g$ } - lower and upper limits of double integral

$n(t, \Delta\sigma, r)$ - random number of stress variation in time t with stress difference $\Delta\sigma$ and the stress ratio r

$N(\Delta\sigma, r)$ - random number of stress variations with parameters $\Delta\sigma$ and r from Wohler's experiment (specimen fracture with constant amplitude)

If corrosion is present at fatigue, lower limit of double integral can be zero.

4.RESULTS FROM PRACTICE

Reliability of four different detail categories was analysed on a steel bridge element (Fig.4).

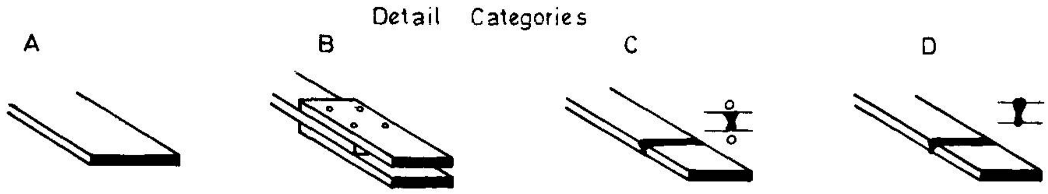

Fig.4 Analysed detail categories

The calibration procedure of existing railway bridges on the Zagreb-Karlovac railway line provided the safety indices for uncorroded details A,B,C and D according to[2].

SAFETY INDICES β									T(YEARS) l(SPAN)			
$\alpha = \frac{\Delta\sigma_R^c}{\Delta\sigma_R}$	T=10 l=4m				T=10 l=10m				T=50 l=10m			
	DETAIL				DETAIL				DETAIL			
	A	B	C	D	A	B	C	D	A	B	C	D
1.0	7.51	5.10	3.79	2.68	9.43	7.16	5.36	4.33	6.71	4.60	3.34	2.32
0.9	7.30	4.81	3.60	2.46	9.31	6.98	5.21	4.16	6.62	4.39	3.29	2.16
0.8	6.96	4.44	3.36	2.17	9.15	6.74	5.07	3.95	6.35	4.13	3.01	1.96
0.7	6.58	3.97	3.04	1.82	8.81	6.43	4.84	3.69	6.04	3.81	2.78	1.72
0.6	6.09	3.28	2.59	1.35	8.56	6.02	4.56	3.32	5.67	3.37	2.49	1.40
0.5	5.38	2.18	1.92	0.74	8.05	5.43	4.16	2.76	5.16	2.74	2.06	0.96

Table 1 Decrease of safety indices due to corrosion fatigue

The method and way of computer processing as well as the input data are presented in the paper [3].If corrosion circumstances appear,the basic variable of fatigue strength is lowered.Table 1 presents the drop of reliability index depending on assumed different percentages of fatigue strength drop.

The following numerical analyses comprise the relationship of the reliability index and the life span of railway bridges from corrosion fatigue aspect.Fig. 5 shows this relationship for various detail categories,i.e.$\alpha = \frac{\Delta\sigma_R^c}{\Delta\sigma_R} = 1$ (uncorroded detail fatigue) and $\alpha = 0.8$ (corrosion fatigue strength if decreased by 20%).

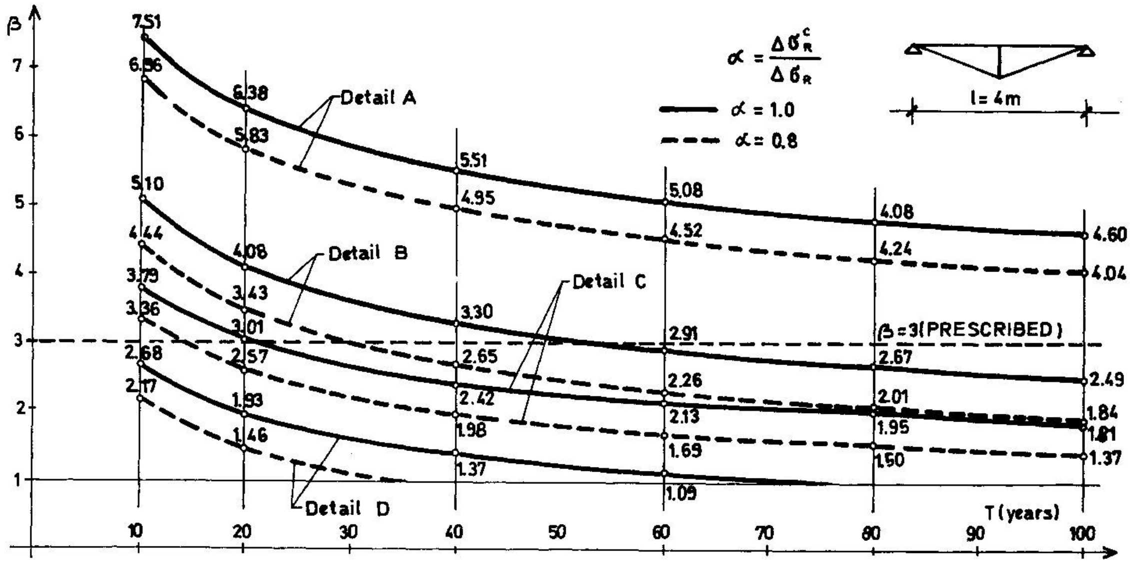

Fig. 5 Durability decrease at corrosion fatigue (α =0.8)

Fig. 6 shows the durability decrease of detail B for various coefficients.These analyses relate to our oldest bridges because rivets were the commonest way of linking the parts.It is highly probable that corrosion process was present here due to negligence or inability to renew the protection.

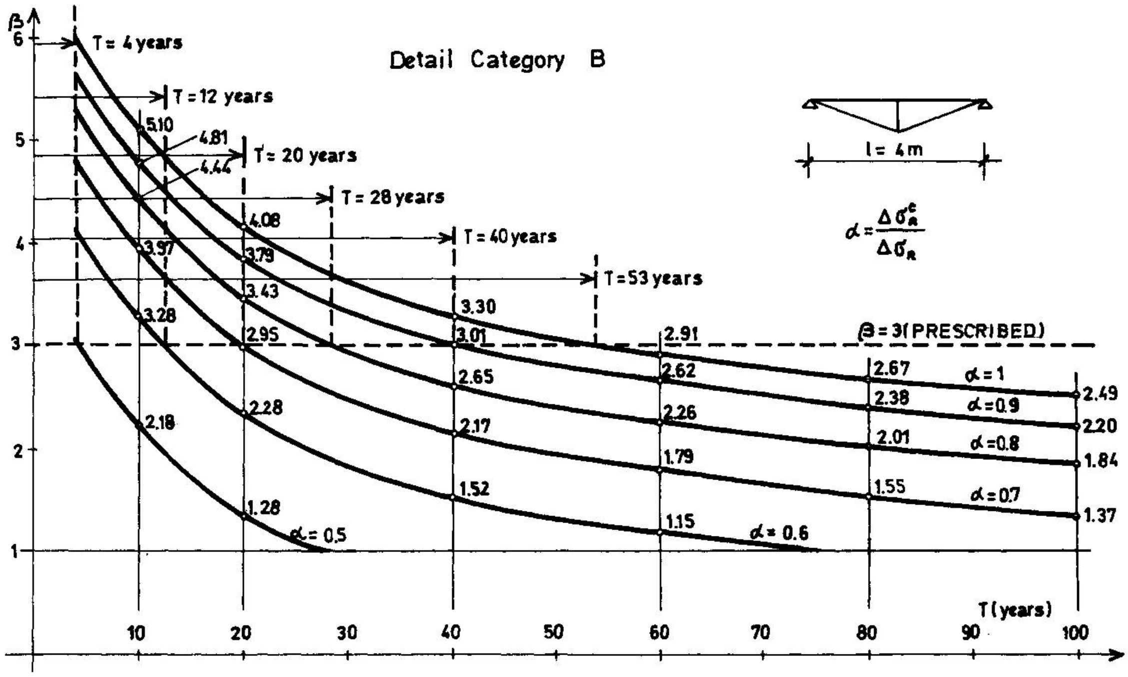

Fig.6 Durability decrease of detail B for various coefficients

The required safety index is $\beta \geq 3.5$ according to recommendations in [4] .Final adoption of safety index is the subject of further discussion.

5.DISCUSSION

The safety of steel structures exposed to corrosion fatigue shows a drastic drop of safety index β .The results of the analyses shown in Table 1 made on a practical example have shown that already after 10 and 50 years of service ,the safety index β can drop far below the recommended values $\beta \geq 3.5$ [4] .In this paper,the required value of safety index will be taken as $\beta = 3$ since there is a tendency to lower the safety index prescribed by ECCS recommendations in [4].In other words,preserving the required safety index,the service life is decreased.In the case of fatigue strength basic variable drop by 50% ($\alpha = 0.5$) [1] in the limit state equation,the service life is greatly decreased,as shown for detail B in Fig. 6.The problem is increased if correlation of basic variables $\Delta\sigma_R^c$ and m is considered.It has,namely,been found that slope regression line shows greater values at lower fatigue strength.

6.CONCLUSION

Corrosion fatigue greatly increases the probability of failure as compared to fatigue without the corrosion process,and the following facts should be noticed on the basis of phenomenological findings:

a) the structure should be designed so that corrosion protection can be regularly repeated and maintained,and every corrosion process should be prevented before it starts.

b) in cases where corrosion has been detected,and the structure has been in service for some time,service life is decreased,and the structure should be inspected more often than is usual for uncorroded structures exposed to fatigue.Obtained safety indices will provide a quantitative estimate of the risk level and compare it to the prescribed value.

The results should start a further discussion of safety problem at corrosion fatigue.The objective of such discussions is to reach decision for codification of the problem.

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Assessment of Existing Structures for their Rehabilitation

Evaluation de structures existantes en vue de leur assainissement

Beurteilung von Bauwerken und deren Instandsetzung

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SUMMARY

In situ non-destructive testing has become increasingly popular as a tool for assessment and re-evaluation, particularly in concrete structures. The main techniques of in-situ non-destructive testing are briefly analysed and their most common applications are listed. Their use is then illustrated with the analysis of two case studies.

RÉSUMÉ

Les essais non destructifs in situ son devenus de plus en plus populaires comme moyen d'analyse et d'évaluation spécialement dans les structures en béton. Les principales techniques de ce type d'essai sont analysées brièvement et décrites ci-après. Deux cas illustrent son application.

ZUSAMMENFASSUNG

Vorwiegend bei Stahlbetonbauwerken haben sich die in situ nicht zerstörenden Testmethoden als Mittel der Analyse und Beurteilung gut bewährt. Die wichtigsten Testverfahren dieser Art und deren häufigste Verwendung werden kurz beschrieben. Ihre Anwendung wird in zwei Beispielen aus der Praxis dargestellt.

1. INTRODUCTION

The increasing costs of building sites, the necessity of preserving the existing urban architectural image and natural or man-caused disasters are among the many factors contributing to the necessity of re-evaluating existing structures in order to widen its service life. There are many uncertainties connected with these studies: the nature of the ageing processes of the construction materials, its quality control in time, the way natural disasters affect the global behaviour of a structure, etc.. The analytical study of these problems is still under way and even the most up-dated state-of-the-art on this subject can not guarantee a reliable re-evaluation of existing structures relying only in mathematical models.

In this context, in-situ non-destructive testing (n. d. t.) is still one of the most effective tool for assessment and re-evaluation particularly in concrete structures. In this paper, the main techniques of in-situ n. d. t. are briefly analysed. Their use is then illustrated with the analysis of two case studies. One in which a fire of small proportions broke up inside a tunnel causing some severe localized damage. The second in which the in-service loads were to be increased in an existing structure for functional reasons.

2. THE NEED OF IN-SITU TESTING

Among other circumstances, in-situ testing is the best solution in the following cases [1]:

- If a structure shows signs of premature deterioration after some time in service - in particular, after having been subject to uncommon actions (earthquake, fire, explosion), the structure needs some analysing to make sure its reliability is still sufficient to fulfill the requirements of the codes. Structures in service for a long time may show signs of surface deterioration due to chemical attack, freeze-thaw cycles, alkali-silica reaction or other factors which may shed some doubts concerning its behaviour.
- When it is necessary to re-evaluate an existing structure - Very frequently, existing structures are asked to cope with in-service loads unforeseen in the original calculations. In-situ testing is the most effective way of knowing whether the structure is able to meet the new requirements without undergoing important changes and, if not, is a touchstone for a proper strengthening plan.

3. IN-SITU NON-DESTRUCTIVE TESTING

Very sophisticated new techniques of in-situ testing keep coming up but most of them are still very much laboratory bound. The most conclusive techniques for in-situ evaluation have been around for a while and have had a good deal of self-testing. They can be divided in two main groups: construction materials tests and structural tests.

The first group includes all the in situ tests in which the main result is some characteristic of the construction materials, usually a mechanical resistance. Among them, the most important are the ultrasonic pulse velocity test, the sclerometer test, the core test and the galvanic cell test.

The structural tests are the ones which study the structure as a whole regardless of the materials. The most well known are the full scale load tests and the dynamic tests.

Apart from the tests referred to, there is a high variety of others which will be listed in Table 1.

TEST	METHOD USED	MAIN RESULTS
Ultrasonic pulse velocity (concrete)	Determination of ultrasonic pulse propagation velocity	- Young modulus - Strength - Crack existance
X and Gama Rays (concrete and steel)	Radiographs	- Dimension and position of reinforcement - Cavities in concrete
Electrical methods (concrete)	Determination of overall capacitance or electrical resistivity	- Moisture content - Thickness of pavements
Dynamometer (steel)	Determination of magnetic permeability	- Reinforcement tension
Thermic methods (concrete)	Determination of maturity	- Strength
Galvanic cell (steel)	Measurement of an electrical potential	- Corrosion areas in reinforcement
Magnetometer (steel)	Measurement of a magnetic field	- Position of reinforcement
Acoustic methods (concrete)	Determination of the intensity of an acoustic emission	- Estimation of ultimate load - Crack existance
Sclerometer (concrete)	Determination of the surface hardness (rebound number)	- Strength
Penetration (concrete)	Determination of the resistance to the penetration of a probe	- Strength
Pull-out (concrete)	Determination of the pull-out resistance	- Strength
Cores (concrete)	Determination of the compression resistance	- Strength
Load Test (structure)	Determination of tensions and deflections	- Ultimate load - Behaviour under service conditions
Dynamic Test (structure)	Determination of the structure's response to dynamic excitation	- Dynamic characteristics

Table 1 [1] In-situ tests

4. RE-EVALUATION AND REPAIR OF A TUNNEL DAMAGED BY A FIRE

The tunnel inside which a fire broke consists of a reinforced concrete arch with a radius of 3 meters and a length of about 100 meters open at both ends. An accident next to one of the ends ignited the wooden formwork. Due to a chimney effect along the tunnel, the fire quickly propagated to all the wooden structure. The consequent high temperatures gave rise to significant deterioration of the gallery next to the formwork. The damage decreased in importance along the length of the tunnel.

Basically, four areas of different deterioration levels were defined:
Area A - no visible signs of deterioration.
Area B - the concrete cover had spalled in patches.
Area C - some of the reinforcement was visible but it was still adherent.
Area D - very high degree of concrete degradation, loose and twisted reinforcement (Fig. 1).

The visual inspection was complemented by an analysis of the materials strength.

Sclerometric tests were made along the total length of the gallery. They showed that superficial concrete in Area D (next to where the fire broke) had a lower residual resistance. To quantify more precisely the heat effect in depth, some cores were taken out and tested in compression (Fig. 2). The results varied between σ_{ck} = 30,6 MPa and σ_{ck} = 22,8 MPa in Area D for concrete not directly exposed to the fire. As all these values were superior to the one used in the initial design (σ_{ck} = 22,1MPa), it was considered that the existing concrete presented no safety problems for the structure.

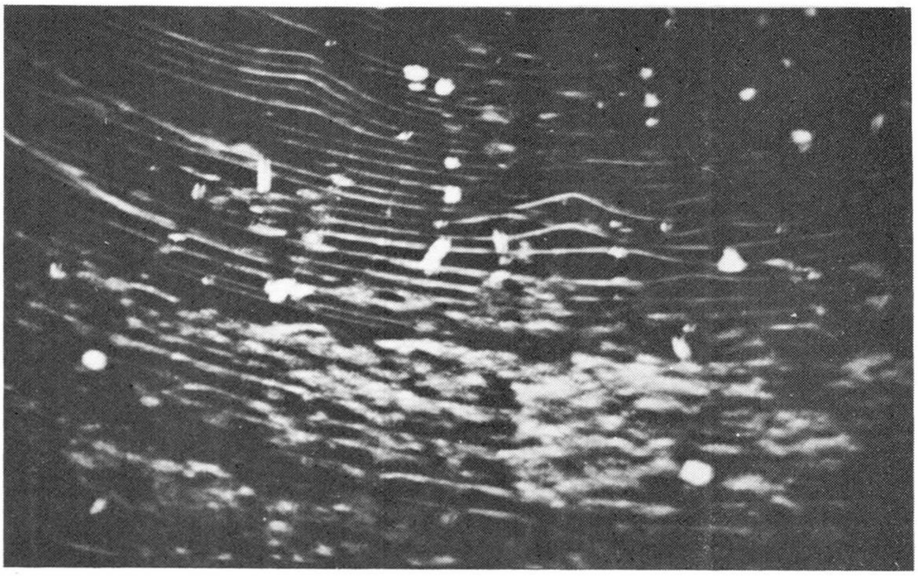

Fig. 1 **[2]** Damage in the cross-section of the tunnel (Area D)

Some steel cores from the various areas were also taken out and tested. Only the reinforcement in Area D showed some decrease in its mechanical characteristics ($f_{s0,2k}$ = 350 MPa < 400 MPa). As its residual extensions were substantial, the reinforcement was replaced. Adhesion tests were also performed before repair took place.

Taking into account these results, the following repair procedure was adopted:
Area A - the soot was removed.
Area B - the vault surface was pick-axed and subsquentely shotcreted in order to obtain the original cover.
Area C - the same as in Area B except that the concrete under visible reinforcement was also removed by pick-axing.
Area D - the reinforcement was replaced, about 3 cm of the surface concrete were removed and the cover was replaced by shotcreting. To efficiently verify the bond between old and new concrete, some cores were taken out and tested to quantify the bond strength of the joint. The ultimate shear stress was about 2,3 MPa, similar to that of 0,00the concrete used in the tunnel construction.

5. EXTENSION AND STRENGTHENING OF A BUILDING

The building occupies an area of 38.7 by 30.0 meters and its construction history is as follows:
- about 40 years ago, the ground floor and an intermediate floor consisting of a reinforced concrete slab resting on a beam grid were built up. The roof topwas a light steel structure.
- in 1961, a first extension of the building was concluded. It consisted of the replacement of the roof top by a pre-stressed small beams slab resting on reinforced concrete beams and of the building of a terrace with the same structural characteristics. These new pavements rested on columns on top of the initially built.

In neither case, was the structure conceived to withstand seismic actions and in both projects there was a remarkable deficiency in detailing namely concerning the reinforcement.

A second extension of the building was now needed in order to build a new floor. The in-service loads were also to be increased in all floors from the initial 3.0 KN/m^2 to up to 5.0 to 8.0 KN/m^2 . The seismic action according to the existing codes had to be considered in the new calculations.

The absence of proper reinforcement detailing for its characterization gave rise to the development of a scheme of structural inspection consisting of:
- groove making in concrete beams to determine the number and size of the reinforcement.
- non-destructive in-situ testing using X Rays to detect and identify the reinforcement in some slabs (Fig. 2). It proved to be efficient in the analysis of slabs up to 30 cm deep with the equipment used and the safety conditions imposed [3]. The radiographies showed the reinforcement layout and size.

According to the results obtained, it was not possible to guarantee analytically the safety factors in the dimensioning of the slabs with the new in-service loads. It was then decided to resort to a full scale load test of the slab with the least favourable conditions. The load consisted of water contained by a small wall specially built for the effect (Fig. 2).

The results showed a good behaviour of the slab with almost no visible cracking for the future in-service loads with a factor $\gamma_f = 1,25$. One of the main reasons for this was the good quality of the mortar layers used on top of the slabs that functioned monolitically with the concrete. There was then no need to strengthen the slab.

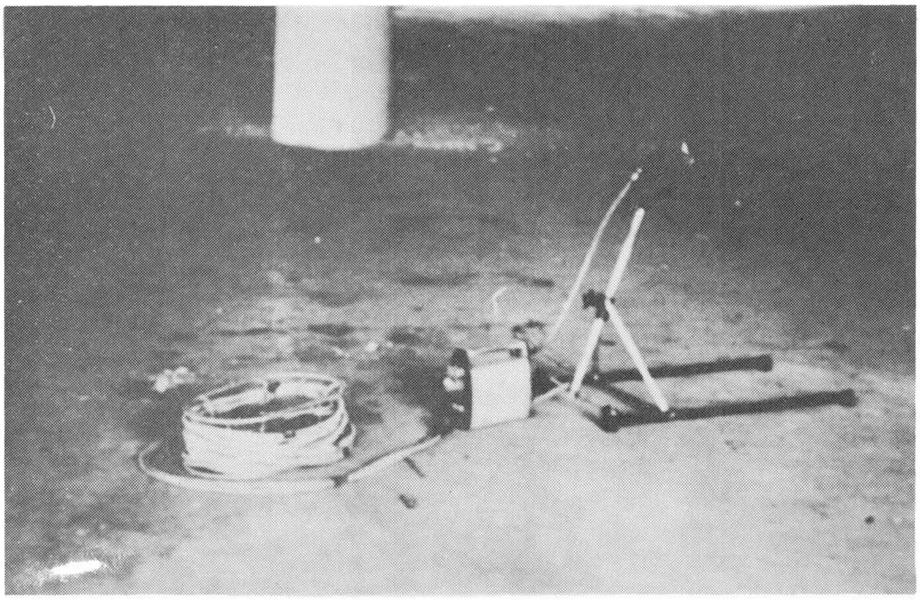

Fig. 2 X Ray equipment and full scale load test of slab

The remaining structure as a whole was strengthened to withstand a seismic action with a return period of a 1000 years according to the code. New shear walls and columns connected with the existing beams were built. Some of the beams were strengthened with new stirrups next to its bearings to guarantee an increased shear strength and improved ductility.

ACKNOWLEDGEMENTS

The authors wish to express their gratitude to GAPRES, LDA., the engineering consulting firm where these practical cases were studied. This paper is included in a research program on durability of structures undertaken by CMEST - Technical University of Lisboa

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Assessing Surface Properties of Concrete by In-situ Measurements

Détermination des propriétés du béton à l'aide d'essais in situ

Bestimmung der Oberflächeneigenschaften von Beton

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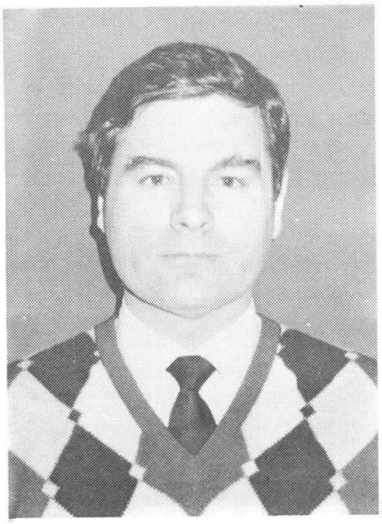

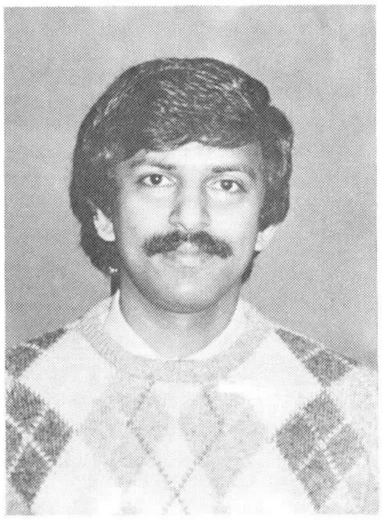

SUMMARY

This paper describes test methods which are employed for the assessment of surface strength, surface permeability and abrasion resistance of concrete on site. The successful application of these three methods in the laboratory and the details of the tests on site are described. By using them in combination, it should be possible to predict many of the problems related to durability of concrete structures before they reach serious proportions.

RÉSUMÉ

Cette communication décrit des méthodes d'essai employées pour l'évaluation de la résistance en surface, de la perméabilité et de la résistance à l'effritement superficiel du béton sur le chantier. Elle décrit l'application de ces trois méthodes en laboratoire ainsi que les modalités des essais sur chantier. En les combinant, il devrait être possible de prévoir de nombreux problèmes relatifs à la durabilité de structures en béton, avant que ceux-ci ne s'aggravent.

ZUSAMMENFASSUNG

Dieser Beitrag beschreibt Prüfverfahren, die zur Ermittlung der Oberflächenzugfestigkeit, der Wassereindringtiefe und des Abnutzwiderstandes von Beton am Bauwerk dienen. Die erfolgreiche Anwendung der drei Prüfverfahren im Labor und die Durchführung der Prüfungen am Bauwerk werden beschrieben. Die Kombination der drei Prüfverfahren soll das rechtzeitige Erkennen der durch äussere Einwirkungen verursachten Schäden am Beton ermöglichen, bevor diese grössere Ausmasse annehmen.

1. INTRODUCTION

As concrete is the most dominant structural material in many parts of the world, its strength and ability to protect reinforcement against corrosion has been the subject of much investigation. Research on durability of concrete has highlighted the importance of producing concrete with the ability to resist the ingress of deleterious agents from the surrounding environments. When testing and evaluating the quality of concrete structural elements, it is often presumed that the concrete within one member is uniform in composition and properties. However, during placing and compacting of the fresh concrete, the surface layer often contains more cement and water than the core concrete. Hence, the composition and properties of the surface layer of the concrete differ from those of the core. Furthermore, during the service life of the structure, the surface layer is usually subjected to more severe environmental attacks than the core concrete. The customary quality control for concrete using compressive strength tests on standard cubes and cylinders, or cores cut from the structure, do not measure the properties of the surface layer, instead they evaluate the properties of the mass. Hence, tests are required to evaluate the quality of the surface concrete, especially on site. In recent years, a wide range of methods have become available for estimating the strength of the in-situ concrete, which include the non-destructive methods and the more recently introduced partially destructive tests. The pull-off test using the LIMPET [1] comes under the latter category of tests which allows the compressive strength of the surface layer to be determined.

It has been recognised over the past few decades that concrete is deteriorating not because of its low strength, but because of its other properties. When concrete is exposed to aggressive environments, its successful performance depends to a greater extent on its durability against the environment than its strength. Permeability has been widely accepted as a measure of the durability of concrete [2,3]. The permeability of the surface layer largely determines the vulnerability of concrete to external agencies, so that in order to be durable, the surface layer must be relatively impervious. The CLAM permeability test [4] enables the coefficient of permeability, as used in Darcy's equation to be determined for the surface layer of concrete.

Even though much work has been carried out in the general area of concrete durability, comparatively little research has been undertaken in one aspect of the subject, viz. the abrasion resistance. Isolated researches [5,6] have been undergoing in the past few years on the assessment of abrasion resistance and yet, there is no universal standard test method for the same. Hence, a research team at the Queen's University of Belfast developed the TEREDO [7] for the insitu use. The viability of the three apparatus has been established by means of extensive experimental work in the laboratory and on site.

2. ASSESSMENT OF SURFACE STRENGTH USING LIMPET

2.1 Procedure and Test Details

The pull-off test using the LIMPET [1] involves bonding a circular steel probe to the specimen under test by means of an epoxy resin adhesive. A slowly increasing tensile force is then applied to the probe until the specimen fails in tension. As the amount of overbreak is usually small, the area of failure can be taken as being equal to the area of the probe. This permits the calculation of the nominal tensile strength of the specimen from which the compressive strength of the surface layer can be estimated by the use of appropriate calibration graphs. The LIMPET is shown in Figure 1.

A correlation curve based on a large number of pull-off tests and corresponding cube compressive strengths is shown in Figure 2. The closeness of the fit between the tensile strength and the corresponding cube strength is indicated by the confidence limit of 95%. It was found from a series of laboratory tests that the test is sensitive to compaction and curing conditions which affect the strength.

2.2. Insitu Tests on Multi-storey Car Park

The accuracy and the reliability of the LIMPET test as a means of quality control was assessed [8] during the construction of a multi-storey car park for the Department of Environment (NI) in Ballymena. The structure involved large number of similar pours which offered an excellent opportunity to monitor the levels of accuracy and the reliability attained by the pull-off method on insitu concrete. The construction was that of insitu beams, slabs and columns resting on pad foundation.

The tests were performed on three different types of members (viz. columns, parapet walls and stair block walls) at both 7 and 28 days. The results of this investigation are summarised in Table 1 which indicates the closeness of the predicted values with the corresponding cube compressive strength. It was observed that there was an increase of strength of columns towards the bottom. The average strength of concrete at the top was 8% lower than that of concrete in the middle of the columns and 12% lower than that at the bottom.It was also noted that concrete at the top of columns was significantly more variable than that at the bottom.

Hence, the pull-off test using the LIMPET was found to be quite satisfactory as an insitu method and problems of variability of concrete can be investigated using it.

3. ASSESSMENT OF SURFACE PERMEABILITY USING CLAM

3.1 Procedure and Test Details

The CLAM (Figure 3) is a compact piece of equipment which is designed to measure the permeability of the surface layer without disturbing the concrete and over an area sufficiently large to rule out the exaggerated aggregate effects. The method involves bonding a steel ring of internal diameter 50mm on to the specimen under test. The body of the apparatus is bolted to this ring and sealed with an O-ring. Water is admitted into the pressure chamber by withdrawing the piston such that it rests above the level of the inlet nozzle. A syringe containing water is connected to this nozzle and the bleed valve is opened to allow air to escape. The pressure inside the chamber is increased to 25 psi using the syringe and is maintained by advancing the piston manually in to the pressure cylinder. A micrometer attached to the piston measures the travel from which the rate of flow into the specimen is determined. This in turn may be converted into coefficient of permeability, K, as used in Darcy's equation, in units of m/s, by multiplying with appropriate calibration factors which have been developed from a theoretical study of flow nets. The CLAM was also found to be useful in estimating the initial surface absorption of concrete.

Detailed laboratory investigation in the past five years or so indicated that the CLAM test is sensitive to the variations in properties of concrete as a result of the variation in mix parameters, age or curing. A modest change in water/cement ratio was found to have relatively little effect on the strength whereas the resulting surface permeability significantly differed.

3.2. Insitu Test Programme using CLAM

The viability of the CLAM as an insitu test device was assessed [9] on a multi-storey car park in Belfast. The tests for CLAM permeability were planned in the same way as that of the pull-off test which was explained earlier and the results were recorded at 7 and 28 days. The average permeability values from this comprehensive test are given in Table 2.

The variation in permeability within columns at different heights was investigated and the results are presented in Table 3. The variation in permeability was found to have followed an inverse relationship with that obtained from the pull-off results. There was a decrease of permeability from top to bottom of column height.
Both the laboratory and insitu test results confirm that the CLAM is a useful tool for measuring the surface permeability of concrete.

4. ASSESSMENT OF SURFACE ABRASION BY TEREDO

4.1 Procedure and Test Details

The TEREDO [7] is based on the principle of accelerated abrasion using rotating wheels and consists of three sets of spiked steel dressing wheels which are mounted on a central spindle (Figure 4). This is rotated at a constant speed of 50 rpm by means of a small DC electric motor with a variable speed control. The central shaft is attached to a hollow ram jack and hence, a force applied on to the spindle through the hollow ram jack enables the rotating wheels to abrade the surface. The machine is so designed that the dressing wheels will follow the contours of the test surface, including any peaks and troughs formed by its abrading action. The depths to which the test surface has been abraded is recorded at 8 points along the abraded annular path after 15 minutes of operation. This is used as a measure of the abrasion resistance of the surface. The machine is bolted on to the test specimens for laboratory tests. However, expanding bolts are used for insitu tests.

4.2. Early Results of the Laboratory Investigation

The investigation into the sensitivity of the apparatus to the concrete mix variables indicated that the TEREDO yields results which may be related to the strength of the concrete. The influence of water/cement ratio, curing regimes, and the method of surface finish on the abrasion resistance are given in Figure 5. The effect of water/cement ratio on the strength of concrete is also indicated for correlation purposes.

5. ASSESSMENT OF STRENGTH AND DURABILITY

The LIMPET, CALM, and the TEREDO have been successfully used in a study on the surface durability of ferrocement and the results are reported else where [10]. It was observed that where the LIMPET gave low values of strength, the CLAM and the TEREDO yielded high values of permeability and abrasion respectively. Hence, by using these three tests in combination, it should be possible to eliminate many of the current strength and durability problems which occur in concrete structures.

6. CONCLUSIONS

On the basis of the extensive experience which has been gained at the Queen's University over the past few years, the following concluding remarks can be made.

1. The LIMPET test for the determination of insitu surface strength of concrete is quick, accurate and gives results within reasonable confidence limits.

2. The CLAM permeability test enables the assessment of the durability of concrete as it measures the surface permeability of concrete. The test is quick and reliable.

3. The TEREDO is portable and allows easy measure of the surface abrasion of concrete floors.

4. As the three apparatus are designed for site use, it should be possible to reduce some of the strength and durability problems in concrete structures in future by employing them in combination.

7. ACKNOWLEDGEMENTS

The contribution made to the development of the LIMPET, CLAM, and TEREDO by Alastair Thompson, Jim Newell, Andrew Murray, Alison Adams and David Lau is gratefully acknowledged.

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Type of Member	Age (Days)	Size of Aggregate	Average Cube str.	Average LIMPET str	Cube str.x100 / LIMPET str.
Column	7	20mm	$22.6N/mm^2$	$23.6N/mm^2$	96%
	28	20mm	$39.7N/mm^2$	$38.1N/mm^2$	104%
	28	10mm	$42.7N/mm^2$	$40.4N/mm^2$	106%
Wall	7	20mm	$20.8N/mm^2$	$21.3N/mm^2$	95%
	28	20mm	$38.3N/mm^2$	$35.5N/mm^2$	108%

Table 1 Summary of Results from Insitu Tests Using LIMPET

Type of Member	Age (Days)	Average Coefficient of Permeability $K(x10^{-13})$
Column	7 28	2.96 1.07
Wall	7 28	3.13 3.10
Beam	7 28	2.28 1.74

Table 2 Summary of Results from Insitu Tests Using CLAM

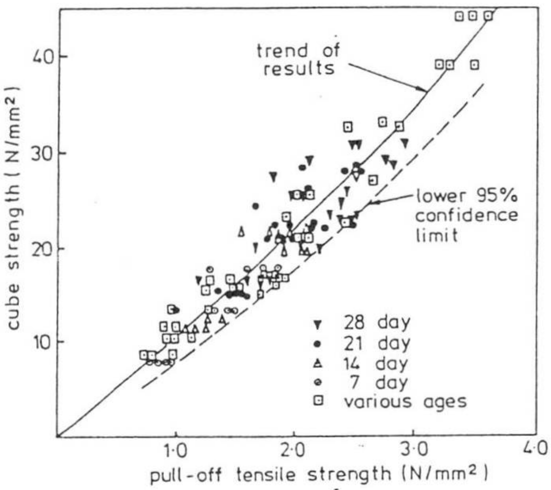

Figure 2 Typical Correlation Curve for LIMPET for concrete of Different Ages

Height up Column	Average Coefficient of Permeability $K(x10^{-13} m/s)$
1800mm	1.52
1470mm	0.91
815mm	0.87
280mm	0.68

Table 3 Variations of Permeability Within Column

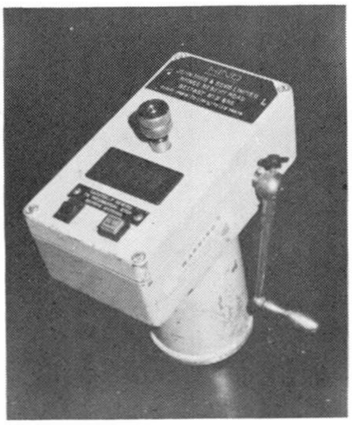

Figure 1 The LIMPET

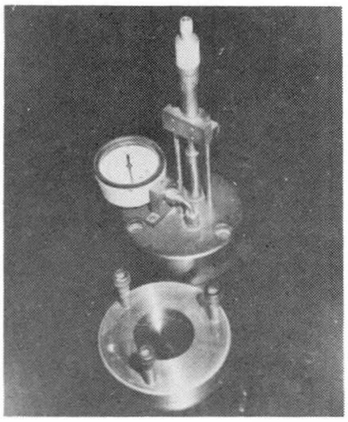

Figure 3 The CLAM

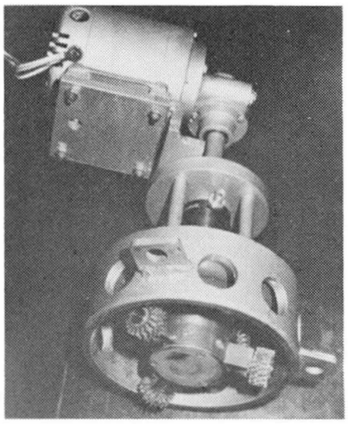

Figure 4 The TEREDO

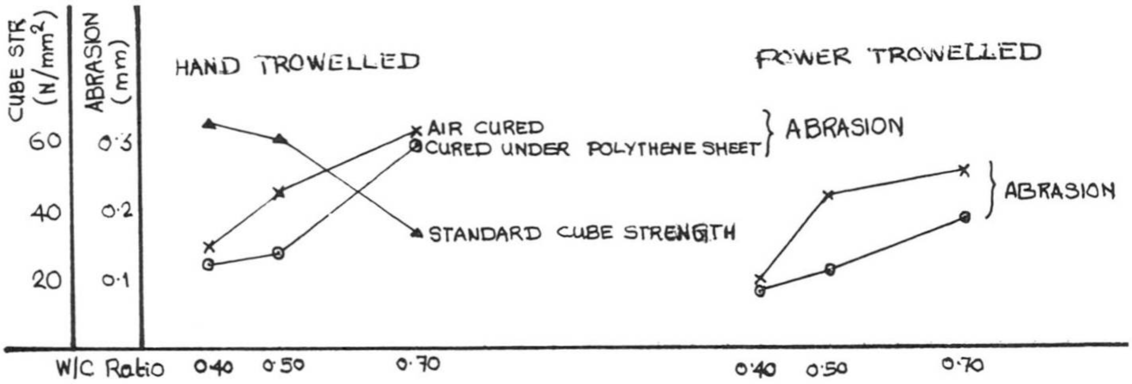

Figure 5 Results of TEREDO Abrasion Test

Zustandsuntersuchungen im Strassentunnel San Bernardino

Investigation of Conditions in the San Bernardino Highway Tunnel

Analyse de l'état du tunnel routier du San Bernardino

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Marc Ladner, geboren 1939, erwarb 1962 das Diplom als Bauingenieur an der Technischen Hochschule in Zürich (ETH), wo er 1968 auch zum Dr. sc. techn. promovierte. Nach einem Studienaufenthalt an der University of Texas at Austion TX, USA, leitet er seit 1971 die Abteilung Massivbau der Eidg. Materialprüfungs- und Forschungsanstalt (EMPA) in Dübendorf.

ZUSAMMENFASSUNG

Nach etwa zwanzigjähriger Betriebszeit müssen im Stassentunnel San Bernardino Instandsetzungsarbeiten durchgeführt werden. Um das Schadensausmass und die klimatischen Bedingungen im Tunnel genau zu kennen, sind vorgängig umfangreiche Zustandsuntersuchungen und Messungen vorgenommen worden, über deren Ergebnisse auszugsweise berichtet wird.

SUMMARY

After a service life of some twenty years the highway tunnel San Bernardino has to be repaired. In order to know more about the existing dimensions of damage as well as the climatic conditions in the tunnel, comprehensive investigations and measurements have been carried out during the two years preceding the repair work. The paper deals with some selected results of these investigations.

RÉSUMÉ

Après une période de service d'environ vingt ans, le tunnel routier du San Bernardino doit être réparé. La grandeur des dégâts et les conditions climatiques dans le tunnel ont été étudiés soigneusement pendant plus de deux ans avant de commencer les réparations. L'article résume quelques résultats intéressants de ces études.

1. DAS BAUWERK

Kernstück einer der wichtigen Alpentransversalen im Schweizerischen Nationalstrassennetz bildet der San Bernardino Strassentunnel. Dieses 6.6 km lange Bauwerk führt je eine Spur der Nationalstrasse N13 in nord-südlicher bzw. südnördlicher Richtung durch die östlichen Schweizer Alpen und verbindet so das Hinterrheintal mit dem nach Süden geöffneten Misox. Im Querschnitt (Fig. 1) ist der einröhrige Tunnel in fünf Räume unterteilt, wobei der grösste Teil vom Fahrraum eingenommen wird. Unter dem Fahrraum befinden sich drei Kanäle; durch den mittleren wird die Zuluft zugeführt, während sich im einen der beiden äusseren ein Abwasserrohr und im anderen verschiedene Kabel befinden. Diese drei Kanäle sind unter einander durch je eine Trennwand getrennt, die Abgrenzung nach oben erfolgt jedoch durch die Fahrbahnplatte. Somit bewegt sich der Verkehr auf einer brückenartigen Konstruktion. Dabei ist die Fahrbahnplatte einerseits auf den Trennwänden, andererseits aber auf den seitlichen Auflagern im Gewölbe abgestützt.

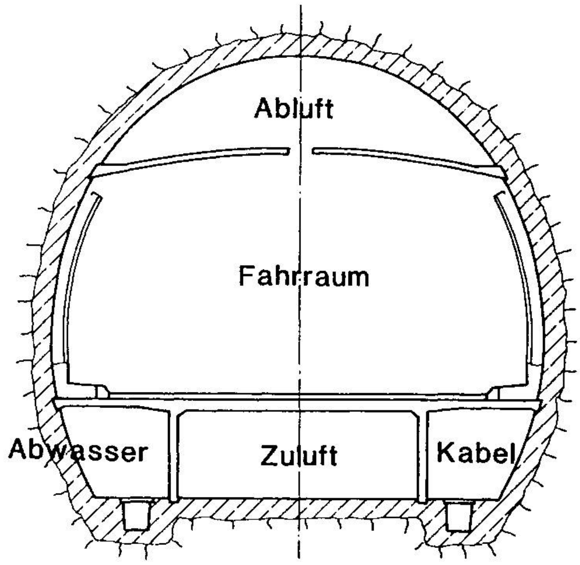

Fig. 1: Querschnitt durch Tunnel

Um sich den in grossen Grenzen schwankenden Temperaturen anpassen zu können, ist die Fahrbahnplatte quer zur Tunnelachse alle 25 m durch eine vollständig durchgehende Dilatationsfuge getrennt; die derart entstehenden Teilabschnitte sind ihrerseits wiederum alle 2.5 m durch Scheinfugen unterteilt, die ebenfalls senkrecht zur Tunnelachse verlaufen.

Die Zuluft wird nicht nur von den beiden Portalen aus, sondern ebenso auch durch zwei etwa in den Drittelspunkten, bezogen auf die Tunnellängsachse, angeordnete Kamine eingeblasen, so dass sehr unterschiedlich erwärmte Luftmassen in den Tunnel einfliessen. Da ausserdem im Winter infolge des sehr intensiven Verkehrs von der als wintersicher unterhaltenen Nationalstrasse grosse Mengen Tausalz in den Tunnel eingeschleppt werden, sind sämtliche Bauwerksteile und Einrichtungen im Tunnel einer extrem hohen Chloridbeaufschlagung ausgesetzt. Insbesondere die Betonfahrbahnplatte, und dort wiederum die Zonen in der Umgebung der Haupt- und Scheinfugen, sind somit nicht nur mechanisch, sondern auch chemisch hoch beansprucht.

Nach rund zwanzigjährigem Betrieb des Tunnels haben sich die Mängel und Schäden nun derart verstärkt, dass eine Instandsetzung unumgänglich geworden ist. Um sich aber vorerst einmal ein genaues Bild über das Ausmass dieser Schäden im jetzigen Zeitpunkt machen zu können, und um andererseits auch die genauen klimatischen Verhältnisse sowie die dadurch verursachten Bewegungen der Haupt- und Scheinfugen kennen zu lernen, wurden im Verlauf der letzten zwei Jahre umfangreiche Zustandsuntersuchungen an allen wichtigen Bau- und Einrichtungsteilen sowie auch entsprechende Messungen von Temperatur, Luftfeuchtigkeit und Fugenbewegungen vorgenommen, so dass für die bevorstehende Instandsetzung genügend abgesicherte Grundlagen zur Verfügung stehen, um die Anforderungen, die an die anzuwendenden Materialien zu stellen sind, formulieren zu können. Auf dieser Basis hofft man, eine Qualitätssteigerung bei den Instandsetzugsabeiten zu erzielen.

2. UNTERSUCHUNGEN UND MESSUNGEN

Sämtliche wesentliche Teile, die für die Betriebssicherheit des Tunnels von Bedeutung sind, wurden in die Zustandsuntersuchungen miteinbezogen. Insbesondere wurden in der Fahrbahnplatte zur Feststellung von Bewehrungskorrosionen Potentialfeldmessungen durchgeführt; die Bewegungen der Haupt- und Scheinfugen wurden in Abhängigkeit der herrschenden Temperatur- und Feuchtigkeitsverhältnisse ebenfalls gemessen und registriert. So ergab sich ein in vier Teile gegliedertes Untersuchungsprogramm, das die folgenden Elemente erfasste:

Stahlbetonbauteile:	Fahrbahnplatte inkl. Potentialfeldmessungen Sohlenbeton Tunneldecke Verkleidungsplatten Betonring
Haupt- und Scheinfugen:	Fugenbewegungen Dübel
Befestigungen, Einbauten:	Halterung der Wandplatten Deckenaufhängungen Lampen, Signale usw.

Als Beispiel ist in Fig. 2 die Anordnung einiger Messfühler über den Querschnitt bei einer Hauptfuge (HF 2915) und bei einer Scheinfuge (SF 2920) dargestellt. Da die beiden Fugen im Tunnel nur 12.5 m auseinander lagen, beschränkte man sich bei der Temperaturmessung auf einen Querschnitt. Für die Fugenbewegungsmessungen wurden auf der Fahrbahnplattenunterseite induktive Weggeber montiert, zur Erfassung der Betontemperaturen wurden Temperaturfühler sowohl auf der Unterseite der Fahrbahnplatte als auch unmittelbar unter dem Fahrbahnbelag angebracht; dazu musste von unten her ein Loch in die Fahrbahnplatte gebohrt werden. Die Lufttemperatur- und -feuchtigkeitsfühler befanden sich ebenfalls jeweils in der Nähe der zugehörigen Messstellen am Beton. Da sämtliche Messdaten quasi-kontinuierlich erfasst worden sind, stehen somit Messdaten über eine Zeitperiode von etwas mehr als zwei Jahre zur Verfügung und umfassen insbesondere drei Winter- und zwei Sommerperioden.

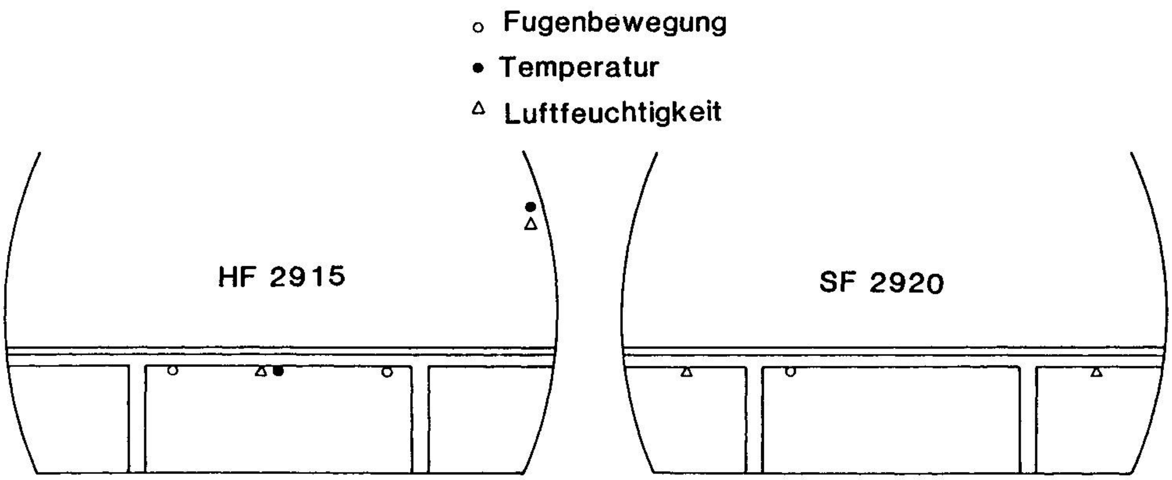

Fig. 2: Anordnung ausgewählter Messfühler über Querschnitt bei Hauptfuge 2915 und bei Scheinfuge 2920

4. ERGEBNISSE

Die Messungen der Fugenbewegungen und der Temperaturen wurden im Winter 1985/86 begonnen und zogen sich insgesamt über etwas mehr als zwei Jahre hin; die Messfühler für die Bestimmung der Luftfeuchtigkeit wurden allerdings erst gegen Ende 1986 versetzt. Fig. 3 zeigt als Beispiel dieser Messungen den Verlauf der Lufttemperaturen im Fahrraum und im Zuluftkanal über diesen Zeitraum sowie die zugehörige Bewegung der Hauptfuge im Bereich des Zuluftkanales auf der Plattenunterseite.

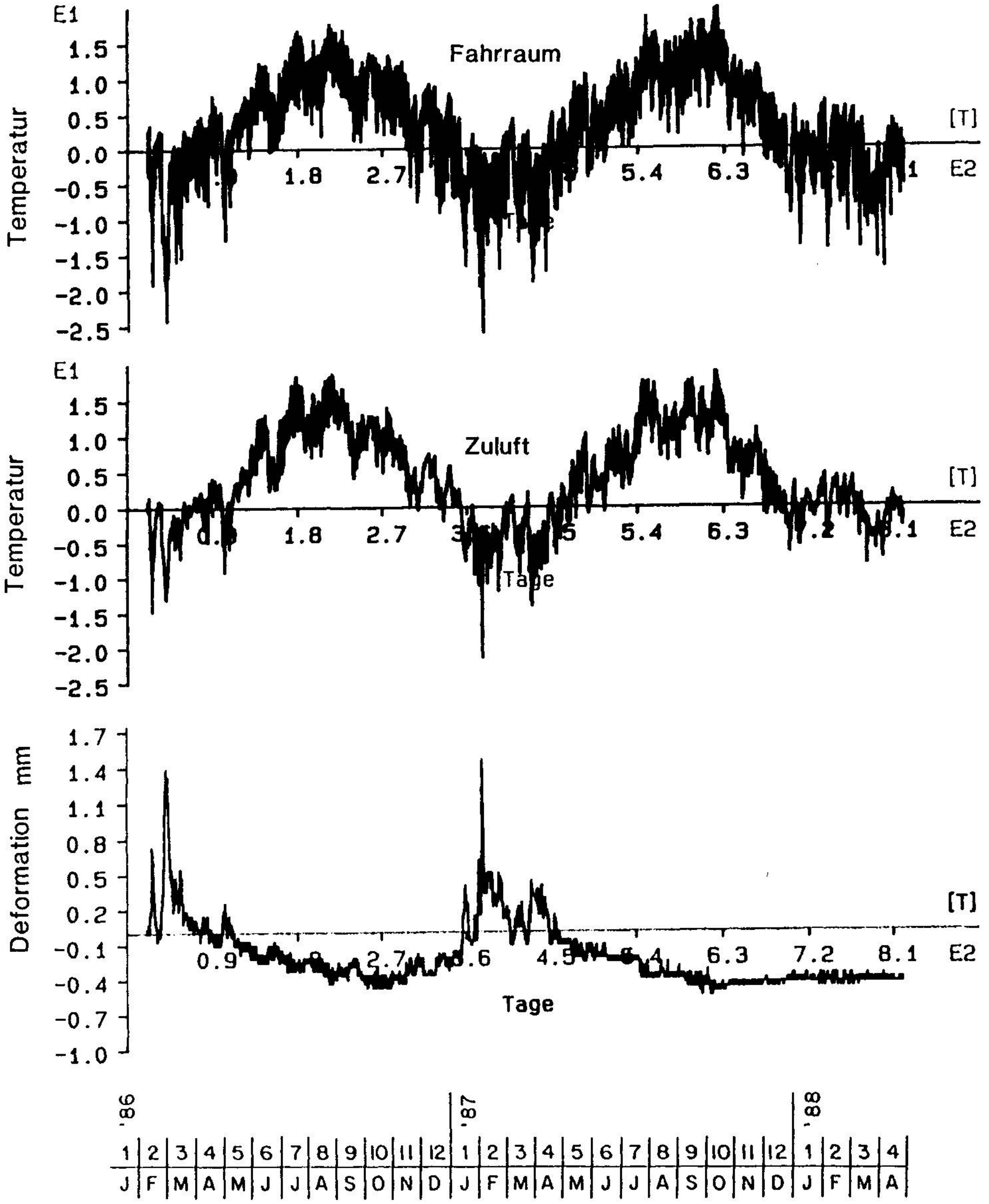

Fig. 3: Verlauf der Lufttemperatur im Fahrraum und im Zuluftkanal im Bereich der Hauptfuge 2915 sowie der zugehörigen Fugenbewegung

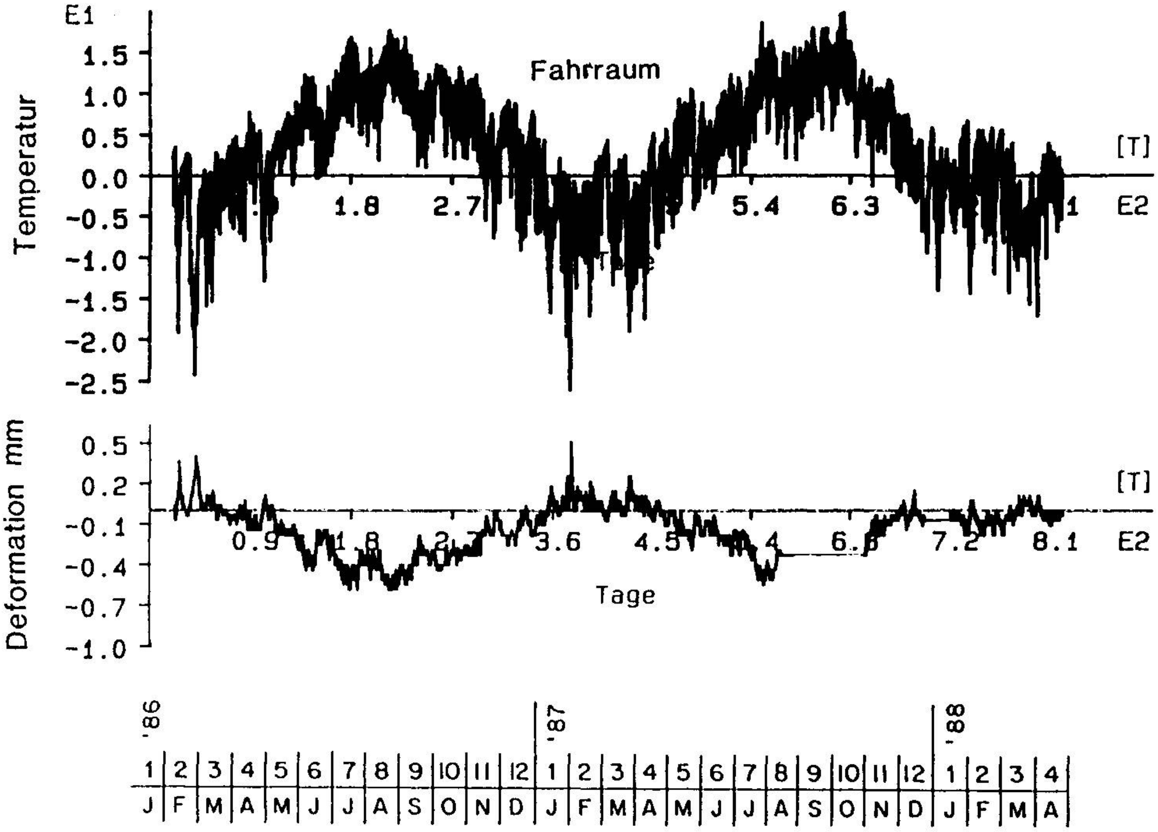

Fig. 4: Verlauf der Lufttemperatur und der Fugenbewegung bei Scheinfuge 2920

Zunächst lässt sich feststellen, dass die Fugenbewegungen einen doppel U-förmigen Verlauf aufweisen. Da mit den Messungen im Winter begonnen wurde, konnte in den meisten Fällen zu Beginn der Messperiode ein Öffnen der Fuge beobachtet werden. Die Grösse der relativen Fugenöffnung lag bei den Hauptfugen in der Grössenordnung von 1.5 mm.

Betrachtet man hingegen die grössten Fugenbewegungen, die sich zwischen den Jahreszeiten (Winter/Sommer) ergeben, dann betragen diese Werte für die Hauptfugen etwa 2.2 mm, bei den Scheinfugen jedoch nur etwa die Hälfte. Mit kurzzeitigen und sehr häufig wiederkehrenden Fugenbewegungen in der Grössenordnung von ±0.1 mm muss unabhängig von der jeweils herrschenden Temperatur zu allen Jahreszeiten gerechnet werden, vereinzelte Spitzenwerte von bis 1.2 mm bei den Hauptfugen bzw. von bis 0.5 mm bei den Scheinfugen können ebenfalls jederzeit auftreten.

Zum Zustand der Fahrbahnplatte im Bereich der Fugen haben die Untersuchungen ergeben, dass kein Zusammenhang zwischen der Karbonatisierungstiefe und der Korrosion der Bewehrung besteht. Ausserdem beschränken sich die Schäden im Beton und an der Bewehrung praktisch vollständig auf den Fugenbereich; Schäden in Plattenmitte konnten beinahe keine beobachtet werden. Zudem sind auch bei den Fugen die grössten Schäden bei den Hauptfugen zu erkennen, die ausnahmslos gerissen sind und grössere Fugenöffnungen aufweisen als die Scheinfugen, die sehr oft nur Fugenöffnungen von Haarrissbreite zeigen. Eine weitere interessante Feststellung konnte schliesslich noch in materialtechnischer Hinsicht gemacht werden, indem sich kein erkennbarer Zusammenhang zwischen der Betonqualität (Festigkeit) und dem gemessenen Chloridgehalt des Betons erkennen liess. Zudem sind Korrosionsschäden an der Bewehrung nur auf der Plattenunterseite gefunden worden, obwohl auch auf der Plattenoberseite im Fugenbereich Chloridgehalte von 1% bis 3%, bezogen auf die Zementmasse, gemessen worden sind!

Schliesslich soll auch noch auf die gute Korrelation zwischen den beobachteten Schäden und den Ergebnissen der Potentialfeldmessungen hingewiesen werden. Hier kann gesagt werden, dass im oberflächlich trockenen Zustand die -350 mV Isopotentiallinie (Potential vs. $CuSO_4$) alle Korrosionsprozesse umfasst. Es werden damit auch Zonen erfasst, in denen die Korrosion eben erst begonnen hat und die noch keine sichtbaren Schäden aufweisen. Hingegen ist immer dann mit starken Schäden wie etwa Flächenabtrag oder gar lokale Korrosionen zu rechnen, wenn sich die Bewehrung in einer Zone befindet, die Potentiale unter -450 mV, bzw. solche aufweisen, die um etwa 250 mV negativer liegen als der durchschnittliche Potentialwert der intakten Zone.

Die häufigsten Schadensformen und -ursachen der Fugenbereiche sind abschliessend nochmals in Tabelle 1 zusammengefasst. Es zeigt sich, dass in erster Linie das Eindringen der Chloride bei den Undichtheiten sowie die grossen Bewegungen infolge der Temperaturschwankungen für die Schäden in diesen Zonen verantwortlich gemacht werden muss.

Bauteil oder Bauelement	Häufigste Schadensform Befund	Hauptschadensursache
Hauptfugen	Korrosionserscheinungen Ausblühungen, Rissbildungen	Eindringen von Chloriden infolge Undichtheiten, grosse Rissbewegungen infolge Temperaturänderungen
Scheinfugen	Schäden nur bei etwa 40% aller Fugen, Erscheinungsbild jedoch grundsätzlich ähnlich jenem der Hauptfugen	Gleiche Ursachen wie bei Hauptfugen, obwohl kleinere Bewegungen der Fugen

Tabelle 1: Häufigste Schadensformen und -ursachen im Bereich der Fugen

5. VERDANKUNG

Der Verfasser möchte an dieser Stelle den Verantwortlichen des Tiefbauamtes des Kantons Graubünden, insbesondere Herrn P. Mantovani, ganz herzlich dafür danken, dass sie ihm die Veröffentlichung der Ergebnisse gestattet haben.

Load Testing as an Assessment Method
Essais de charge comme méthode de jugement
Probebelastung als Beurteilungsmethode

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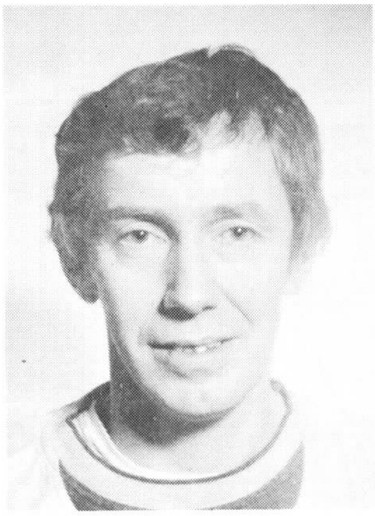
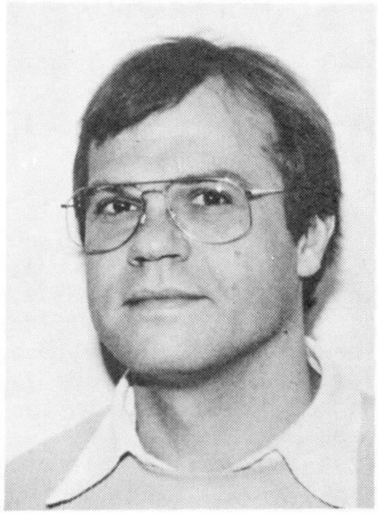
Arne Jensen Mondrup, born 1944, obtained his M. Sc. Degree at the Technical University in Copenhagen. He has specialized in bridge rehabilitation, maintenance and inspections. Arne Jensen Mondrup has diploma from the Danish Concrete Institute and Nitro Nobel, Sweden, in the durability of concrete.



Jens Ole Frederiksen, born 1950, obtained his M. Sc. Degree at the Technical University in Copenhagen. For twelve years he has been involved in experiments on building structures in the laboratory and "in the field".



Hans Henrik Christensen, born 1952, graduated in 1977 in structural and civil engineering at the Technical University in Copenhagen. 11 years of experience in design, repair, rating and load testing of bridges.

SUMMARY

Repeated load testing of a heavily deteriorated structure is an efficient method to evaluate whether the structure still meets the specified requirements with regard to load capacity. Two load tests of a motorway bridge showed that this structure could meet the requirements for load capacity 10 years after the bridge would, according to the traditional method of evaluation, have to be replaced. Generally, a load test is an economically favourable solution compared with the interest paid for a new investment.

RÉSUMÉ

Des essais de charge répétés sur une structure fortement endommagée représentent une méthode efficace pour juger de la capacité portante. Deux essais de charge sur un pont autoroutier ont démontré que sa durée de vie dépasse de dix ans celle donnée par des méthodes d'évaluation traditionelles. En général, un essai de charge est une méthode économique face aux dépenses d'un nouvel investissment.

ZUSAMMENFASSUNG

Die wiederholte Belastung von stark geschädigten Bauwerken ist eine effiziente Methode zur Ueberprüfung der vorhandenen Tragfähigkeit. Zwei Belastungsversuche an einer Autobahnbrücke zeigten, dass diese eine 10 Jahre höhere Lebensdauer aufwies als mit traditionellen Beurteilungsmethoden ermittelt worden wäre. Allgemein sind Belastungsversuche eine wirtschaftliche Methode, verglichen mit den Zinsbelastungen durch verfrühte Neuinvestitionen.

1. INTRODUCTION

Repeated full scale load tests can be used to verify the load carrying capacity of concrete structures, and the replacement of a damaged structure can therefore be postponed for many years. The method is particularly useful when inspections indicate a degree of deterioration that makes it difficult to calculate the load carrying capacity on the basis of material strength or a structural model similar to the deteriorated structure.

The load test of a motorway bridge at Skovdiget north of Copenhagen will be presented as an example of a load test of a deteriorated concrete structure. The twin motorway bridges (Eastern and Western respectively) were constructed in 1965-67 for the Copenhagen Highway Authorities. The bridges now belong under the Danish Road Directorate. The first load test was performed in 1984 and the second in 1988.

2. STRUCTURE

Each bridge is approx. 220 m long and approx. 22 m wide. The concrete superstructures consist of two main girders joined with transverse ribs at intervals of approx. 2 m, with cantilever wings on the outsides. The average length of the spans is approx. 20 m and at the ends approx. 10 m. The span across the S-train tracks is approx. 24 m. Each main girder, continuous over 12 spans, is supported by 11 Franki-pile founded columns.

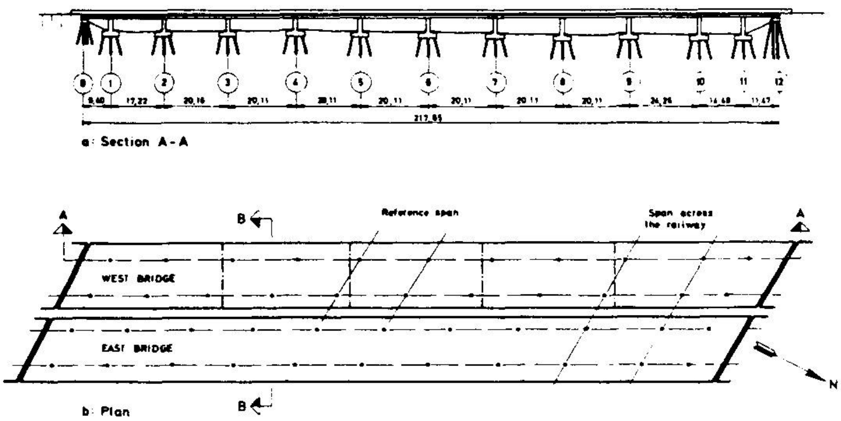


Fig. 1 Elevation and plan view

Each bridge was cast in five sections and prestressed and posttensioned longitudinally and transversely using the BBRV system.

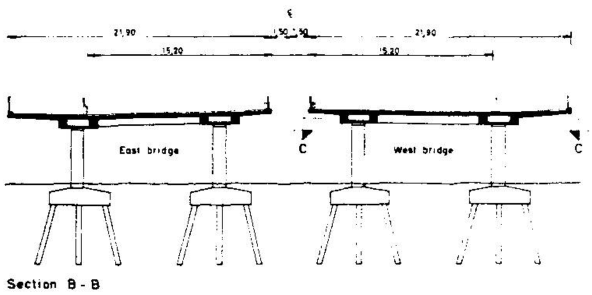


Fig. 2 Cross section

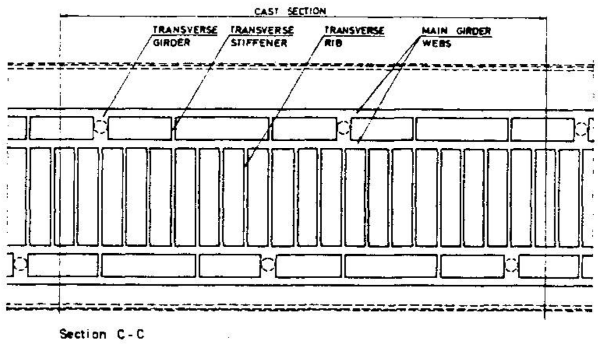


Fig. 3 Sectional plan view

The main girders are constructed as box sections with a cross sectional height of approx. 1 m. Transverse stiffeners 1.4 m wide are cast in the main girders over the columns as transverse girders with short cables arranged as a reversed U. The structure of the bridges is shown in figures 1-3.

3. HISTORICAL BACKGROUND

3.1 The Eastern Bridge

In 1976, the normal repair work at the Eastern bridge began. The main elements of the repair work were replacement of the old insulation and surfacing, including a new concrete surface, and an overall improvement of the dewatering system.

As the repairs progressed, however, a number of other damages were discovered that had to be remedied. Surface water, which had penetrated the insulation and was retained in cavities, had percolated into the concrete. Further, the concrete proved liable to alkali-siliceous reaction, and in many parts on the top side of the bridge deck, especially near drains, had deteriorated to a degree that necessitated replacement.

The prestressing cables were heavily corroded and sometimes uninjected. Consequently, the work was extended to include a relatively extensive examination and repair of the prestressing cables by reinjection. Other damages were discovered, and the complete repair work on the Eastern bridge lasted until 1984 and cost approx. DKK 25 million.

3.2 The Western Bridge

Preliminary studies had indicated that the same deficiencies would be found in the Western bridge. Consequently, the Danish Road Directorate initiated a study in 1982 to find alternatives to a costly overall repair. The outcome was among other things that the construction of a new bridge would cost approx. DKK 50 million, including the demolition of the old one.

4. DECISION ON LOAD TEST

On the basis of the above it was decided that the Western bridge should remain for the time being, provided the load capacity was sufficient to meet a given minimum load requirement. Only strictly necessary repairs will be carried out, such as securing a tight, even surface with sufficient friction to maintain reasonable road comfort and security.

The following procedure was decided upon to find out whether the bridge was sufficiently safe against collapse:

- Verification that the minimum load capacity existed all over the bridge by load test.
- Periodical inspections of the bridge until such a time when the bridge had to be demolished.

The periodical inspections include visual inspection of the bridge and measuring of deflections and deformations (gradient of column etc.). Further, the loading test has to be repeated at regular intervals.

5. PLANNING OF THE LOAD TEST

5.1 The Critical Regions of the Bridge

Based on the experience gained from the Eastern Bridge, the potentially damaged areas, which would affect the safety of the bridge, were pointed out:

- The transverse girders over columns where the short U-cables might be heavily corroded.
- The construction joints in the main girders where the longitudinal cables often were damaged by corrosion in the couplings.
- The cantilever wing on account of corrosion of the transverse prestressing cables.

- The span across the railway tracks where the concrete appeared heavily deteriorated.
- The foundation of the abutments.

The loading test procedure was organized to take all critical regions into account, both with regard to the application of the load and the positioning of the measuring equipment.

5.2 Loads

The size of the test load was determined in order to meet the requirement that, in the two traffic lanes and the service lane of the overpass, the bridge had to conform with the specification Civil Class 45 (in accordance with the Road Traffic Act), and in one lane, while the bridge was closed to traffic, Civil Class 100.

The load test was applied to the bridge using two drays, one with a constant load of 600 kN and the other with loads varying from 300 to 900 kN, using combinations of 12 concrete blocks of 50 kN each. The main girders were loaded in five stages up to the maximum of 1500 kN: 900, 1100, 1300, 1400 and 1500 kN. The drays were brought to a standstill at the various load points while the relevant measurements were made.

5.3 Measurements

The following measurements were made with regard to the main girders:

- With the load on mid-span, levellings were made to the centre of the span in question.
- With the load at a construction joint, levellings were made to the centre of the span in question and strain measurements were made at the joint.

Supplementary strain measurements were made for the two main girders of the span across the railway tracks which were placed in the section that had deteriorated most, and for a second span chosen for reference:

- With the load placed at mid-span, strain measurements were made at the centre of these two spans and at the cross sections above the nearest column.
- With the loads on the transverse girders close to these two spans, strain measurements were made across the bridge at the top and at the bottom of the girder and furthermore along the bridge at the underside of the girder in question.

6. ELECTRONIC EQUIPMENT

6.1 Equipments for Measuring Strain

Pin gauges with built-in displacement transducers (with linear variable differential transformer) were used for the strain measurements. The length of the pin gauge was chosen sufficiently big (1000 mm at mid-span, above columns and at transverse girders, 500 mm at construction joints) to facilitate the measuring of minor strains and to record beginning cracks, if any. The sensitivity of the transducer was better than 10^{-3} mm/m. Because of the variations in the air temperature during the relatively long loading test cycle, the pin gauges were protected by a jacket of insulating material.

6.2 Mobile Data Centre

The recording system attached to the pin gauges was installed in a mobile data centre. The system consisted of one data logger, one computer, two terminals with graphic screens, one printer and one plotter. This equipment ensured that the result was continuously recorded, processed and plotted out as load/deformation curves.

Each measurement of strain included measurement of the temperature of the pin gauge. Furthermore, the temperature of the air was recorded in addition to the temperature of the concrete at the centre of the top and bottom of the main girder cross section.

7. THE RESULTS OF THE FIRST LOAD TEST IN 1984

The deflection measurements at mid-span, with the load applied at mid-span, showed fine linear relations between the load and deflection. At main girder No. 3, a maximum deflection of approx. 7.5 mm was measured for normal spans and of 12.0 mm for the span across the railway tracks. The measurements indicate that a cracking stage had not yet been reached in the main girders, probably because the tensile strength of the concrete was higher than anticipated. Further, it may by interpreted as a sign that existing prestressing is sufficient.

The strain measurements, made at the transverse girders over the columns supporting the span across the railway tracks and the reference span, generally showed minor strains, which meant that temperature variations had affected the results considerably. After adjustment for the effect of the temperature variations, fairly linear relations between load and strain were established. The measurements made at transverse girders showed no sign of cracks, and there was no significant difference between the measurements made at the span across the railway tracks and the reference span, respectively.

The strain measurements at construction joints showed only minor strains, which again means that temperature variations have affected the measurements. As the strains measured were very small, it was difficult to interpret the results. It was, however, possible to conclude that no cracking had occurred at the construction joints during the loading test.

8. THE RESULTS OF THE SECOND LOAD TEST IN 1988

The load test in 1988 was performed with exactly the same equipment as the load test in 1984, both with regard to measuring system and applied drays with concrete blocks. Even the pin gauge was used in the same position.

A comparison shows that the results of the load test in 1988 are almost identical with the results of the load test in 1984. Only few examples of the measured result can be shown here, but measurements were made on a total of about 100 different spots.

8.1 Measurements of the deflection at mid-span across the railway tracks

In 1984, the deflection at the main girder No. 3 was measured at 12 mm and in 1988 at 12.5 mm. The curves show fine linear relation between load and deflection. It should be noted that the deflection of failure will be in the magnitude of 150 mm.

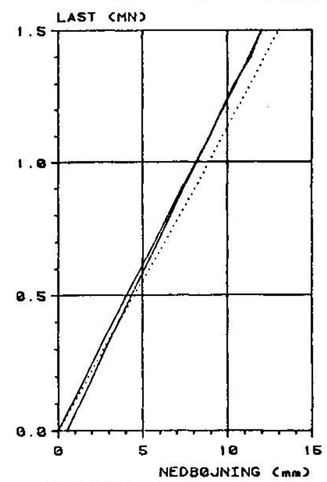


Fig. 4 1984

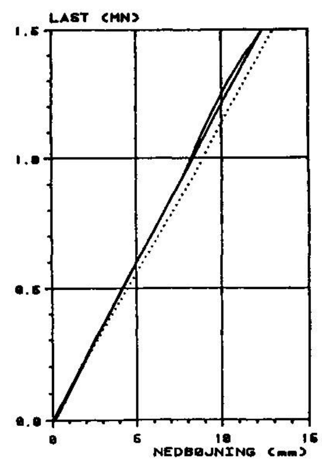


Fig. 5 1988

8.2 Measurements at the top of the transverse girder over columns

The results of the two load tests are very close to each other. Compared with the results from 1984, the deformation measured in 1988 is minor.

8.3 Measurements at the construction joint in the main girder

At the load test in 1984, the maximum deformation at the eastern side of main girder No. 3 was measured at 15/1000 over a length of 500 mm. In 1988, the deformation was measured at 14.5/1000 at the southern side of the joint. At the northern side the measurement was in 1984 11.5/1000 and in 1988 13/1000.

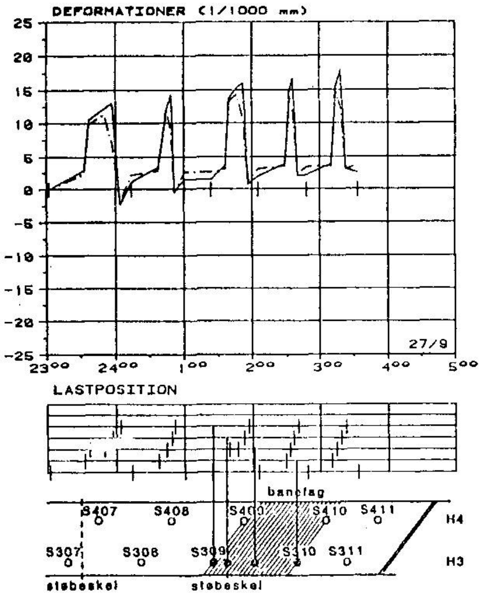


Fig. 6 1984

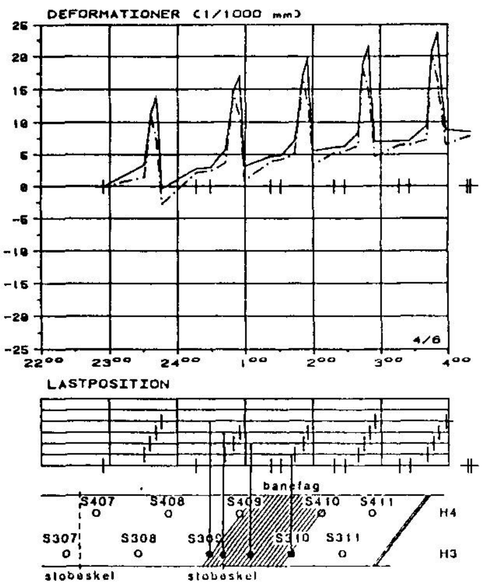


Fig. 7 1988

9. CONCLUSION

There are no signs of failure of any kind, neither in the substructure nor in the superstructure. It may thus be taken as verified that under the existing conditions, the bridge could meet the specified requirements with regard to load capacity.

However, the loading test provided no definite answer with regard to the safety factor against future collapse, nor of the expected service life of the bridge.

On the other hand, the construction has shown such a small increase in the measured deformation (measured on about 100 spots) that it has been decided to use the bridge for the next 4-6 years. Till that time, load tests will be carried out, but it is presently being expected that the remaining lifetime will be more than 10 years.

It should be noted that the bridge is carefully inspected four times a year in order to observe any increase in deterioration.

It has been possible to perform the load test during periods of slack traffic with a minimum of obstruction to the traffic.

Bridge Strengthening Using Load Relieving Techniques

Renforcement de ponts en utilisant les techniques de relevage

Brückenverstärkung durch Entlastungstechniken

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SUMMARY

The paper describes several new techniques for strengthening existing bridges to withstand increased loading by imposing dead load relief or load sharing. The techniques cover external prestressing, the installation of extra shear connectors and the use of shock transmission units. They all benefit from requiring minimum, if any, traffic disruption.

RÉSUMÉ

L'article expose quelques techniques nouvelles pour le renforcement de ponts existants, dans le but de supporter un accroissement de charge en imposant à l'ouvrage des sollicitations de soulagement ou des charges réparties. Les techniques comprennent une pré-tension externe, l'installation de connecteurs qui travaillent au cisaillement et l'usage d'unités de transmission de choc. Le procédé ne nécessite qu'une interruption de trafic minimale.

ZUSAMMENFASSUNG

Der Beitrag beschreibt verschiedene Techniken zur Verstärkung bestehender Brücken auf höheren Nutzlasten durch Verminderung der Eigenlasten oder durch Lastaufteilung. Es handelt sich dabei um aussenliegende Vorspannung, den Einbau von zusätzlichen Schubverbindungsmitteln und die Verwendung von Schock-Uebertragungselementen. Der Vorteil dieser Massnahmen liegt in der minimalen Beeinträchtigung des Verkehrs.

1. INTRODUCTION

Strengthening of the world's bridge stock is a growth industry. This is inevitable as the years pass because existing bridges are expected to carry traffic of increasing loading and intensity for which they were not originally designed. The same passage of time also means that existing bridges are increasingly subjected to weakening environmental hazards, ranging from winter de-icing salt to polluted atmospheric carbonation. Strengthening of an existing bridge may become necessary because of increasingly apparent overloading or because major repairs are required and the opportunity is taken to strengthen the bridge to higher standards while traffic restrictions are in operation. The traffic restriction aspect is usually dominant and often precludes straight bridge replacement. It also strongly influences the method of repair and those methods which involve little or no traffic restriction are strongly favoured. Strengthening operations by various load relieving and sharing techniques fall into this category. Some new techniques for load relieving existing bridges with minimum, if any, traffic disruption are described.

2. DECK BENDING RELIEF BY EXTERNAL PRESTRESSING

2.1 General

Conventional prestressing of a bridge deck imposes a permanent direct compression together with a bending moment which counters the applied dead load moments. The two effects can be most beneficial to tension-weak concrete decks and together they allow the deck to carry further superimposed dead and live load moments without exceeding the permissible bending stresses or load factors.

The bending moment reduction effect of added prestressing can also be used to advantage in relieving dead load bending in existing overloaded decks of reinforced concrete, steel or composite concrete deck/steel girder structures. This dead load bending relief can be sufficient to reduce the deck bending under full dead and live loading to permissible limits. Alternatively a bridge deck can be upgraded to carry increased superimposed dead and/or live loading.

In general the direct compression effect of the added prestressing is not helpful. Reinforced concrete allowable compressive stresses are usually lower than with prestressed concrete and extra compression in steel structures can lead to plate stability problems. It is therefore beneficial to mobilise as much of the prestressing bending moment reduction as possible and there is every advantage in locating the prestressing tendons at the beam extremities or even beyond.

2.2 External Prestressing Applied to an Existing Composite Deck

Rakewood Viaduct carries the M62 motorway between Lancashire & Yorkshire across a 36m deep valley, Figure 1. The 256m long six span continuous deck, completed in 1969, consists of ten 3m deep steel plate girders carrying and composite with an insitu reinforced concrete deck slab. The viaduct required upgrading to cater for a proposed increase in traffic and the more onerous requirements of the newly introduced BS5400 bridge code. The main shortfall was identified as an approximate 40% overloading in the steel girder compression flanges over the piers. Upgrading by 'unloading', using external prestressing, was found to provide an economical strengthening procedure with minimal disruptions on this heavily trafficked motorway.

Figures 2 & 3 indicate the procedure, which first requires the attachment of fabricated steel anchors to the locally stiffened underside of each steel beam bottom flange by HSFG bolting. Three pairs of 50mm or 36mm diameter Macalloy prestressing bars of overlapping lengths are then attached under each flange between piers. Upon stressing, hogging bending is set up in the mid span regions of the beam. However, it is the parasitic sagging moment over the piers, caused by deck continuity, which performs the required 'unloading' to acceptable stress limits in the bottom girder flanges over the piers.

The dispersion of the high anchorage loads into the girder flanges and webs and the associated local design had been examined using three dimensional finite element techniques. Special consideration has also been given to the provision of intermediate supports to prevent wind vibration of the stressing bars and anti-corrosion protection.

It so happens that a similar deck unloading procedure is being applied to an understrength three span composite girder viaduct in Iowa State, USA this year. Prior experimental work on large scale models has already been undertaken and covered in several recent papers by Professor F.W. Klaiber and his colleagues at Iowa State University. It has been agreed by both parties to undertake and compare monitoring of prestressing bar loads during and after construction.

2.3 External Prestressing Applied to a Reinforced Concrete Deck

Figure 4 shows how a similar external prestressing technique was used to 'unload' the rectangular beams of an understrength two span continuous reinforced concrete deck in South Wales. In this case prestressing was by cables located on the sides of the beams and anchored and deflected by steel assemblies attached by epoxy grouted bolts passing through the beams.

3. SUBSTRUCTURE TRACTION & BRAKING LOAD RELIEF USING SHOCK TRANSMISSION UNITS (STUs)

3.1 General

A large number of our existing stock of viaducts feature long sequences of simply supported deck spans, often supported on a series of high & substantial piers. This is particularly evident in major river crossings where high navigation clearances require long approach viaducts, Figure 5. The piers under each simply supported span inevitably carry fixed bearings for one span alongside free bearings for the adjacent span. This means that the design longitudinal traction & braking plus wind forces must be individually applied to each deck span throughout the viaduct. Main resistance is offered by the pier carrying the fixed bearings of that particular span.

Current integrity assessments of a number of these viaducts often indicate that the piers are understrength due to increases in the deck longitudinal loading since original design, sometimes accompanied by damage generated by road salt, carbonation or ASR. A substructure of this type with, say, 10 equal height piers has a total resistance capacity of approximately 10 times the original deck design traction & braking longitudinal loads. This total resistance capacity can be mobilsed by providing load transfer mechanisms across the deck joints, ideally shock transmission units.

3.2 Shock Transmission Units (STUs)

STUs are mechanisms which are connected across movement joints between structural elements. They transmit slow joint movements like temperature and shrinkage with negligible resistance and, when required, transmit momentary impact forces like traction, braking & earthquake with negligible movement.

A simple, economical & minimum maintenance bridge STU was developed in the UK some years ago. Instead of oil the STU utilises the peculiar properties of 'bouncing putty', a silicone compound which will readily deform under slow pressure but becomes rigid under impact. The unit consists of a steel cylinder containing a loose fitting piston fixed to a transmission rod, the void round the piston being filled with the silicone putty. Under slow movement this putty is squeezed around the piston and displaced from one end of the cylinder to the other, Figure 7.

3.3 Load Relief using STUs for Viaduct Piers of the London Docklands Light Railway

The newly completed viaducts carrying London's Docklands Light Railway, Figure 6, were designed for a train service which, due to a breathtaking increase in adjacent development, will now require considerable expansion before 1990. This will mean heavier & more frequent trains, which will add braking & traction effects in excess of those originally catered for.

Figure 7 shows a typical as-built seven span deck unit, continuous between expansion joints. Train traction and braking loads are currently shared among the slender piers, which generally support the deck via rubber bearings. STUs are being installed at rail level between joints such that, when the new increased longitudinal traction & braking loading is applied to one particular seven span unit, load is beneficially transmitted and shared with adjacent seven span decks sufficient to require no pier and foundation strengthening in any substructure. This simple procedure represents a tremendous saving in cost and interference with the existing train service.

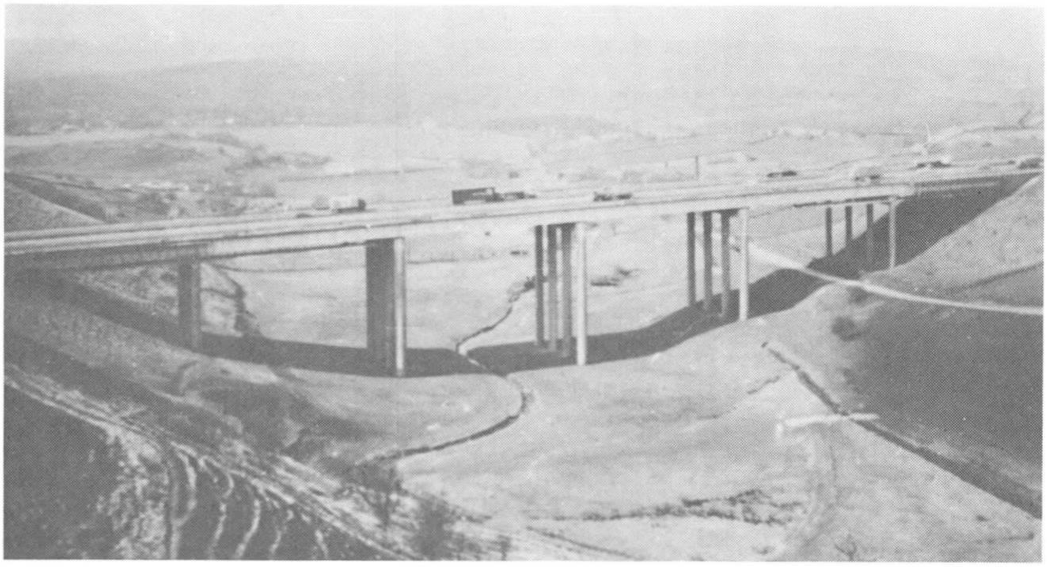

Fig. 1 Rakewood Viaduct

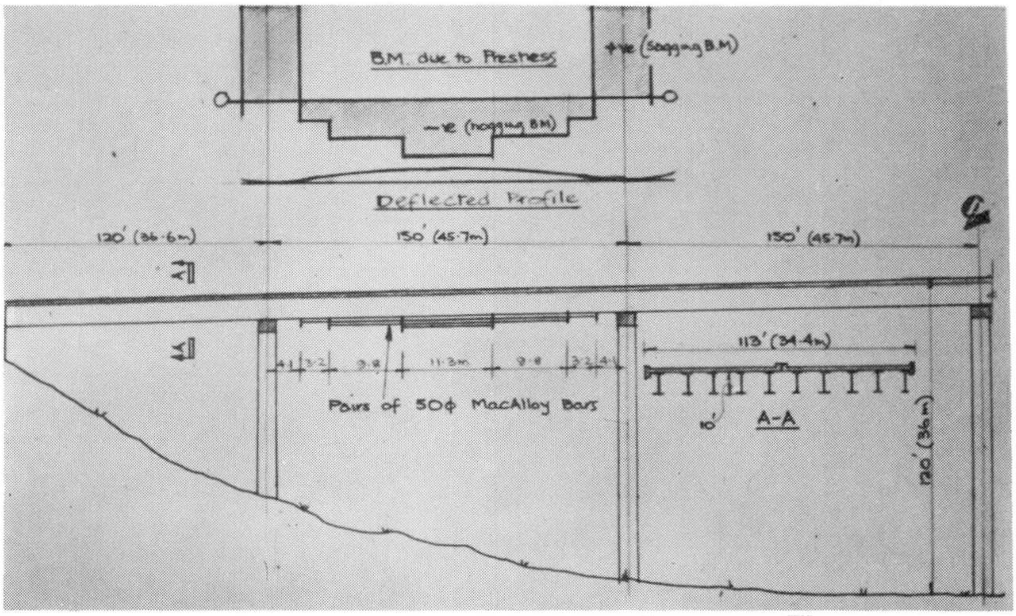

Fig. 2 External Prestressing

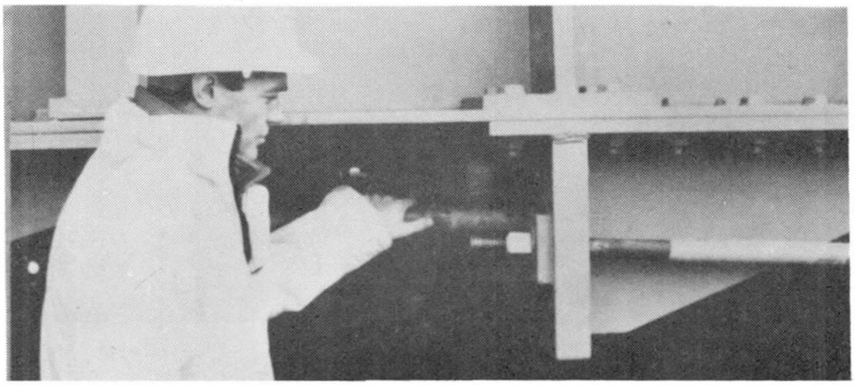

Fig. 3 Prestressing Anchorages

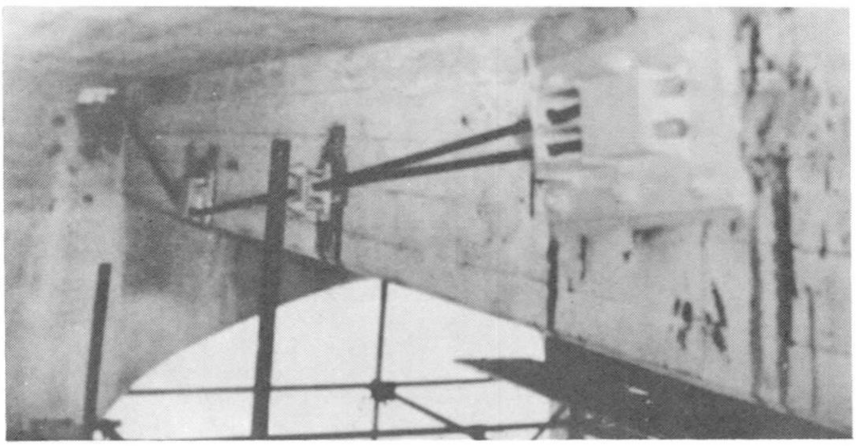

Fig. 4 External Prestressing of RC Beam

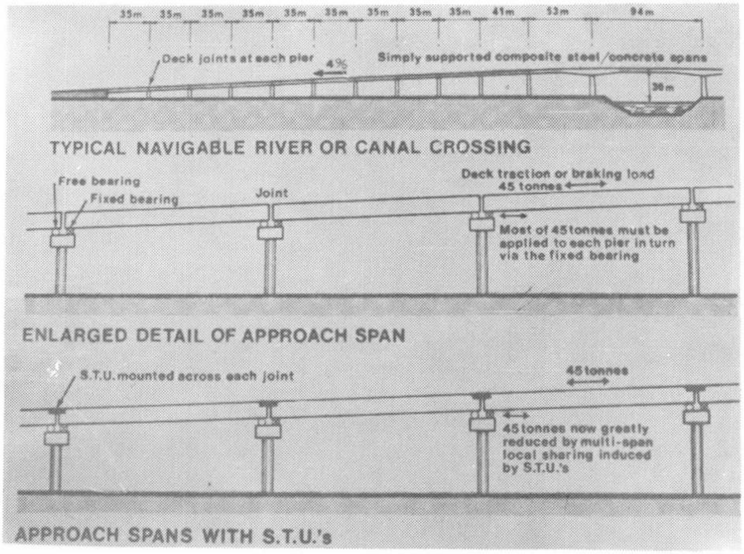

Fig. 5 Substructure Strengthening using STUs

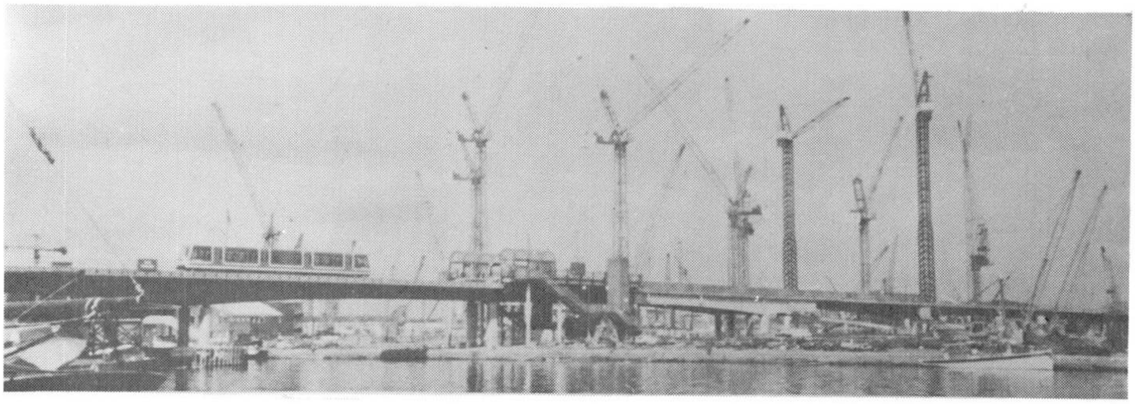

Fig. 6 Docklands Light Railway Strengthening

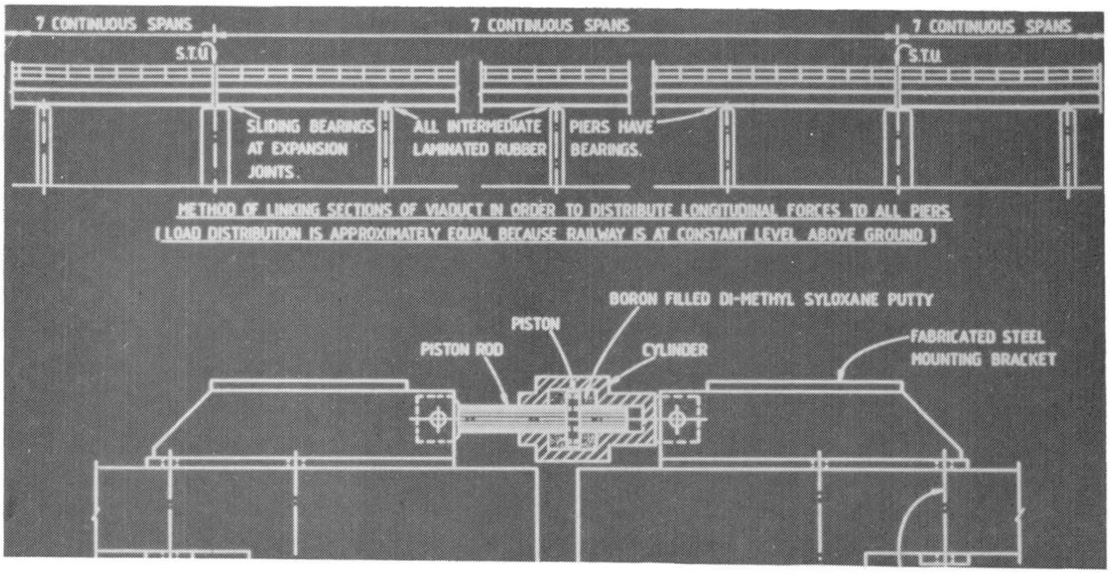

Fig. 7 Detail of STU & Mounting

Shotcrete for Concrete Repairs — Function and Durability

Béton projeté pour les réparations du béton — fonction et résistance

Spritzbeton zur Betonreparatur — Funktion und Beständigkeit

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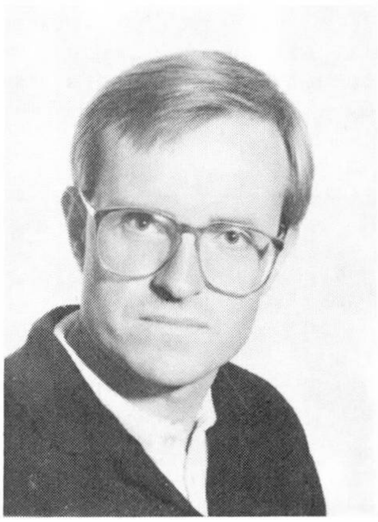

Anders Andalen, born 1955, got his civil engineering degree at Chalmers University of Technology, Gothenburg, Sweden. For three years he worked within an engineering consulting firm. In 1982 he was employed by the National Testing Institute, and is now responsible for the concrete division at the Department of Building Technology.

SUMMARY

The aim of this project, which is to continue until 1990, is to increase knowledge of the function and durability of shotcrete for concrete repair and to develop suitable test methods. Methods for testing bond, salt-frost resistance, strength and shrinkage have been analysed for a number of different qualities of shotcrete. The bond between shotcrete layers and sawn concrete surfaces has been studied for different kinds of surface treatment. The highest bond was obtained when the shotcreting was carried out on dry surfaces.

RÉSUMÉ

L'objet de cette recherche, dont la durée s'étendra jusqu'en 1990, est d'améliorer nos connaissances sur la fonction et sur la résistance des réparations réalisées avec du béton projeté et de développer les méthodes d'essai adéquates. Les essais d'adhésion, la résistance du béton frais vis-à-vis du gel et du sel, la résistance d'échantillons et de retrait sont autant de paramètres déterminés pour différentes qualités de béton projeté. L'adhésion entre les couches de béton et les surfaces sciées a été étudiée pour différents traitements de surfaces. La meilleure adhésion est obtenue lorsque le béton projeté est appliqué sur des surfaces sèches.

ZUSAMMENFASSUNG

Ziel dieses Projektes, das bis 1990 fortgeführt wird, ist es, das Wissen über die Funktion und die Beständigkeit von Spritzbeton bei Betonreparaturen zu erhöhen und passende Testmehoden zu entwickeln. Methoden zur Untersuchung der Haftfähigkeit, der Frostbeständigkeit bei Gegenwart von Salz, der Festigkeit und des Schwunds wurden für einige Spritzbetonsorten unterschiedlicher Qualität analysiert. Die Haftfähigkeit zwischen Spritzbetonschichten und gesägten Betonoberflächen wurde bei verschiedenartiger Oberflächenbehandlung studiert. Die beste Haftung wurde bei Anbringung auf trockenen Oberflächen erzielt.

1. INTRODUCTION

The aim of the present project, which is to continue until 1990, is to increase knowledge of the function and durability of shotcrete for concrete repair and to develop suitable test methods. Of special interest is the rehabilitation of bridges and other structures subjected to frost and deicing agents in aggressive environments.
A shotcrete testing laboratory was established at the National Testing Institute, where it is possible to perform shotcrete in full scale under controlled conditions, see fig 1. All shotcreting necessary for the tests presented in this paper was carried out by contractors, under the supervision of personnel from the Institute.

Fig. 1 Shotcrete gunned towards formwork, with bottoms of sawn concrete surfaces.

As a part of the project the bond between shotcrete layers and sawn concrete surfaces was studied. These surfaces were preparated in six different ways. The shotcrete, both dry mix and wet mix, was gunned into moulds where the bottom consisted of a sawn concrete surface. The shotcrete layers were also used for testing strength, salt-frost resistance and shrinkage. The shotcreting was performed by two contractors with experience in dry mix and wet mix techniques, respectively. According to the contractors experience, the shotcrete composition and the equipment used were expected to give high bond strength and good durability.

2. BOND STRENGTH

2.1 Test method

The shotcrete was sprayed into molds with bottom surface dimensions of 400 x 400 mm. The bottoms consisted of 100 mm thick concrete slabs with sawn surfaces and the walls of steel plates with an inclination of 45 °, see fig 2. After shotcreting the panels were covered with a plastic film and stored in 20±2 °C for one week. The film was then removed and the slabs kept in a laboratory environment at 20±2 °C, RF 50± 10 %. Four cylinders, all with a diameter of 70 mm were drilled out from each of the panels. The cores were sawn to a length of 70 mm, with the adhesion zone in the center of the specimen. When the specimens were 5 weeks old, stiff adapters with screw connectings were glued to the ends of the specimens, and the bond strength was determined in a tensile testing machine.

2.2 Manufacturing of the contrete slabs

To obtain a well-defined surface, it was decided to perform the shotcreting on concrete slabs with sawn surfaces. Two different concrete qualities were used for the slabs: the first with a water cement ratio (w/c) of 0.43 and an air content of 5.5 % and the second with a w/c of 0.57 without any entrained air. One week after casting, the slabs were sawn into two pieces, each with a thickness of about 100 mm and with a sawn surface of 400 x 400 mm. The sawn slabs were stored in water for one further week and then in a climate room, 20±2 °C, RF 50± 5 %, until preparation before shotcreting.

2.3 Preparation of the surfaces

Before shotcreting was carried out, the sawn slabs were adapted to the metal molds. These were then fixed to a rig in the shotcrete testing ground, see fig 2. The surface was subjected to one of the following preparation procedures:

a) in air until the time of shotcreting
b) in water for 48 h before shotcreting
c) in water for 24 h before shotcreting
d) the surface was sprayed with water for 0.5 h before shotcreting
e) the surface was prepared with a bond improving agent before shotcreting
f) the surface was splatterdashed 0.5 h before shotcreting

2.4 Dry mix concrete

The dry mix concrete was delivered by the contractor. It consisted of one part of Portland cement and four parts of aggregates, with a maximum particle size of 8 mm.

2.5 Wet mix concrete

The wet mix concrete was delivered by the contractor in big bags, and consisted of cement, aggregates, silica and a super plasticizing agent. Air-entraining agent was added to the water during mixing, and an accelerating admixture was added to the nozzle when the concrete was sprayed.

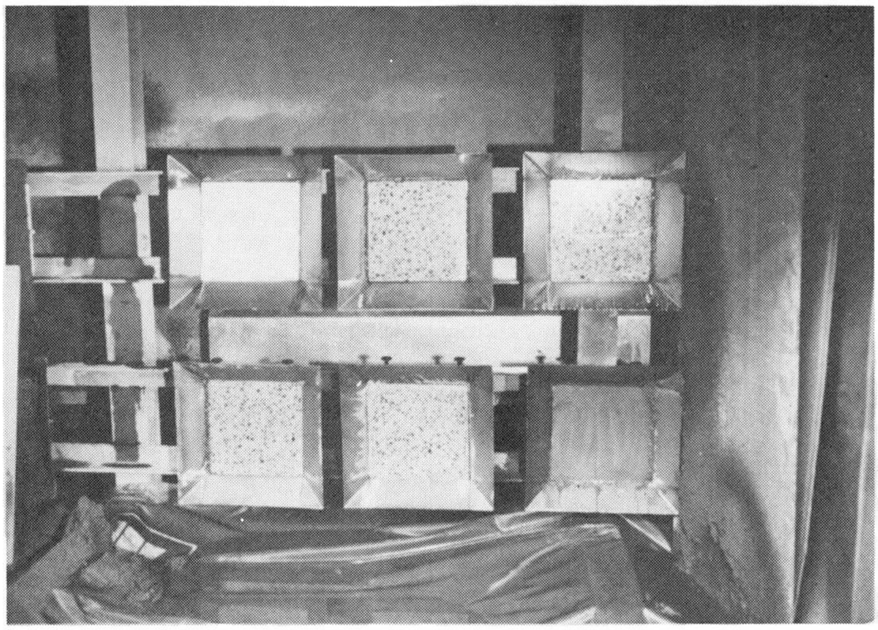

Fig. 2 Six prepared sawn surfaces before shotcreting.

2.6 Test results

The bond strengths for dry mix and wet mix shotcrete are shown in the table below. A complete presentation of all test results can be found in (2). Dry mix A and B are identical, but were used on different occasions. There is no significant difference between the bond due to the two different concrete qualities of the slabs. Bond strength for the wet mix shotcrete was lost during handling of the specimens for all preparations but preparation in air. These low bond strength values are assumed to be equal to zero in table 1. The highest bond strengths were obtained when shotcreting was carried out on dry surfaces. This means that when shotcreting is performed on real structures no water should be sprayed on the surfaces before the spraying of shotcrete.

Preparation of the surfaces of the slabs before shotcreting

Shotcrete mix		Air	Water for 48 h	Water for 24 h	Water for 0.5 h	Bond agent	Splatter-dashed
Dry mix A	Mv	1.80	0.17	0.59	0.60	0.75	0.78
	(s)	(0.34)	(0.20)	(0.15)	(0.23)	(0.18)	(0.18)
Dry mix B	Mv	1.37	0.43	0.50	0.53	0.54	0.75
	(s)	(0.67)	(0.33)	(0.14)	(0.16)	(0.13)	(0.24)
Wet mix	Mv	1.74	0	0	0	0	0
	(s)	(0.11)					

Table 1 Bond strength (MPa) for dry mix and wet mix shotcrete. Each value is the mean value of four test results and the values within brackets are standard deviation.

3. Salt-frost resistance

A high salt-frost resistance is of utmost importance for materials used in the repair of bridges and other structures in aggressive environments. Factors affecting the salt-frost resistance are analysed in this project. To begin, a comparison was made of the dry mix and the wet mix shotcrete qualities used for testing the bond strength.

3.1 Test method

Salt-frost resistance was tested according to Swedish standard SS 13 72 44 (1,3). All tests were carried out on the top surface of specimens with a depth of 50 mm and a diameter of 100 mm. All specimens used in this investigation were sawn from drilled cores. The specimens were stored for 7 days after sawing in a climate room with a temperature of 20 ± 2 °C and RH 50 ± 5 %. During this period, rubber cloth was glued to all surfaces of the specimen, with exception of the test surface. When the specimens had been in the climate room for 7 days, tap water was poured on the test surface. The test was started 3 days afterwards. Before the specimens were placed in the freeze chamber, the water on the test surface was replaced by a 3 % NaCl solution and all surfaces except the freeze surface were covered with a thermal insulation layer. Plastic film to protect from evaporation was applied over the salt solution. The specimens were then subjected to repeated freezing and thawing for 56 cycles. Each temperature cycle lasted for 24 hours, and the temperature in the salt solution varied between 20 °C and -18 °C. After 7, 14, 28, 42 and 56 cycles, scaled material from the test surface was collected and dried. The test results examined were the losses of mass per square meter.

3.2 Test results

Mean values of the scaling in the salt-frost resistance test are shown in table 2. A complete presentation of all test results can be found in (2). The highest scaling after 56 cycles for the dry mix shotcrete was 6.9 kg/m²; the lowest was 0.30 kg/m². It is normally assumed for concrete that the spalling for a fair salt-frost resistance should not exceed 1 kg/m². In the present project, this criteria was fulfilled for the wet mix shotcrete, while the results for the dry mix shotcrete were greater than twice this value. It can also be seen that there is a large standard deviation for the dry mix shotcrete. This is probably because it is difficult to maintain a constant w/c-ratio in a dry mix process, as the water content is manually controlled at the nozzle during shotcreting.

Shotcrete mix		Scaling after n cycles (kg/m²) 7	14	28	42	56
Dry mix	Mv	0.28	0.78	1.42	1.80	2.22
	(s)	(0.19)	(0.52)	(1.10)	(1.7)	(2.14)
Wet mix	Mv	0.17	0.32	0.52	0.62	0.71
	(s)	(0.11)	(0.20)	(0.29)	(0.36)	(0.40)

Table 2 Salt-frost resistance for dry mix and wet mix shotcrete. (Mv = mean value, s = standard deviation).

4. Conclusions

The following primary conclusions can be drawn according to the test results of this project:

- The highest bond strength seems to be obtained for both wet mix and dry mix shotcrete when the shotcreting is performed on dry surfaces.
- The bond strength was very poor for the wet mix shotcrete when shotcreting was performed on wet surfaces.
- The quality, i.e. the strength and air content, of the concrete underlayer seems to have very little influence on bond strength.
- Of the two shotcrete qualities tested in this project, wet mix shotcrete demonstrates a better salt-frost resistance than dry mix shotcrete.
- The scattering of the salt-frost resistance is wide for dry mix shotcrete. This is probably because it is difficult to keep the w/c-ratio constant during the dry mix process.
- The methods used in this project for testing bond strength and salt-frost resistance seem to be suitable for evaluating the quality of shotcrete.

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Method of Repair for Fatigue Damage in Steel Bridges

Méthode de réparation de fissures par fatigue des ponts métalliques

Verfahren zur Reparatur von Ermüdungsschäden an Stahlbrücken

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SUMMARY

Recently, steel highway bridges in Japan have suffered fatigue cracks. This paper deals with some cases of fatigue damage occurring in steel roadway bridges and the tungsten inert gas (TIG) arc remelting process being effective as a method of repairing such fatigue damage. It also includes discussions on detailed re-welding procedure, optimum condition and effects of retrofitting on the fatigue strength.

RÉSUMÉ

Récemment, les ponts routiers métalliques au Japon subissent des fissurations par fatigue. Cette communication présente certains exemples de dommages par fatigue survenus dans les ponts routiers métalliques, ainsi que le procédé de refusion à l'arc avec électrode de tungstène sous gaz inerte (TIG), procédé effectif pour la réparation de tels dommages par fatigue. Elle traite également la procédure détaillée de resoudage, la condition optimale du rétro-ajustement et ses effets sur la résistance à la fatigue.

ZUSAMMENFASSUNG

In letzter Zeit hat man in Japan Rissschäden an Strassenbrücken aus Stahl festgestell. Die vorliegende Abhandlung behandelt verschiedene Fälle von Ermüdungsschäden an Strassenbrücken aus Stahl sowie die wirksame Anwendung des Wolfram-Inertgas-Lichtbogenschweissverfahrens (WIG) zum Ausbessern solcher Ermüdungsschäden. Weiter werden das Nachschweissverfahren in seinen Einzelheiten und die optimalen Bedingungen sowie die Auswirkungen einer Nachrüstung auf die Dauerfestigkeit diskutiert.

1. INTRODUCTION

Recently, fatigue cracking has damaged steel highway bridges in Japan. It has resulted from higher volume of traffic and heavier truck weight than originally anticipated, lower rigidity of floor systems and a lack of attention to fatigue cracking on welded connections in the design stage.

Fatigue damage appears first in the reinforced concrete floor slab 5 to 10 years after the bridge commences service. Over a period of 15 years, fatigue crack begins to be detected mainly at the toe and/or bead surface of the boxing-welds on connection plates [1]. Investigations and analyses have revealed how cracking occurs. However, a question arises frequently as to what is a proper method of repairing or retrofitting the damaged structure because of the absence of past experience in this regard in Japan.

The strengthening of a cracking detail by welding a new member in the field appears to result in "defective repair," as reported in U.S.A. [2], and there is a possibility of new fatigue cracks being induced due to a change in the stress distribution by splicing plates. Therefore, repair using the tungsten inert gas (TIG) arc remelting process seems to be effective.

This paper deals with the status quo of the fatigue cracks in Japan, how to apply the TIG arc remelting process, a detailed method of re-welding, optimum condition, and effects of retrofitting on the fatigue strength.

2. CASES OF FATIGUE DAMAGE

The cases of fatigue damage reported to have developed in many of the steel highway bridges in Japan are shown in Fig. 1. Most fatigue cracking has concentrated at the secondary members such as the transverse stiffeners with sway bracing members, and gusset plates with lateral bracing members.

These members are not subject to a safety verification for fatigue resistance in ordinary

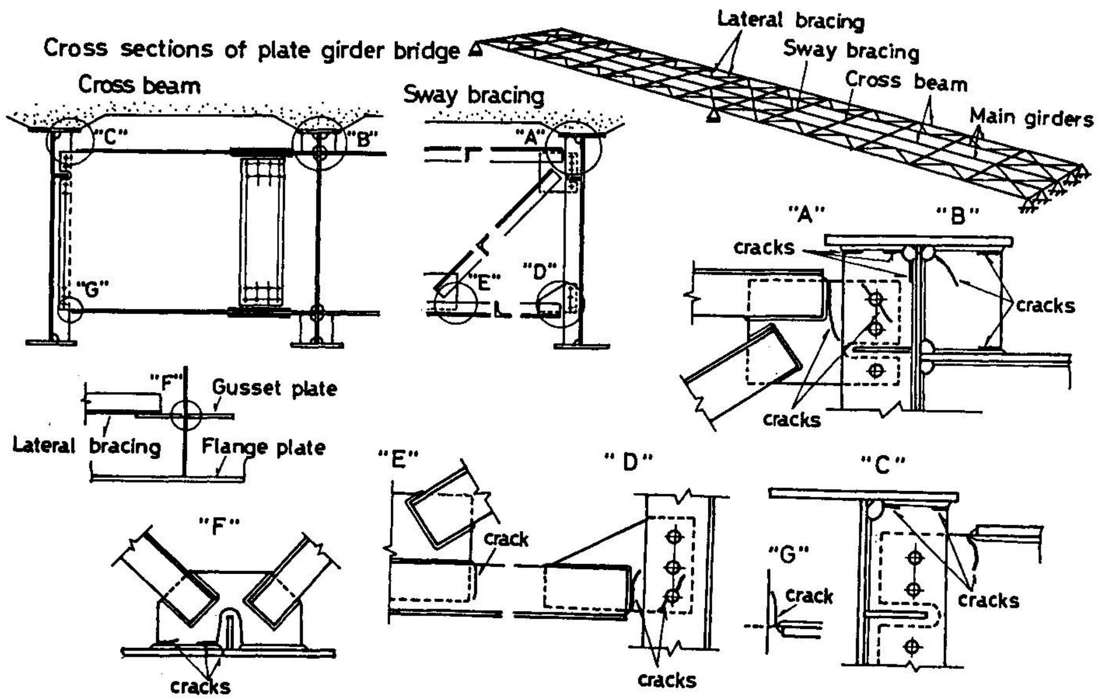


Fig. 1 Cases of fatigue damage in steel highway bridges

design practice, and these details were applied to the steel highway bridges as the standard details until the early 1970s.

Examining the locations of fatigue damage by the type of joint connection, the majority have developed in the welded joints, particularly in the fillet-welded joints.

Of the cracks, those that have developed at the toes of fillet-welded joints, which connect the transverse stiffeners to the upper flanges or the webs of girders with sway bracings, are numerous and the most typical in Japan's highway bridges. Such a crack is characteristic in that it occurs simultaneously in the bridges of same structural type with identical weight, traffic volume, and speeds of traveling vehicles. This situation urges the need for clarification of the cause of crack occurrence and the development of reliable methods of repairing and strengthening. The probable cause of crack occurrence that can be considered at present is that, as shown in Fig. 2, it is generated by the local stresses due to the differences in deflection of the main girders with sway bracing members and the deformation of reinforced concrete floor slabs [3]. The properties of these local stresses, however, would vary with such factors as the spacing between the main girders, the thickness of the deck, the construction of the transverse stiffeners or sway bracings, and the location of traffic lanes.

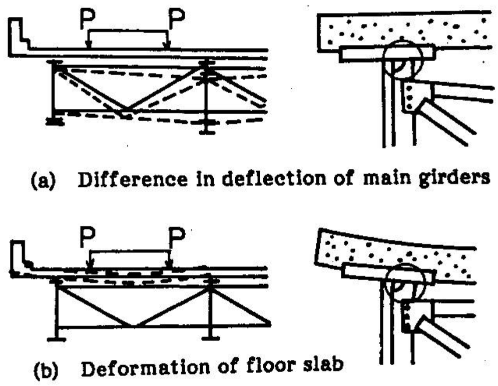


Fig. 2 Probable cause of local stress occurrence

3. RETROFITTING OF FATIGUE-DAMAGED DETAILS

3.1 Repair Methods

In retrofitting of fatigue-damaged bridge members, it is necessary to investigate the causes of crack occurrence and extent of damage to determine a repair method capable of eliminating the causes.

The idea of preventing the occurrence of fatigue damage, and also its repair, consists of two cases; one is to reduce the stresses that are generated in the damaged member under live load and the other is to enhance the fatigue strength of the damaged member. In the former case, it is necessary to change the type of structure or details. In the case of applying it to the existing structure, achieving a construction or joint, and considering on-site workability, is very often difficult.

Most fatigue damage occurs at welded joints, initiating often from the toes of fillet-welded joints where stresses are highly concentrated. As shown in Fig. 3, the crack develops from boxing-welded joints. In many cases, cracks that occur at such locations are relatively small in size, and the extent of damage is also minor. In such a case, an effective method would be to carry out the repair welding of only the damaged area, followed by smoothing the weld toe to increase the fatigue strength of the joint.

The tungsten inert gas (TIG) arc remelting process is shown in Fig. 4. TIG remelting process is usually used to finish the weld toe by smoothing its shape by melting the toe with a non-consumable tungsten electrode. This process surpasses grinding finish in work execution time and cost. Also, with cracks already developed, the use of the TIG process would be effective to remelt the fatigue crack into the base metal. Primarily this process was applied to

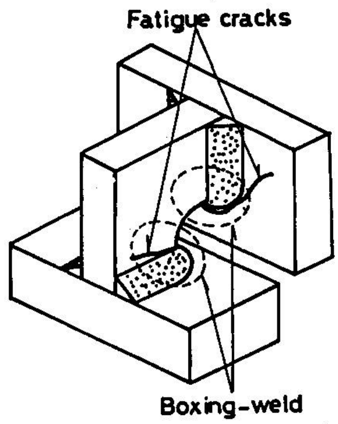


Fig. 3 Fatigue cracking initiated at boxing-welds

offshore structures, though it has been applied also to bridge fatigue damage recently [4][5].

The remelting equipment ordinarily used is driven by DC power with drooping V-I characteristics. A high frequency source is used to generate arc. The electrode is 3.2mm or 4.0mm in diameter and composed of 2 percent thoriated tungsten. The shielding gas is 100% argon. The welding unit is equipped with a water tank and a recirculating pump to cool the torch.

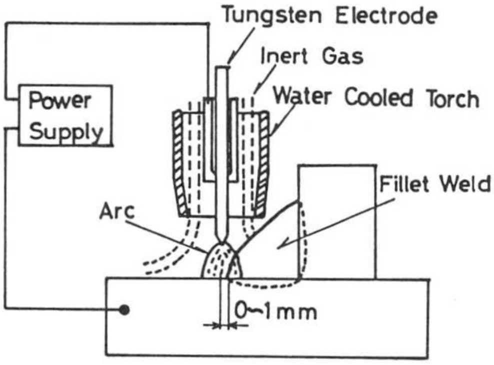


Fig. 4 TIG arc remelting process

Experience in laboratory and field has proved that few-hour training is all that required to achieve the desired retrofit condition. The results of in-situ retrofitting test revealed that a small change in the arc length due to the vibration of the bridge did not affect the TIG remelt, and under the normal service conditions, the flow of traffic was not disrupted.

3.2 Effects of the TIG Process

The possible mechanism of increasing the fatigue strength by means of the TIG process is to reshape the profile of the weld toe and to increase the tensile strength with an increased hardness of the TIG processed area. TIG process was applied to the boxing-toe of fillet weld as shown in Fig. 5, so as to examine the profile of weld toe and depth of penetration. The specimen was sectioned, polished and etched after the performance of TIG remelting. Fig. 6 (a) and (b) show the appearances and macroscopic photographs of boxing-toes. The remelting conditions are shown in Table 1. The remelting of all welds was performed at the flat position. Although there seems no difference in the appearances of bead surfaces shown in Fig. 6 (a), the weld toe was smoothed by the TIG process as shown in the macroscopic photographs. Thus, the application of the TIG process enables the profile

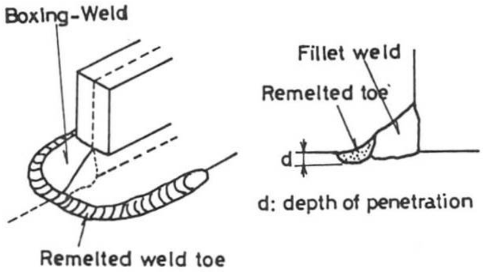
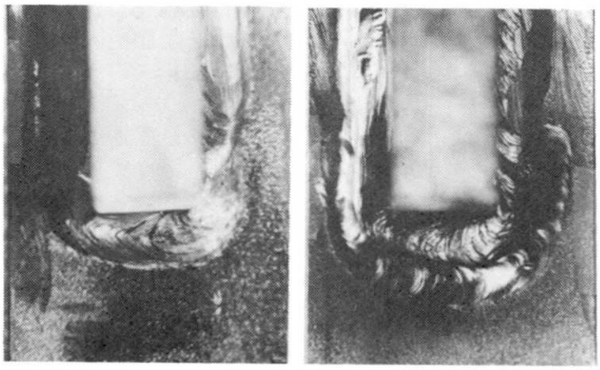


Fig. 5 TIG process at a boxing-weld

Table 1 TIG arc remelting condition

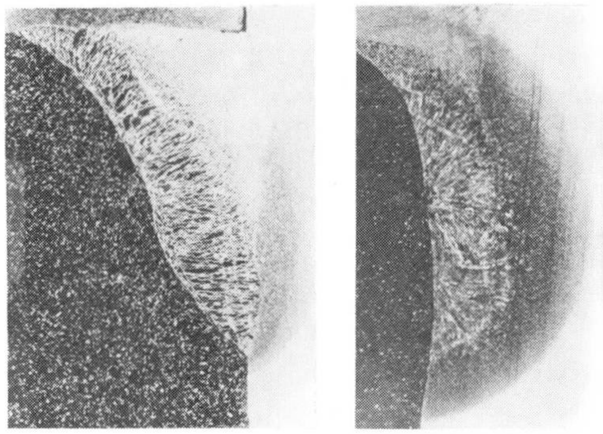
polarity	dirrect current electrode negative
electrode	2% thoriated tungsten (3.2mm ϕ)
welding position	flat position
current	240 A
speed	50s / 100mm
flow rate of gas	10l / mm (argon)



As-welded TIG-remelted

Fig. 6(a) Appearance of bead surface at a boxing-weld of the weld toe to reduce the magnitude of stress concentration, thereby preventing the occurrence of cracking at the weld toe. Fig. 7 shows the results of measuring the Vickers' hardness near the weld toe before and after the application of the TIG process. While the maximum hardness before the TIG process is H_∇ = 240, that after the process is H_∇ = 275, indicating an increase in the hardness. This could be another factor in improving the fatigue strength.

In the case of remelting to remove the fatigue crack by TIG process, the depth of remelt penetration is an important factor for the effectiveness of retrofitting. Being evident from Fig. 6(b), 2.5mm is the approximate remeltage by the TIG process, and cracks with a depth less than 2mm can be removed by the TIG process. On the other hand, the crack that occurs at the fillet-weld toe is semi-elliptic in the early stage of crack growth. As shown in Fig. 8, the ratio of its surface length ℓ to depth a is a/ℓ = 1/5. Thus, with cracks less than 10mm in length, it is possible to estimate that their depth is less than 2mm. By the current magnetic particle examination, it is fully possible to detect a crack with a surface length of about 10mm. Furthermore, the estimated crack length corresponds well to the actual crack length [6]. Therefore, with the crack length measured in advance by the magnetic particle examination, on finding that its length is less than 10mm, the crack can be removed by TIG process.



As-welded TIG-remelted

Fig. 6(b) Macroscopic photographs of boxing-weld

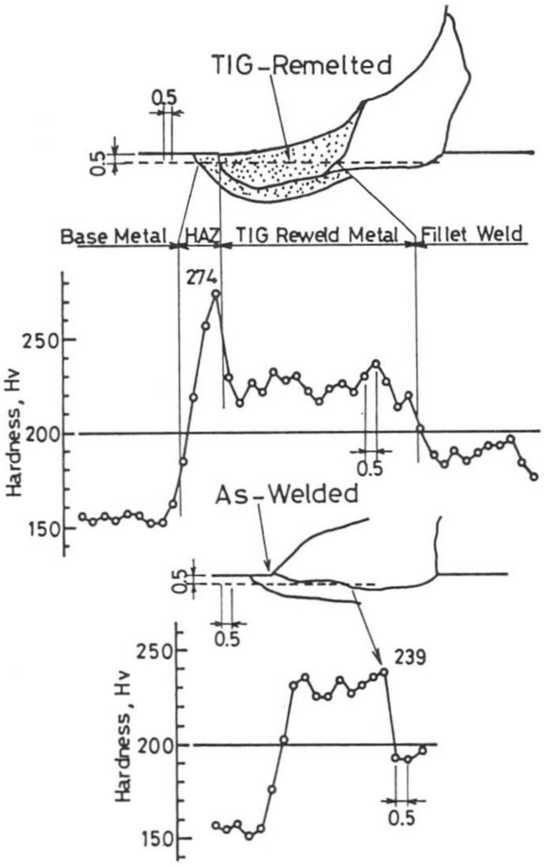


Fig. 7 Vickers' hardness near a boxing-weld toe

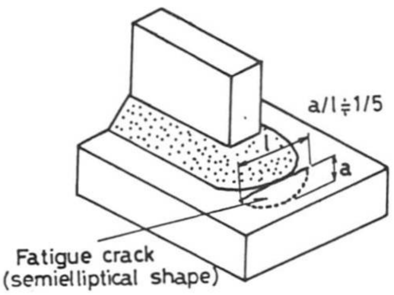


Fig. 8 The profile of the fatigue crack

In addition, the remelting position slightly affects the depth of reweld penetration. In the case of remelting at the overhead position, cracks smaller than approximately 2.0mm deep can be removed.

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Seismic Shear Strengthening for Short Columns

Renforcement de colonnes endommagées par rupture fragile lors de séismes

Verstärkungtechniken für Stützen unter Erdbebenbeanspruchung

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SUMMARY

To develop an effective method for strengthening the existing reinforced concrete short columns which are expected to fail in brittle shear mode during several earthquakes, experimental studies have been conducted. Test results demonstrate that, if the short column is strengthened by a welded steel square tube, then brittle shear failure does not occur and the colunm can develop its ultimate flexural moment capacity. The proposed method is also applicable for repair and rehabilitation of damaged short columns failed in shear mode during severe earthquakes.

RÉSUMÉ

Afin de développer une méthode judicieuse pour le renforcement des colonnes en béton armé faiblement élancées, lesquelles risquent, durant quelques séismes, une destruction par rupture fragile, des études expérimentales on été conduites. Les résultats ont montrés que les ruptures fragiles peuvent être évitées lorsque les colonnes courtes sont renforcées à l'aide d'un tube rectangulaire en acier soudé. Dès lors, la colonne peut développer sa pleine capacité de résistance vis-à-vis des moments de flexion. La méthode proposée est applicable pour la réparation et pour la réadaptation des colonnes faiblement élancées, dont les dommages furent causés par une rupture fragile liée à l'action de séismes.

ZUSAMMENFASSUNG

Zur Entwicklung wirksamer Verstärkungstechniken für kurze Stützen, welche unter Erdbebenbeanspruchung ein sprödes Schubversagen aufweisen, wurden experimentelle Untersuchungen durchgeführt. Die Resultate zeigen, dass mit einer Umhüllung aus Stahlblech kein sprödes Versagen auftritt und der volle Biegewiderstand entwickelt werden kann. Die Methode eignet sich auch zur Instandstellung erdbebengeschädigter kurzer Stützen.

1. INTRODUCTION

Numerous examples of shear failures in reinforced concrete (R/C) short columns have been reported during recent earthquakes in Japan and other earthquake countries. In Japan a new design code provisions for R/C columns was proposed in 1970 in order to prevent the short columns from brittle shear failures. On the contrary, a large number of R/C building structures having short columns which were designed in accordance with the old design provisions are still in use throughout the country. Preliminary analysis by authors indicates that most of those old short columns and some of the new short columns, especially in school buildings, are expected to fail in brittle shear failure modes during strong earthquakes. Therefore, it is one of the important engineering problems to be solved to develop an effective method for strengthening, repair and rehabilitation of those existing structural members at minimum cost. Herein, seismic shear strengthening, repair and rehabilitation methods by using steel plates are proposed to improve the seismic behavior of R/C short columns in the existing building structures practically, easily and inexpensively.

2. SPECIMENS

Test specimens adopted in the present study are 1/3-scale models of 1.5-story beam-column subassemblage belonging to lower levels of 4-story R/C school buildings. Overall dimensions of a typical subassemblage (Specimen CE) are shown in Fig.1 together with cross-sectional details of the first-story short column and spandrel beams of test specimens of Group E in Table 1. Shear-span-to-depth ratio of the first-story short column is 1.0 for all of the specimens. Nine different specimens tested are listed in Table 1, where expected failure modes determined by theory are also schematically illustrated. Each of the test specimen is classified into three groups; Group E (Existing), Group S (Strengthening) and Group R (Repair and Rehabilitation) as shown in Table 1. Specimens CE and BE in Group E are model subassemblages of two existing R/C school buildings which were designed and constructed in accordance with the current aseismic design provisions in Japan. Both of the specimens have the same cross-sectional details in their short columns, where area of longitudinal reinforcement is p_g=2.50 percent of gross area of the column section and shear reinforcement ratio is p_w=0.33 percent. Only difference in details between two specimens is total amount of longitudinal reinforcement in the spandrel beams.

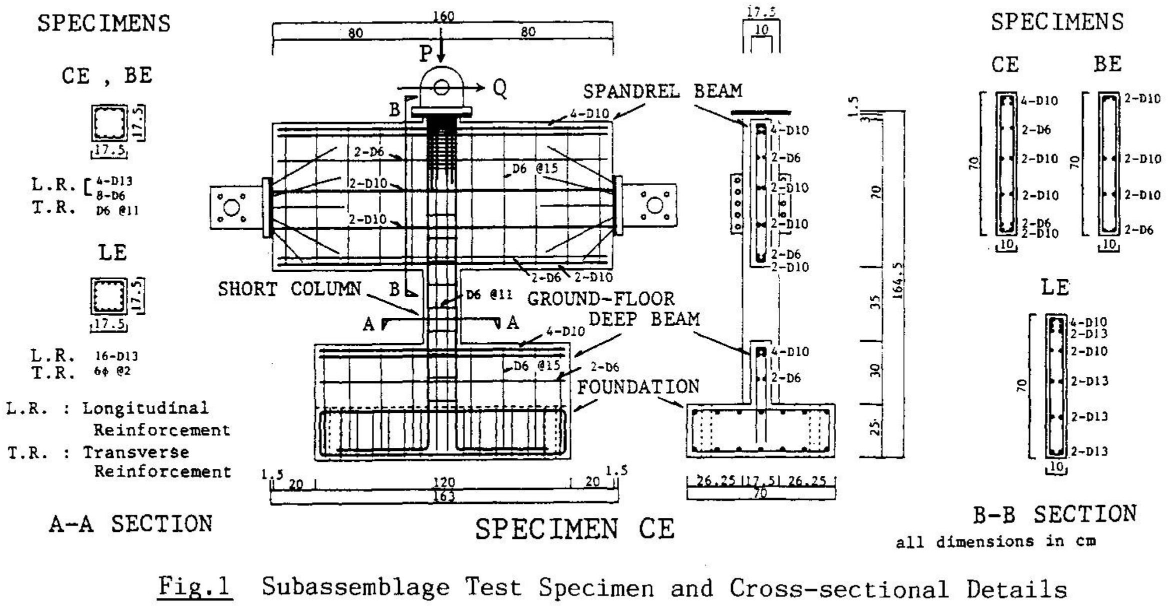

Fig.1 Subassemblage Test Specimen and Cross-sectional Details

In addition to these two specimens, Specimen LE was constructed to investigate validity of a repair and rehabilitation method proposed in this paper. In Specimen LE, quite large amount of longitudinal and shear reinforcements are provided in its short column, that is, p_g=6.64 percent and p_w=1.64 percent respectively. According to a preliminary analysis against severe earthquakes, both of the first-story columns of Specimens CE and BE had been expected to fail in brittle shear mode. In order to prevent these short-columns from shear failure during strong motion earthquakes, same specimens with the test Specimens CE and BE in Group E were constructed, and their first-story short columns were strengthened by a welded steel square tube, which are designated as Specimens CE-S and BE-S respectively. Since all of the tested short columns of the three specimens in Group E had failed in brittle shear modes and had not been able to develop their ultimate flexural moment capacities, damaged first-story short columns were repaired and rehabilitated also by using a welded steel square tube in order to recover the lost seismic capacity. The name of these specimens repaired has a letter "R" after the specimen number in Group E, such as CE-R and BE-R as shown in Table 1.

	Test Group					
	E (Existing)		S (Strengthening)		R (Repair and Rehabilitation)	
Specimen Number and Expected Failure Mode	CE	8.2 SF	CE-S	13.1 FF	CE-R	?
	BE	8.9 SF	BE-S	12.2 FF	BE-R	?
	LE	13.3 SF	LE-S	24.9 FF	LE-R	?

SF : Shear Failure in First-story Short Column
FF : Flexural Failure
○ : Location of Plastic Flexural Hinge

all dimensions in 10^4 N

Table 1 List of Test Specimens and Expected Failure Modes

3. STRENGTHENING METHOD (TEST GROUP S)

Strengthening method and procedure provided into first-story short columns in the test Group S is in the following:
(1) Machine a flat mild-steel plate into L-shape plate by using a press-machine. Steel plates used are 6 mm in thickness. (2) Weld each corner of faced two L-shape plates to form a square tubular section as shown in Fig.2. Clearance between steel tube and short column surface is approximately 5 mm. (3) After fixing the steel tube by spacers, top and bottom of the welded steel tube were sealed by inorganic sealer allowing no liquid leakage, and at the same time, aluminium pipes with 10 mm diameter were buried in the top and bottom sealing materials. (4) By using the bottom aluminium pipes, inject epoxy-based polymer cement under pressure into the clearance between steel tube and R/C column surfaces. Role of the top aluminium pipes is to exhaust the air from the clearance during injection of polymer cement. (5) After curing, cut off the inorganic sealer at top and bottom of the strengthened short column. This is to make the steel tube not to carry longitudinal stresses but only to carry transverse stresses during lateral loading reversals. Compressive strength of the polymer cement injected was 28.5 MPa at the time of experiment. Provided that shear failure does not occur in those strengthened short columns during lateral loading reversals, failure mechanism at their ultimate state of the Specimens CE-S and BE-S become ductile flexure modes, that is, column mechanism in CE-S and beam mechanism in BE-S respectively as shown in Table 1.

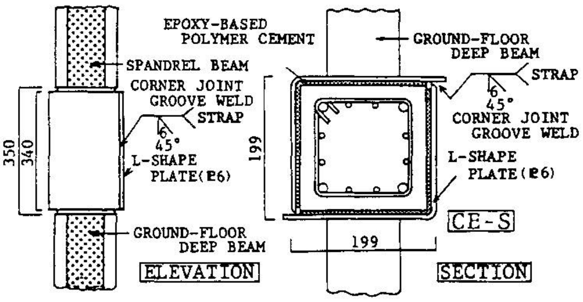

Fig.2 Details of Strengthened Short Column

4. REPAIR AND REHABILITATION METHOD (TEST GROUP R)

Repair and rehabilitation technique provided into damaged short columns in the test Group E is as follows (see Fig.3): (1) Remove residual interstory displacement occurred in the first-story short column. (2) Cut off cracked cover concrete around core of each short column. (3) Surround the naked short column with a welded steel square tube as mentioned in the strengthening method in the test Group S. (4) After sealing bottom of the steel tube by inorganic sealer and burying aluminium pipes in the sealer, put round coarse aggregate into clearance between steel tube and concrete core of the short column. Maximum size of the coarse aggregate is 10 mm in diameter. (5) Seal the top of steel tube and also bury the aluminium exhaust pipes. (6) Inject epoxy-based polymer cement under pressure from the bottom pipes. (7) After curing the polymer cement more than two-weeks, cut off the top and bottom sealer. Compressive strength of the polymer concrete was 25.3 MPa at the time of experiment. (8) By using epoxy resin adhesive (epoxy-based putty adhesive), repair and recover the cracked and lost sections in cover concrete near the top and bottom of short column (see Fig.3).

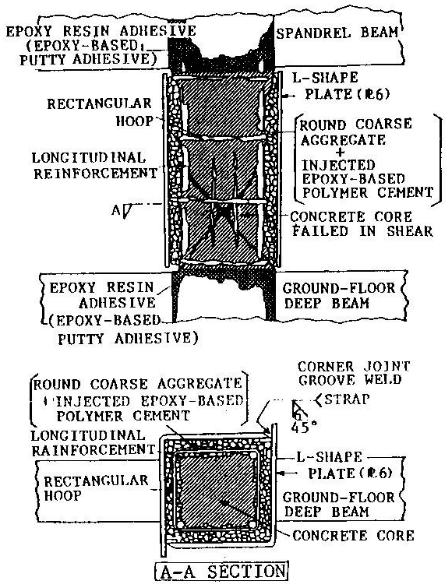

Fig.3 Details of Repaired Short Column

5. EXPERIMENTAL TEST-SETUP AND PROCEDURE

All of the specimens were tested by using a test setup shown in Fig.4, in which all of boundary conditions required in testing such types of the cruciform beam-column subassemblages as shown in Fig.1 are taken into consideration. Details of this test setup are discussed in Ref.1. Axial load to the column and alternately repeated lateral forces were applied at mid height of a second-story short column by using hydraulic jacks 1 and 2 in Fig.4. Since vertical reactions and displacements at left and right supports of the spandrel beam should be always kept equal respectively, "VERTICAL REACTION AND DISPLACEMENT EQUALIZER" is installed, and by using "MOMENT AND ROTATION EQUALIZER", bending moments and rotation angles at the left and right beam supports can be equalized, respectively. All of the tests except for Specimens, CE-S and CE-R, were conducted under a constant gravity load: $P/(A_c f_c) = 0.1$, where P, A_c and f_c are axial load, gross-area of column and compressive strength of concrete in each specimen, respectively. Value of the applied gravity load is a corresponding value of axial load to which first-story columns of low- and medium-rise school buildings are

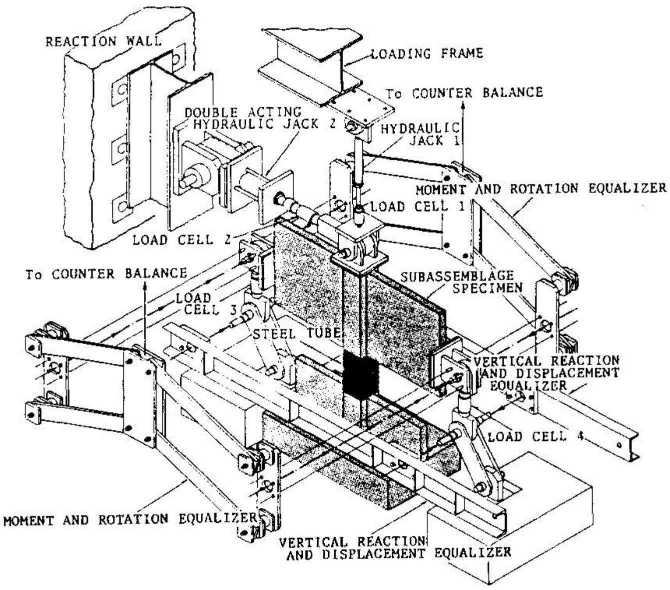

Fig.4 Experimental Test-Setup

subjected. In the Specimens, CE-S and CE-R, initial values of column axial load, $P/(A_c f_c) = 0.1$, varied and increased to about 0.2 at larger lateral displacements because of elongation of short columns due to cracking (refer to Figs.6(a) and 7(a)). Displacement-controlled procedure was adopted for loading program and lateral displacement amplitude of each loading cycle was gradually increased.

6. EXPERIMENTAL RESULTS

Applied lateral-load,(Q) versus interstory displacement,(Δ) relations observed in all the specimens are shown in Figs.5, 6 and 7. Fig.5 shows that all test specimens in Group E failed in brittle shear mode when interstory displacements in the first-story was not larger than 0.5 % rad., and these specimens were not able to develop their ultimate flexural moment capacities, which are determined by a theory and are shown by solid lines parallel to a horizontal axis in each figure. On the contrary, specimens in Group S whose short columns were strengthened by a steel square tube did not fail in brittle shear mode but were able to reach to ductile flexure mechanism showing their ultimate moment capacities as shown in Fig.6.

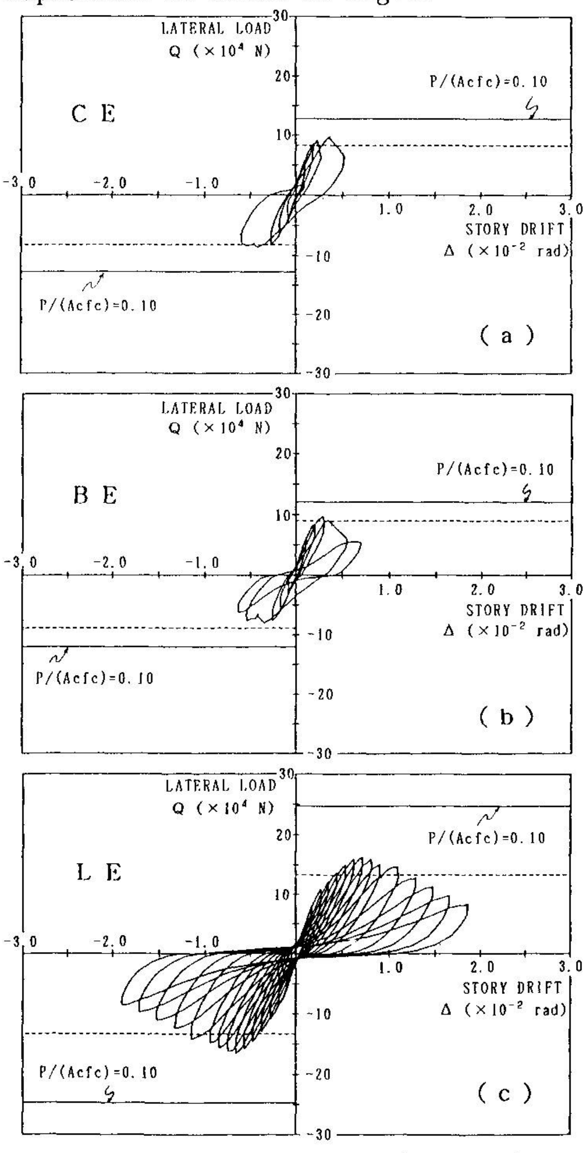

Fig.5 Q-Δ Relations (Group E)

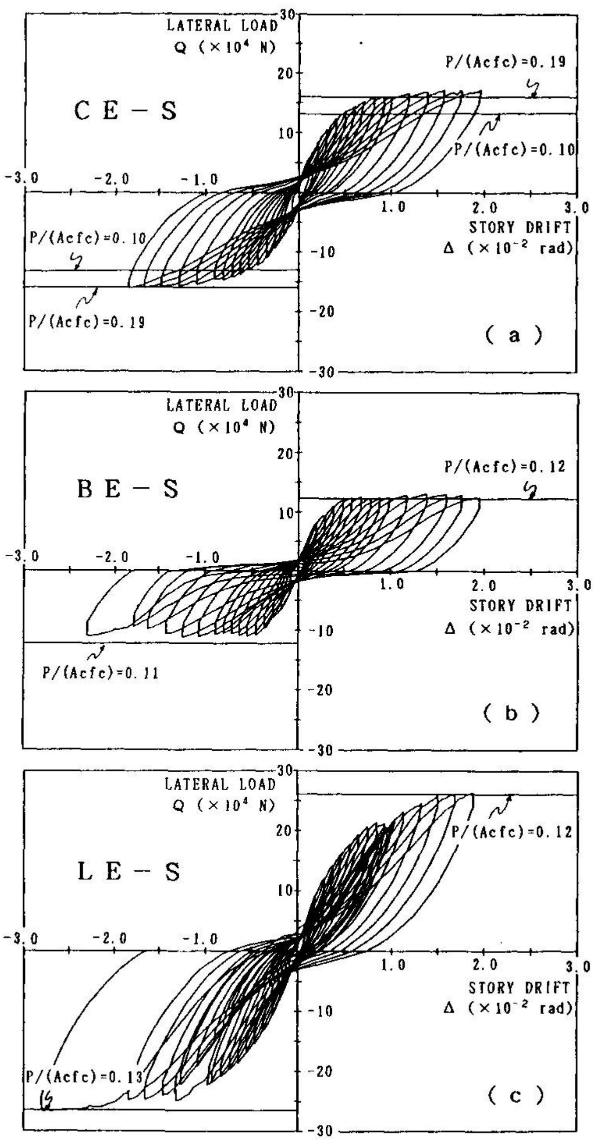

Fig.6 Q-Δ Relations (Group S)

It is worthy of note that, by repairing and rehabilitating the damaged short columns in Group E specimens by a welded steel square tube, repaired specimens (Group R) did no longer fail in brittle shear mode and showed large ductility. Although the short column in Specimen, LE-R, could not develop its ultimate flexural moment capacity, it was able to show excellent and ductile deformability as shown in Fig.7(c). Main reason to this cause is by a local bond deterioration occurred in the longitudinal reinforcement bars at top and bottom of the short column. In addition, these repaired specimens, except for Specimen, LE-R, were able to develop their ultimate flexural moment capacities as can been seen in Figs.7(a) and (b), and furthermore, these repaired specimens showed very close load-carrying capacities to the strengthened specimens shown in Figs.6(a) and (b), respectively. As the result, it may be concluded that the proposed strengthening, and/or repair and rehabilitation method by a steel tube is quite effective to improve the seismic behavior of R/C short columns which are expected to fail or damaged in brittle shear mode during severe earthquakes.

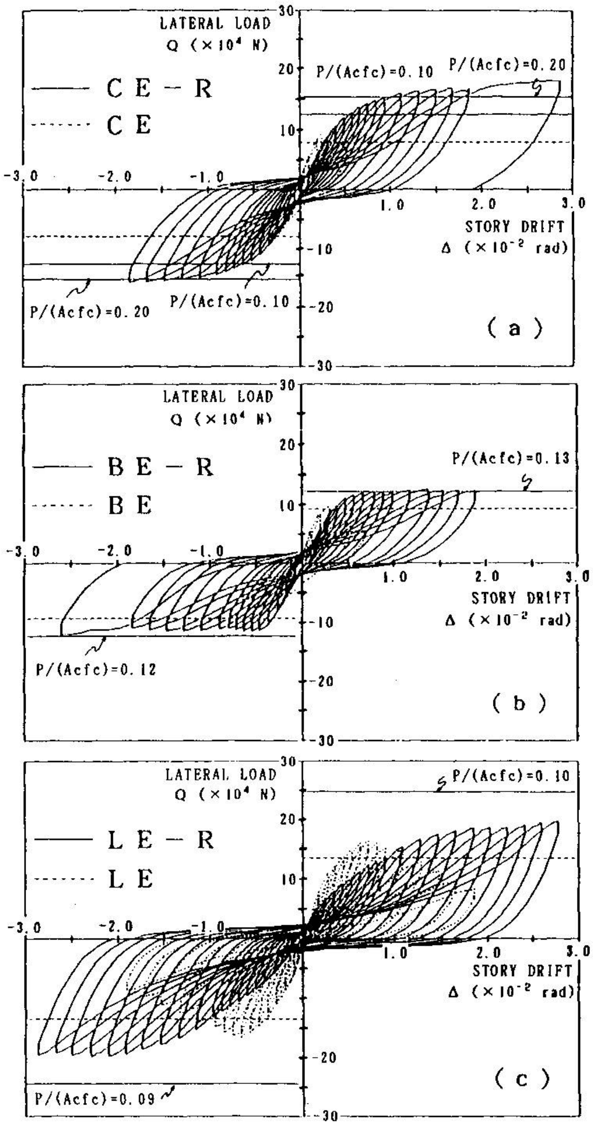

Fig.7 Q-Δ Relations (Group R)

7. CONCLUSIONS

By using a welded steel square tube, strengthening method for R/C short columns which are expected to fail in brittle shear mode was proposed, and validity of the proposed method to improve the seismic behavior of R/C short columns in existing building structures was verified by the experiment. In addition, this reinforcing method by using steel plates is also applicable to repair and rehabilitate the damaged short columns during strong motion earthquakes. This method proposed is quite practical because of being effective, inexpensive and very easy.

ACKNOWLEDGEMENTS

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Dauerhaftigkeit von hydrophobierten Sichtmauerwerksfassaden

Durability of Hydrophobed Facades of Brickwork

Durabilité de façades de maçonnerie hydrophobée

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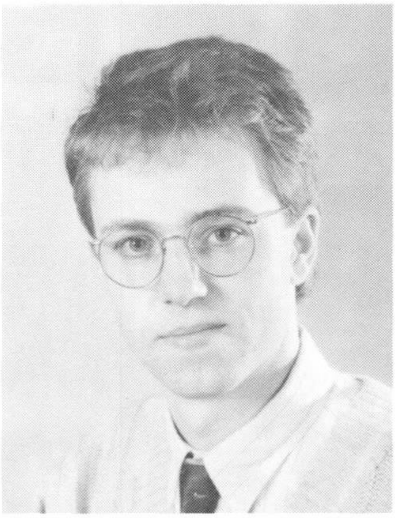

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ZUSAMMENFASSUNG

Die in letzter Zeit an Mauerwerksfassaden in zunehmendem Umfang festgestellten Schäden in Form von Abwitterungen, Abblätterungen und Absprengungen infolge Frosteinwirkung sind häufig auf nicht fachgerecht ausgeführte Hydrophobierungen zurückzuführen. Es werden vier derart geschädigte Gebäude beschrieben und die Schadensursachen und -mechanismen diskutiert. Vor einer Hydrophobierung kann jedoch in einfacher Weise mit Hilfe einer speziellen Prüfplatte zuverlässig festgestellt werden, ob diese Behandlung sinnvoll ist und welche baulichen Massnahmen gegebenenfalls. getroffen werden müssen.

SUMMARY

Damage to the facades of brickwork like weathering, scaling and spalling, which could be detected recently to an increasing extent due to the influence of frost, are often traced back to a non-professional treatment with hydrophobing agents. Four buildings with such defects are described and the causes and mechanisms of damage are discussed. Before treating a facade, one can determine easily with the help of a special test plate, whether the handling with hydrophobing agents is suitable and what kind of constructional measure must be carried out, if any.

RÉSUMÉ

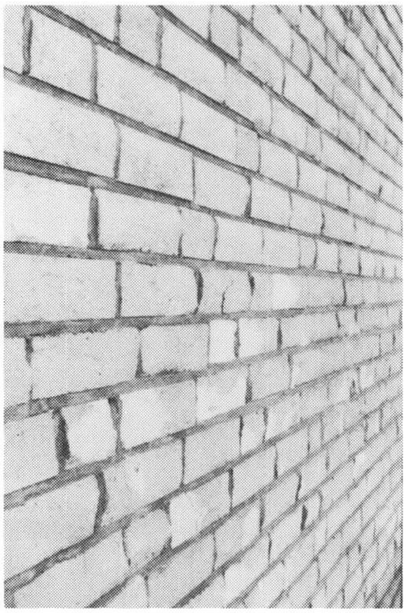
Les dommages causés aux façades peuvent être liés à l'action des intempéries, à l'écaillage ou à l'exfoliation, suite au gel. Toutefois, ces dégats sont souvent dûs à un traitement inadapté, par l'utilisation abusive de produits hydrofuges. On décrit le cas de quatre bâtiments ayant subi de tels dommages et où les causes et les mécanismes sont étudiés. Avant un traitement de façade, on peut constater simplement, avec une plaque d'essai, si des mesures préventives sont raisonnables et lesquelles devront être prises, le cas échéant.

1. GEBÄUDEBESCHREIBUNGEN, SCHADENSBILDER UND UNTERSUCHUNGSERGEBNISSE

Beispielhaft werden vier Sichtmauerwerksfassaden, drei Ziegel- und eine Kalksandsteinfassade mit Schäden in Form von Abplatzungen, Abblätterungen und Abwitterungen, die infolge einer nicht fachgerechten Hydrophobierung hervorgerufen wurden, gezeigt.

1.1 Kalksandsteinfassade (Bild 1 und 2)

Die Außenwände sind eine zweischalige Mauerwerkskonstruktion ohne Luftschicht. Die äußere Schale besteht aus KS-Struktur-Vormauersteinen mit einer bruchrauhen Steinoberfläche. Aus architektonischen Gründen wurden die Fugen zum Bauzeitpunkt bis auf eine Tiefe von ca. 10 mm ausgekratzt. Wahrscheinlich nach dem Auftreten von Durchfeuchtungen wurde die Fassade mit einem Zementmörtel nachträglich verfugt, ohne auf eine Mindesttiefe von 15 mm, wie es die DIN 1053 fordert, ausgekratzt zu werden. Da auch diese Maßnahme die Durchfeuchtungen nicht aufzuheben vermochte, wurde eine Hydrophobierung einige Jahre später durchgeführt. Im folgenden Winter stellten sich dann die in Bild 1 gezeigten Abplatzungen (die verwendeten Vormauersteine haben nach DIN 106 in der Prüfung auf Frostwiderstandsfähigkeit nur 25 Frost-Tau-Wechsel, im Gegensatz zu Verblendern mit 50 Frost-Tau-Wechseln, zu überstehen) ein. Bild 2 zeigt die Fugen im Detail. Sie sind zum großen Teil locker, teilweise herausgefallen und zum Zeitpunkt der Begutachtung vereinzelt sogar mit Moos bewachsen, was auf eine ständige starke Durchfeuchtung hinweist. Weiterhin sind Fugenablösungen von den Steinen festzustellen mit Rißbreiten größer als 0,5 mm.



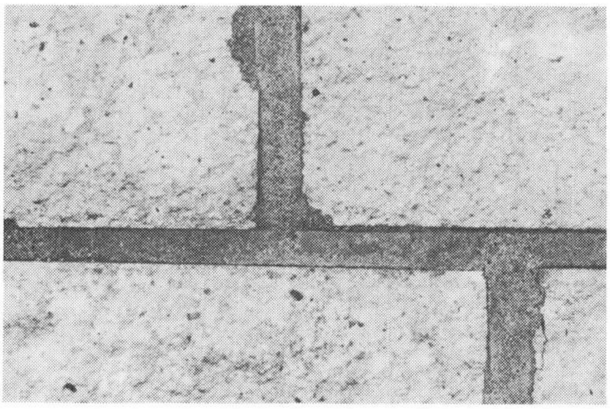
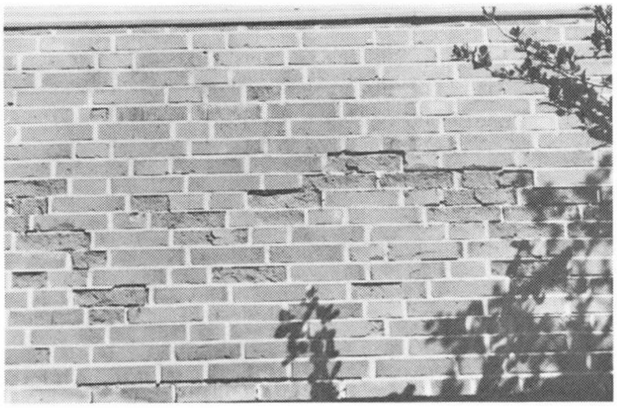
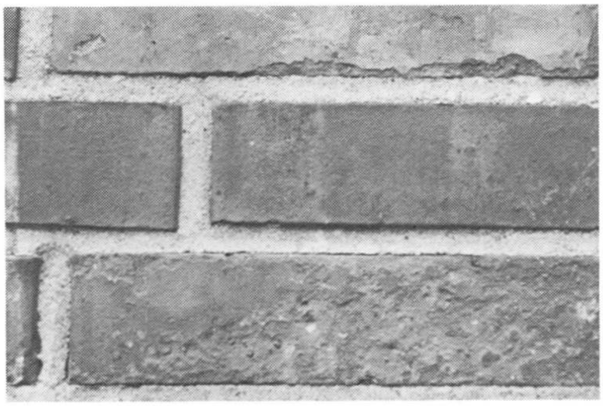


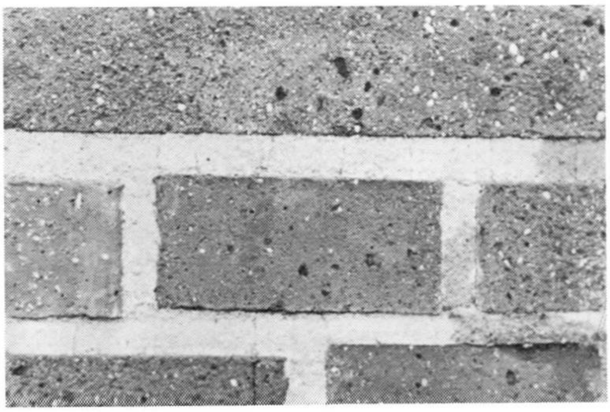
Bild 1: Fassade A mit Frostabplatzungen
Bild 2: Fassade A in Nahaufnahme mit Flankenablösungen

1.2. Ziegelfassaden

Bei den Fassaden der in Bild 3 und 4 dargestellten Gebäude handelt es sich um einschaliges Verblendmauerwerk. Die Steine wurden bei einer Prüfung nach DIN 52 252 Teil 1 (Tauchversuch) als nicht ausreichend frostbeständig eingestuft. Vor etwa 15 Jahren wurde die Fassade B aus Bild 3 mit einem Hydrophobierungsmittel, laut Datenblatt ein Polysiloxan, behandelt, und die







Fassade C, dargestellt in Bild 4, ebenfalls vor etwa 15 Jahren hydrophobiert. Bild 5 und 6 zeigen ausgewählte Bereiche der Fassaden B und C in Nahaufnahme. Neben Abplatzungen mit 5 bis 10 mm Scherbendicke sind deutliche Abblätterungen des Steinmaterials und starke Fugenschäden zu erkennen.

Eine weitere stark geschädigte Ziegelfassade ist in Bild 7 dargestellt. Hier wurde im Labor ebenfalls eine ausgeführte Hydrophobierung nachgewiesen. Es ist gut zu erkennen, daß die Ziegeloberfläche feine Risse aufweist. Ferner zeigten sich nach dem Abplatzen von Schalen dunkle Kerne im Inneren der Steine. Dieses läßt auf Brennfehler, verbunden mit verminderter Frostbeständigkeit, schließen.

Bild 3: Fassade B mit großflächigen Abwitterungen und Abblätterungen

Bild 4: Fassade C mit Frostabplatzungen und Abblätterungen

Bild 5: Fassade B in Nahaufnahme mit Flankenablösungen und Abwitterungen der Ziegeloberfläche

Bild 6: Fassade C nach der Reinigung mit Hochdruckwasserstrahlen; Risse in den Mörtelfugen

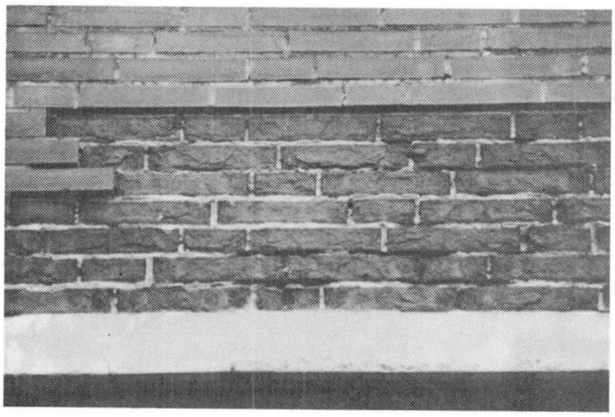
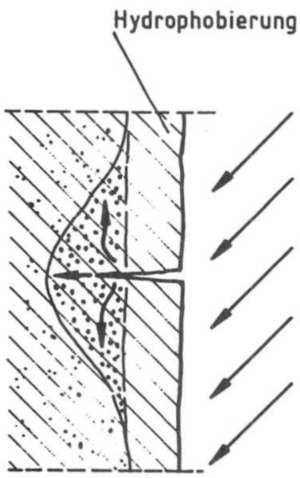


Bild 7: Fassade D mit starken Frostschäden

2. SCHADENSURSACHEN UND -MECHANISMUS

Aus Abschnitt 1 geht hervor, daß die Steine der geschädigten Fassaden entweder als nicht frostwiderstandsfähig eingestuft wurden oder zumindest für eine exponierte Lage, wie sie bei dem Gebäude mit Kalksandsteinfassade vorliegt, nicht geeignet sind. Die Schäden sind jedoch in erster Linie nicht auf die mangelnde Frostbeständigkeit, als auf eine unfachmännische Hydrophobierung zurückzuführen. Zum einen weisen die hydrophobierten Bereiche besonders starke Schäden auf. Dieses läßt sich aus der Gegenüberstellung von hydrophobierten und nicht hydrophobierten "Referenzflächen gleicher Orientierung und Ausführung eindeutig folgern. Zum anderen wirkt sich erfahrungsgemäß eine mangelhafte Frostbeständigkeit nur an besonders exponierten Stellen, undichten Fallrohranschlüssen oder bei hohem Spritzwasseranfall aus. Bei den vier beschriebenen Mauerwerksfassaden waren die Fugen in einem sehr unbefriedigendem Zustand. Es konnte Wasser durch Risse in den Fugen oder sogar Risse in den Steinen und Flankenablösungen oder durch Kapillarwirkung in die saugfähigen oder bereits wieder saugfähigen Fugen in das Mauerwerk oder infolge ungleichförmiger Imprägnierung, hinter die behandelte Oberfläche der Steine gelangen. Dieser Mechanismus wird schematisch in Bild 8 gezeigt. Die Trocknung hydrophobierter Fassaden kann nur noch durch Dampfdiffusion stattfinden, die um Größenordnungen kleiner ist als die Wasserabgabe infolge Verdunstung und Kapillarwirkung /1/. Somit kann es insbesondere bei hydrophobierten Fassaden mit Rissen, die durch die Hydrophobierung nicht abgedichtet werden können oder bei undichten Fugen zu einem Ansteigen des Wassergehaltes kommen, und unweigerlich stellen sich bei frostempfindlichen Steinen nach Überschreiten eines kritischen Wassergehaltes Frostschäden ein. Häufig treten die ersten Frostschäden bei Fassaden, die zum Zeitpunkt der Maßnahme in oben beschriebener Weise ungeeignet sind, bereits beim nächsten intensiven Frost nach starken vorhergegangenen Regenfällen auf (Fassade A). Falls ungeeignete Hydrophobierungsmittel verwendet werden oder resistentere Ziegel vorliegen, können die Schäden auch mit mehrjähriger Verzögerung sichtbar werden (Fassaden B und C).



Feuchtigkeitsanreicherungen und Frostgefahr bei Wasserdurchtritt durch Fehlstellen.

Bild 8: Mechanismus der Wasseraufnahme durch Fehlstellen in der Fassade

3. UNTERSUCHUNGSMETHODEN

Für die Beurteilung von hydrophobierten Mauerwerksfassaden durch Wasseraufnahmemessungen wurde eine spezielle Prüfplatte entwickelt (Bild 9) /2/. Sie überdeckt die Fläche, die sich ergibt aus der Länge eines Normalformatziegels und der dazugehörigen Stoßfugendicke (also 250 mm) multipliziert mit der Ziegeldicke und der dazugehörigen Lagerfugendicke (also 81 mm). Die Platte wird lediglich mit einem Kittband an das Mauerwerk angeklebt. Es können verschiedene Stärken von Schlagregen simuliert werden, indem die Druckhöhe variiert wird. Mit dieser Prüfplatte kann recht zuverlässig die Durchlässigkeit des Mauerwerks bei Schlagregen abgeschätzt werden. Umfangreiche Untersuchungen und Berechnungen, wie die Ermittlung des Wasseraufnahme- und Abgabeverhaltens von hydrophobierten Mauerwerksfassaden und die Betrachtung des statistischen Wettergeschehens, ergaben Anhaltswerte für die Wassermengen, die maximal von einer hydrophobierten Sichtmauerwerksfassade über die Fugen aufgenommen werden dürfen.

Die Wasseraufnahme einer Mauerwerksfassade bei Schlagregen bzw. im Plattenversuch wird charakterisiert durch die Beziehung:

$$W_{ges} = W_{St} + W_F \quad \text{mit} \quad W_{St} = \overline{w}_{St} \cdot A_{St} \cdot \sqrt{t}$$

$$W_F \approx \overline{w}_M \cdot A_F \cdot \sqrt{t} + \overline{w}_F \cdot p \cdot A_F \cdot t \quad .$$

Nach einer Hydrophobierung nehmen die Steine und der Fugenmörtel kein Wasser mehr auf, lediglich durch Haarrisse und Flankenablösungen kann bei Schlagregen weiter Wasser durch die Fugen eintreten.

Eine Wasseraufnahme der Fassade läuft dann nach Nullsetzen der Wasseraufnahmekoeffizienten w_{St} und w_M ab nach der Beziehung:

$$W = \overline{w}_F \cdot p \cdot A_F \cdot t \quad ,$$

d. h. sie ist proportional dem Wasseraufnahmekoeffizienten der Fugen $\overline{w}_F$ [$kg/(m^2 \cdot h \cdot Pa)$], dem Staudruck p [Pa], der Fugenfläche A_F und der Einwirkungsdauer t.

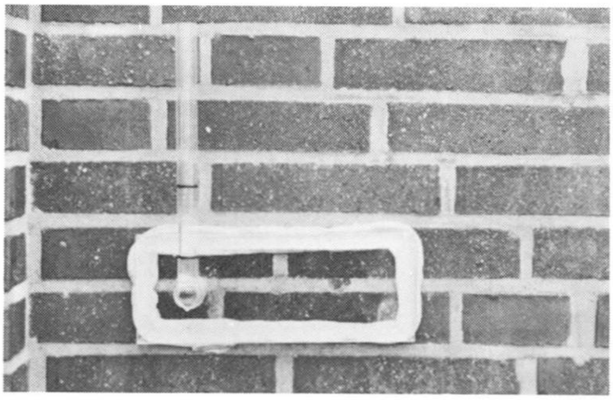


Bild 9: Spezielle Prüfplatte zur Messung der Wasseraufnahme von Fassaden

Die in vielen Messungen nachgewiesene lineare Abhängigkeit der Wasseraufnahme von hydrophobierten Mauerwerksfassaden mit undichten Fugen wurde wiederum bei Fassaden A und B bestätigt, vgl. Bild 10. Entsprechend den Ausführungen in /2/ können kritische Wasseraufnahmen für die Fugen von hydrophobierten Fassaden angegeben werden, bei deren Überschreiten die Gefahr einer zunehmenden Durchfeuchtung exponierter Fassadenbereiche besteht.

Unter Voraussetzung einer mittleren Schlagregendauer von 1 h je Tag in den Monaten November bis Februar kann man mit den Wetterdaten im norddeutschen Raum (Schlagregengruppe III) die kritische Wasseraufnahme über die Fugen hydrophobierter Fassaden zu 0,2 $kg/(m^2 \cdot h)$ abschätzen, gemessen bei einer Druckhöhe von 50 mm Wassersäule. Bei einer 15 min Messung mit der dargestellten Prüfplatte sollte daher die Wasseraufnahme nicht höher als 1,0 g sein.

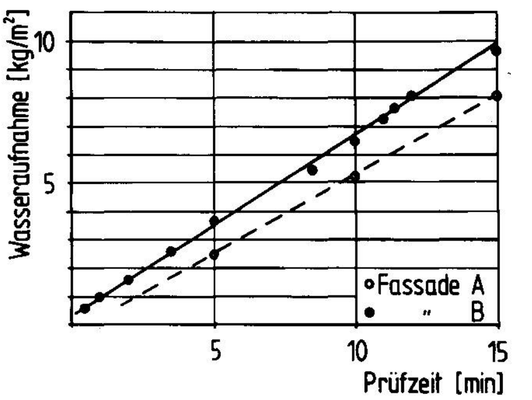


Bild 10: Wasseraufnahmen mit der Prüfplatte an hydrophobierten Sichtmauerwerksfassaden mit Haarrissen in den Fugen, Druckhöhe 100 mm

4. BEMERKUNGEN ZUR INSTANDSETZUNG VON SICHTMAUERWERK

Für die Entscheidung, ob bei durchfeuchteten bzw. geschädigten Fassaden Hydrophobierungen durchgeführt werden sollten, ist es von Bedeutung die Hydrophobierbarkeit an Testflächen grundsätzlich zu überprüfen. Hierfür kann mit Hilfe der Prüfplatte getestet werden, wie sich die Fassaden bei Schlagregenbeanspruchung bezüglich der Wasseraufnahme verhalten. Ein Grenzwert ist im Abschnitt 3 angegeben.

Weiterhin hat der Grad der Schädigung einen entscheidenden Einfluß auf die Instandsetzungsmaßnahmen. Bei Fassade C und D konnte die Instandsetzungslösung schon aus optischen Gründen nur in einer großflächigen Auswechselung der hydrophobierten geschädigten Bereiche bestehen. Die Steine von Fassade A konnten nach einer Reinigung mit Hochdruckwasserstrahl wegen der ohnehin bruchrauhen Oberfläche nahezu ganz erhalten werden. Ebenso konnte mit Fassade B verfahren werden, da hier Abblätterungen und Absprengungen nur geringer Dicke vorlagen.

Die primäre Ursache der Frostschäden an den hydrophobierten Fassaden ist in dem unbefriedigenden Fugenzustand zu suchen, welcher in einigen Fällen erst nach der Reinigung der Fassade gut sichtbar wird (Fassade C). Wegen der Bedeutung von intakten und dauerhaften Fugen für die Dauerhaftigkeit der gesamten Fassade müssen unter Umständen die gesamten Mörtelfugen der geschädigten Bereiche mindestens bis zu der erforderlichen Tiefe von 15 mm freigelegt werden und sollten anschließend mit einem geeigneten Fugenmörtel (in der Regel Werktrockenmörtel) neuverfugt werden. Erst nach einer Mindesterhärtungszeit von vier Wochen kann sich eine erneute Hydrophobierung der gesamten Fläche mit einem geeigneten, d.h. in der Regel alkalibeständigen und auf Ziegeluntergrund dauerhaftem Hydrophobierungsmittel anschließen. Die erforderliche Hydrophobierungsmittelmenge sollte, wie die Hydrophobierbarkeit und die geeignete Reinigungsart, auf Testflächen am Bauwerk ermittelt werden. Schließlich kommt der Güte der Ausführung aller Schritte eine entscheidende Rolle für die Wirksamkeit und Dauerhaftigkeit der Instandsetzungsmaßnahmen zu.

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Corrosion et anticorrosion des ouvrages en acier

Korrosion und Korrosionsschutz bei Bauwerken aus Stahl

Corrosion and Anti-Corrosion Protection in Steel Structures

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RÉSUMÉ

La pérennité des ouvrages d'art en acier dépend de l'agressivité du milieu et du type de protection anticorrosion qui a été mis en oeuvre. En France, une règlementation technique précise les techniques et les moyens permettant d'assurer des protections de longue durée. Deux enquêtes montrent que les dépenses d'entretien peuvent être estimées et que les périodicités de remise en peinture sont de l'ordre d'une quinzaine d'années.

ZUSAMMENFASSUNG

Die Dauerhaftigkeit von Bauwerken aus Stahl hängt von der Aggressivität der Umgebung und vom Korrosionsschutz ab. In Frankreich beschreibt ein Reglement die Methoden zur Sicherstellung eines langandauernden Korrosionsschutzes. Anhand von zwei Untersuchungen wird gezeigt, dass die Unterhaltsanwendungen abgeschätzt werden können und dass die Anstriche etwa alle 15 Jahre zu erneuern sind.

SUMMARY

The durability of steel structures depends on the aggressivness of the environment and the anti-corrosion protection. In France there is a design standard which describes the available methods for an effective longterm protection against corrosion. Based on two investigations it is shown that maintenance works can be estimated and that repainting is necessary about every fifteen years.

1. IMPÉRATIFS TECHNIQUES ET RÉGLEMENTAIRES

La protection anticorrosion des ouvrages aériens en acier demande un niveau de qualité très important afin d'obtenir la durée de vie la plus longue possible et, ainsi, de minimiser les coûts et les gênes d'exploitation apportés par les opérations d'entretien.

En France, les protections anticorrosion des ouvrages métalliques font appel, soit à des revêtements métalliques recouverts ou non de peinture, soit à des systèmes de peinture appliqués directement sur l'acier.

La conception d'un système de protection sur un ouvrage neuf doit toujours être réalisée dans l'optique des entretiens ultérieurs. Ces entretiens devront être les plus espacés et les plus simples possible, d'autant plus que l'environnement de l'ouvrage doit être le moins perturbé et pollué.

1.1. Le Fascicule réglementaire "Protection des ouvrages métalliques contre la corrosion".

Pour atteindre ces objectifs de qualité et de durabilité de la protection, une réglementation technique interministérielle a été mise en place en France. En particulier, pour les marchés publics de travaux le fascicule N°56, Protection des ouvrages métalliques contre la corrosion, permet aux Maîtres d'Ouvrages de spécifier des systèmes de protection, de s'assurer de leur mise en oeuvre correcte et de choisir un niveau de garanties compatible avec les exigences de l'ouvrage et de l'environnement

Ce fascicule concerne les structures métalliques de génie civil ou assimilées et les structures en câbles. Il s'applique aux travaux neufs et aux travaux d'entretien.

A partir des niveaux de garantie souhaités, il permet au Maître d'Ouvrage de sélectionner un type de protection compatible avec la catégorie de l'ouvrage et avec l'ambiance dans laquelle il se situe.

La garantie caractérise en fait la qualité de la protection apportée par le procédé choisi et comprend :

- la garantie anticorrosion basée sur les clichés de l'Echelle européenne de degrés d'enrouillement pour peinture antirouille.
- la garantie d'aspect basée sur l'absence d'altérations du revêtement et de sa couleur en référence aux normes existantes.

Les différentes garanties correspondent aux procédés suivants :

- galvanisation à chaud,
- zingage électrolytique,
- mise en peinture,
- métallisation par projection à chaud suivie de mise en peinture,
- galvanisation suivie de peinture,

et s'appliquent :

- aux ouvrages aériens en atmosphères rurales, urbaines,industrielles et maritimes,
- aux ouvrages immergés en eau douce ou en eau agressive.

Le document comporte pour chaque procédé des tableaux donnant par catégorie d'ouvrage et par type de préparation de surface, différents niveaux de garantie allant de 7 ans à 2 ans.

1.2. Utilisation de systèmes de peintures homologués.

Pour le cas de la mise en peinture, l'utilisation par le Maître d'Ouvrage d'un système homologué par la commission interministérielle d'homologation permet d'obtenir un niveau de garantie de 7 ans au cliché Ri 1 pour les ouvrages d'art aériens, quelles que soient les ambiances.

Il existe un recueil des systèmes de peintures homologués, classés par ambiance.
Cette homologation repose sur une aptitude à la fonction du système et non sur un critère de formulation chimique des différentes peintures composant le système.

Le fabricant soumet à la commission un système qui est examiné à la fois au niveau de ses performances en essais de laboratoire et au niveau de ses références d'emploi en vraie grandeur.

Le plan d'assurance qualité de l'usine de fabrication est étudié par la commission et, dans la mesure ou le système est homologué,le fabricant s'engage à assurer un contrôle externe de ses fabrications et à ne pas modifier la formulation d'un des composants sans en demander préalablement l'autorisation.

L'utilisation d'un système homologué avec une durée de garantie de 7 ans cliché Ri 1 conduit à une durée de vie efficace de la protection comprise en fait entre 15 et 20 ans ,soit 2 à 3 fois la durée de garantie contractuelle.

1.3. Préparation de surface avant peinture.

Bien entendu ces durées de vie sont obtenues avec une exécution correcte des travaux tant pour la préparation de surfaces que pour l'application des produits.

La préparation de surface doit être telle que l'on obtienne un support totalement débarrassé de traces de salissures, souillures diverses, et, bien entendu, des différents oxydes tels que calamine et rouille.

Il est nécessaire d'utiliser un décapage par projection d'abrasifs avec un degré de soin et un degré de rugosité adaptés à la nature du primaire anticorrosion choisi.Cette préparation de surface peut être réalisée par grenaillage en atelier ou encore par décapage sur le site avec des abrasifs tels que corindon ,laitiers, etc., avec un rendement plus faible et des risques de pollution importants pour l'environnement.

Une autre technique est l'utilisation de produits grenaillés et peints de façon automatique (PGP), qui sont des produits industriels, normalisés, qui offrent une qualité de préparation de surface répondant aux exigences des textes réglementaires.

1.4. Application des peintures.

L'application des couches de protection demande un très grand soin et doit suivre un certain nombre de règles :

- respect du délai entre décapage et couche primaire, délai qui est fonction des conditions atmosphériques;
- respect des conditions d'application de chacun des produits constituant le système :
    - conditions atmosphériques,
    - mode d'application,
    - durée de séchage,
    - épaisseur sèche,
    - délais de recouvrement,
- respect du délai de mise en service après application de la dernière couche.

Le plan d'assurance qualité de l'entreprise d'application est donc une pièce fondamentale pour le respect de tous les critères énumérés ci -dessus.
En particulier, les contrôles internes prévus doivent permettre de s'assurer de la régularité de la préparation de surface et du respect des épaisseurs sèches prévues pour chacune des couches du système homologué.
Ce système d'homologation préalable des produits et de plans d'assurance qualité de l'entreprise d'application, ne dispense toutefois pas d'un contrôle extérieur, de la part du Maître d'Ouvrage, qui s'exerce par sondages.
Cette procédure qui était réservée jusqu'en 1985 traditionnellement aux grands ouvrages d'art, ponts ,portes d'écluses, etc., s'étend actuellement à des ouvrages privés pour lesquels on recherche un niveau de protection contre la corrosion particulièrement performant.

2. PÉRENNITÉ, ENTRETIEN ET COUTS.

L'acier est largement utilisé pour la construction des ouvrages d'art, mais il rencontre parfois certaines objections relatives aux sujétions de protection et d'entretien.L'opinion souvent répandue - et non fondée - selon laquelle il faut repeindre fréquemment les ouvrages métalliques peut être mise en défaut.

Deux enquêtes récentes auxquelles l'OTUA a participé, permettent de balayer les idées reçues et d'avoir des données plus réalistes sur les coûts et les périodicités d'entretien.

La première, effectuée auprès des organismes gestionnaires d'ouvrages maritimes et portuaires montre que les dépenses d'entretien sont parfaitement estimables et que les délais préconisés entre les remises en peinture sont de l'ordre d'une quinzaine d'années.

La seconde, menée auprès des principales administrations gérant des ponts métalliques (routes, chemins de fer, métropolitains.....) montre que les temps moyens entre deux remises en peinture sont de l'ordre de 21 à 23 ans.

Les deux enquêtes font état des coûts moyens des protections anticorrosion appliquées, qui sont de l'ordre de 3,5 % du coût total des ouvrages neufs.

2.1 Enquête OTUA.

Pour répondre aux objections formulées sur la corrosion des ouvrages métalliques et les coûts d'entretien qu'elle entraîne, l'OTUA a mené à bien une enquête auprès des responsables de l'entretien des installations des grands ports maritimes français.

En effet, la concentration exceptionnelle d'ouvrages d'art et d'engins métalliques dans les ports maritimes permet un test à vraie grandeur en matière d'entretien pour ces ouvrages et ces installations.

2.1.1 Déroulement de l'enquête

Au cours de l'année 1982, quatre ports importants ont été visités :

- La Rochelle,
- Marseille,
- Dunkerque,
- Le Havre.

Dans chacun d'eux, les questions suivantes ont été posées :

- Nature , consistance et périodicité des opérations d'entretien ,
- coût de ces opérations selon la nature des ouvrages, leur âge et les systèmes de protection mis en oeuvre.

Sans entrer dans le détail du questionnaire, on peut en souligner les rubriques principales qui portaient sur les points suivants :

- situation et conditions d'utilisation des ouvrages :
    - génie civil : ouvrages immergés
      ouvrages à l'air libre
    - outillage : engins maritimes
      engins à quai
    - ouvrages ou engins en service permanent
    - ouvrages ou engins en service intermittent

- prévention et protection anticorrosion
    - ouvrages existants :
        - surveillance
        - entretien
    - ouvrages neufs :
        - protection active,

- protection passive

- servitudes d'entretien liées aux conditions de service et d'accessibilité des ouvrages.

Il paraît intéressant de noter que la périodicité, et donc le coût, de l'entretien des ouvrages observés ont fortement varié dans le temps, suivant trois périodes :

- ouvrages réalisés avant 1940 pour lesquels les systèmes de peintures anticorrosion donnaient des durées de protection de l'ordre de 15 ans.
- ouvrages réalisés ou entretenus à partir de 1950 avec des peintures nouvelles apparues à cette époque et qui donnaient généralement des durées de protection réduites, ne dépassant pas 5 ans dans les cas les plus défavorables.
- période actuelle (à partir de 1965-1970), pour laquelle l'utilisation de systèmes de peintures agréés a permis de retrouver des durées de protection de 12 à 18 ans, voire davantage lorsqu'ils sont associés à des protections actives pour les ouvrages particulièrement exposés (protection cathodique en immersion).

2.1.2 Résultats de l'enquête.

- Eléments de durée

- Ouvrages existants.

Suivant l'importance des dégradations, l'implantation et l'accessibilité de l'ouvrage et sa fonction, l'entretien se fait - soit sous forme préventive par l'application d'une couche de peinture tous les 5 ans environ, - soit sous forme curative, par application d'un système homologué après décapage par projection d'abrasifs, tous les 10 à 15 ans environ.

D'après l'expérience des responsables d'entretien, il apparaît que pendant les périodes où les peintures à base d'huile de lin ont été remplacées par des peintures glycérophtaliques, les cycles d'entretien se sont nettement raccourcis, la durée de vie des revêtements dépassant rarement 5 années.Par contre, depuis l'utilisation de peintures à liants epoxydiques, les durées de vie des revêtements anticorrosion sont estimées à 10-12 ans (Dunkerque),voire 12-15 ans (Le Havre et Marseille).

- Ouvrages neufs.

Le choix des moyens de protection dépend de la situation des ouvrages et de leurs contraintes d'exploitation.

- Ouvrages immergés :

La protection anticorrosion est généralement assurée par l'application d'un système de peinture homologué, complété par un système de protection cathodique.
L'expérience montre que les durées d'efficacité de ces systèmes est de l'ordre de 15 ans (Le Havre et Marseille) et même de 18 ans (Dunkerque) entre deux cycles d'entretien.

- Ouvrages à l'air libre :

Les durées de vie des revêtements de protection sont estimées entre 10 et 15 ans pour les engins de levage à Marseille, et entre 18 et 20 ans pour les appareils et ouvrages à Marseille, dans les conditions suivantes :

- emploi de produits grenaillés et peints,
- stockage à l'abri des produits avant et après usinage,
- seconde préparation de surface et reconditionnement des zones affectées,
- application d'un système de peinture homologué.

2.2 Enquête du Comité Ponts Métalliques.

2.2.1 Déroulement de l'enquête.

Le groupe de travail Pérennité des Ouvrages, sur recommandation du Comité 2, a élaboré une fiche protection anticorrosion, dont les exemplaires complétés et retournés par les gestionnaires ont été ventilés dans les catégories suivantes :

ouvrages neufs	ponts-routes ponts-rails
ouvrages anciens	ponts-routes ponts-rails ponts-canaux

2.2.2 Résultats de l'enquête.

- Eléments de coût

Les coûts moyens pondérés de protection par m2 varient entre 64 F.pour les ouvrages aériens neufs et 90 F. pour les ouvrages anciens.

- Eléments de durée

Les temps entre deux remises en peinture de la totalité de la surface des ouvrages sont les suivants :

- Ponts-routes : de 18 à 27 ans, temps moyen de 23 ans
- Ponts-rails : de 10 à 42 ans, temps moyen de 21 ans.

Les périodicités de réfection de la protection anticorrosion préconisées par les gestionnaires varient de 5 ans (ponts mobiles en site maritime) à 20 ans, avec une position moyenne située à 14 ans, soit le double de la durée de garantie contractuelle habituelle (7 ans au cliché Ri1).

3. CONCLUSIONS

Il est important de noter que dans les deux enquêtes, les coûts et les durées de protection concordent pour les ouvrages d'art.De plus, il ressort que les dépenses d'entretien, pour un ouvrage donné, peuvent être estimées avec précision et que les opérations sont parfaitement programmables à long terme, puisque, en général, une remise en peinture complétée tous les 14-15 ans est préconisée par les différents gestionnaires.

Il convient également de souligner l'intérêt des réfections partielles de la protection anticorrosion qui permettent de retarder la réfection totale et d'éviter les réparations de parties qui seraient corrodées au moment de la remise en peinture générale.

Ces réfections partielles, qui peuvent être effectuées après chaque visite réglementaire, entraînent généralement une réduction des frais d'entretien des ouvrages.

Par ailleurs, les ouvrages modernes sont, de par leur conception, moins sensibles à la corrosion et plus faciles à protéger.Cette évolution associée à celle des techniques de protection contre la corrosion, permettra de minimiser les coûts d'entretien.

Tous ces éléments permettent de répondre aux objections relatives aux dépenses d'entretien des ouvrages en acier et à leur fréquence qui sont souvent présentées comme excessives. Il n'en est donc rien et la protection de l'acier, qui assure la pérennité des ouvrages tout en leur apportant un aspect esthétique certain, n'est pas plus onéreuse que l'entretien ou les réparations à posteriori des ouvrages réalisés avec d'autres matériaux.

Sauvegarde de monuments historiques grâce à la précontrainte

Erhaltung historischer Bauten dank Vorspannung

Preservation of Historic Monuments using Prestressing

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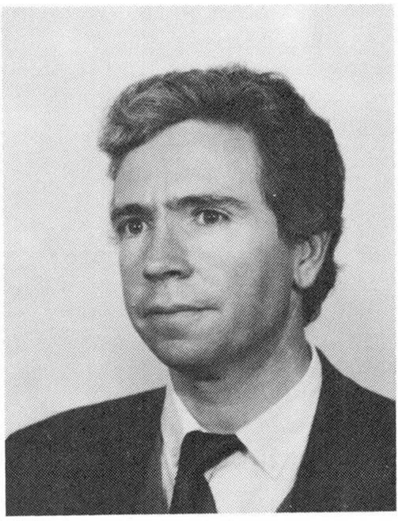


Né en 1951, obtient son diplôme d'ingénieur à l'Institut National des Sciences Appliquées de Toulouse. Depuis 12 ans chez Freyssinet il est responsable de L'agence Ile de France, et s'intéresse tout particulièrement aux techniques de renforcement de structures.

RÉSUMÉ

L'évolution constante des techniques de précontrainte extérieure déjà très utilisées dans le cas de renforcement de structures en béton armé ou précontraint leur permet aujourd'hui d'être appliquées aux ouvrages anciens en maçonnerie. L'idée de mise en charge active lors de reprise en sous oeuvre répond à un besoin croissant de confortement des édifices anciens. Ces techniques dynamiques nécessitent des études spécifiques, et une réalisation très méthodique. Le confortement de l'Arc de Triomphe à Paris illustre parfaitement l'application de ces techniques nouvelles au cas des édifices anciens.

ZUSAMMENFASSUNG

Die Technik der aussenliegenden Vorspannung ist derart ausgereift, dass sie auch auf historische Mauerwerksbauten angewandt werden kann. Die Vorspannung im Laufe von Instandstellungsarbeiten entspricht einem wachsenden Bedürfnis zur Erhaltung historischer Bauten. Diese neuen Techniken erfordern spezifische Untersuchungen und eine sehr methodische Anwendung. Das Beispiel des Triumphbogens von Paris zeigt bestens die Anwendung dieser neuen Technik bei Baudenkmäler.

SUMMARY

The constant development of exterior prestressing for structures in reinforced concrete facilitates their application to historic masonry structures. Prestressing during rehabilitation works responds to a growing need. This novel technique requires specific studies and a methodical realisation. The example of the 'Arc de Triomphe' in Paris illustrates perfectly the application of this new technique to ancient structures.

1. HISTORIQUE

L'Arc de Triomphe de l'étoile a été construit de 1805 à 1835. Napoléon voulu ériger l'Arc de l'Etoile à la gloire des armées de la Révolution immortalisées sous son commandement alors qu'il n'était que le Général BONAPARTE. Lentement construit d'après les plans de Chalgrin, la construction connu bien des hésitations. Lors de l'abdication de Napoléon en 1815, l'imposte de la grande arcade était posée avec la 45ème assise.
Louis XVIII ne reprit la construction qu'en 1824 sous la direction des Architectes GOULT et HUYOT. En 1830, Louis PHILIPPE reprit la pensée initiale de Napoléon au profit des armées républicaines. BLOUET succéda à HUYOT en 1833 et introduit dans le volume général la grande salle qui réhaussait de 8 m la terrasse terminale. Le monument mesure 45 m de longueur par 25 m de largeur et 50 m de hauteur, pour un poids total de 50000 tonnes.

2. PATHOLOGIE DE L'EDIFICE

2.1 Apparition des désordres

En 1985, quelques pierres se détachent de la voute et de la façade de l'Arc. Examen visuel permet d'identifier les fissures et d'en tracer le relevé. Début 1987, le Cabinet Michel BANCON spécialisé dans les études de structure et de la réabilitation des édifices anciens a été chargé d'une expertise du bâtiment afin de définir un programme de réabilitation.

2.2 Campagne de mesure et d'essai

Afin d'établir un diagnostic précis et déduire les origines du phénomène et la nature des travaux les plus rationnels, une série de mesures a été opérée :
- mesures de vibrations au sol et dans la partie supérieure,
- équipement des fissures et mesures de leur évolution,
- pose sur l'édifice de niveau de précision et suivi de leur évolution,
- mesures de la rotation des piles et de leur verticalité,
- mesures de l'horizontalité des corniches sur les 4 faces,
- forages dans les fondations au droit des piles avec examen endoscopique,

2.3 Analyse des désordres

Cette analyse, facilité par l'existence des plans de l'édifice a permis à Michel BANCON de constater que le bâtiment souffrait d'un tassement des fondations avec un mouvement hélicoïdal de l'Arc.
Les fondations constituées de gros blocs en pierre ont subi des mouvements consécutifs à la dégradation de leur joint.L'eau de pluie de l'esplanade, l'eau de ruissellement des façades et l'eau de terrasse canalisée vers des collecteurs sans doute fuyards est la cause des circulations d'eau qui délavent les joints de fondation entrainant une forte altération du mortier à la chaux aérienne. Le tassement différentiel des fondations ainsi généré, entraine une déformation dite en selle de cheval en partie supérieure de l'édifice avec une tendance à l'éloignement des sommets de piles dans le sens des petits côtés et d'une convergence des piles dans l'autre sens. Michel BANCON explique ce comportement différentiel par la configuration des nombreuses cavités ménagées dans l'Arc qui par leur emplacement et leur géomètrie sollicitent plus le bâtiment dans l'axe des petits côtés. Une analyse par libération des contraintes montre que celles-ci sont varient à l'intérieur des maçonneries de 0 à 50 bars.

2.4 Travaux de confortement

Ces analyses ont permis d'établir un plan de confortement comprenant 5 phases :
a) Traitement des joints de maçonnerie par injection partielle de coulis spéciaux dans les fondations.
b) Traitement des fissures en superstructures par injection de coulis ciment.
c) Confortement des superstructures par mise en place des tirants précontraint à l'intérieur de l'édifice, objet de notre communication.
d) Injections complémentaires au coulis dans les massifs de fondation.
e) Etanchéïfication des abords de l'Arc.

3. CONFORTEMENT DES SUPERSTRUCTURES PAR PRECONTRAINTE ADDITIONNELLE

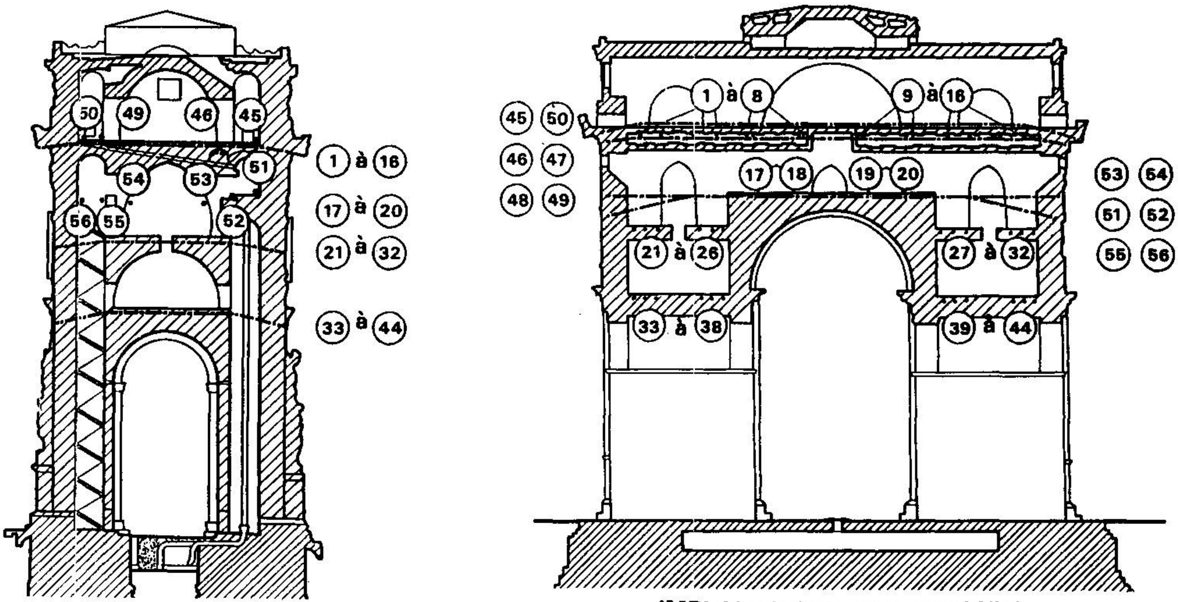
3.1 Principe

Dans le cas de l'Arc de Triomphe, il s'agit d'une précontrainte additionnelle réalisée à l'intérieur de la structure permettant de recomprimer les zones fracturées et de recentrer les efforts obliques engendrés par la poussée des voutes.

Cette précontrainte additionnelle est réalisée par 112 1/2 tirants ancrés dans les parements et raccordés par paire en leur milieu par des coupleurs actifs.

La répartition des 56 tirants tiend compte de plusieurs facteurs :

- le rééquilibrage des contraintes qui nécessite 4 étages de tirants dans le sens du petit côté et 2 étages suivants le grand côté.
- la présence d'équipement existant à l'intérieur de l'ouvrage,
- le phasage des travaux, la mise en tension devant pouvoir se faire de manière progressive, afin d'équilibrer les efforts à répartir.
- la possibilité de règlages ultérieurs des efforts dans les tirants,
- l'esthétique finale du renforcement compatible avec le cadre de l'édifice,



IMPLANTATION DES TIRANTS

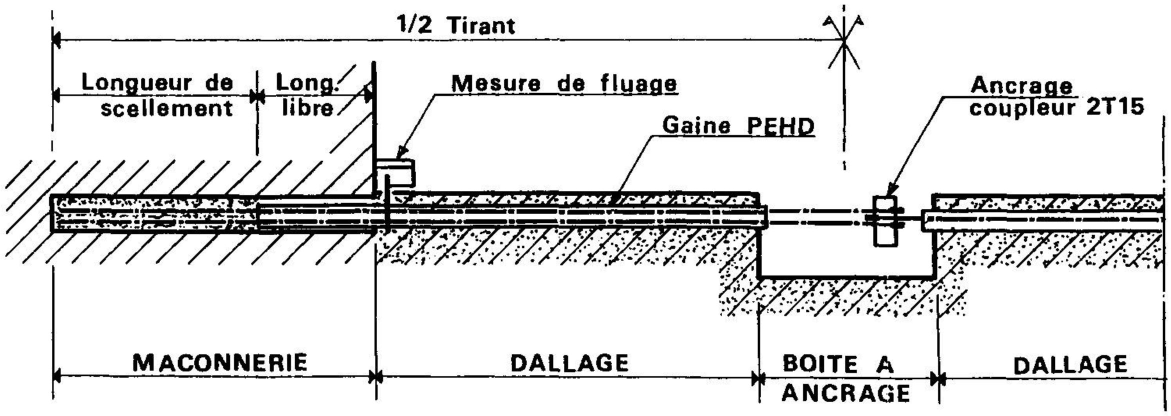
3.2 Définition du tirant

Chaque tirant est réalisé en 2 parties symétriques comprenant chacune 3 zones.

1° La zone d'ancrage située à l'intérieur des maçonneries permettant d'ancrer l'effort de précontrainte au voisinage du parement extérieur.

2° La zone de tension située dans une réservation de maçonnerie au milieu du tirant permettant le raccordement et la mise en tension du tirant.

3° Les zones libres situées entre les ancrages de maçonnerie et le coupleur central permettant la transmission de l'effort de précontrainte.



L'effort nominal maximum à l'ancrage est de 42,5 T. Il est obtenu par 2 torons en acier dur T.15 Euronorm Super tendus à 80% de FRG.
La tension est réalisée par un ancrage coupleur FREYSSINET 2T15 au moyen d'un vérin hydraulique double corps et double effet.

3.3 Ancrage de l'effort dans la maçonnerie

La réservation nécessaire à l'ancrage dans la maçonnerie est matérialisée par un forage en rotation pure, carottage ou marteau fond de trou. Le forage est réalisé de manière à pouvoir s'ancrer le plus près possible du parement extérieur en gardant une marge d'environ 25cm, afin de ne pas le traverser.
Les paramètres qui caractérisent l'ancrage sont :

- diamètre du forage,
- mode d'exécution du forage,
- longueur de scellement,
- méthode de cintrage et de frettage des torons dans la zone de scellement,
- nature du coulis de scellement,

Leur définition a nécessité de nombreux essais préliminaires réalisés in situ avant les travaux de mise en oeuvre proprement dit des tirants et sont décrits au chapitre 4.

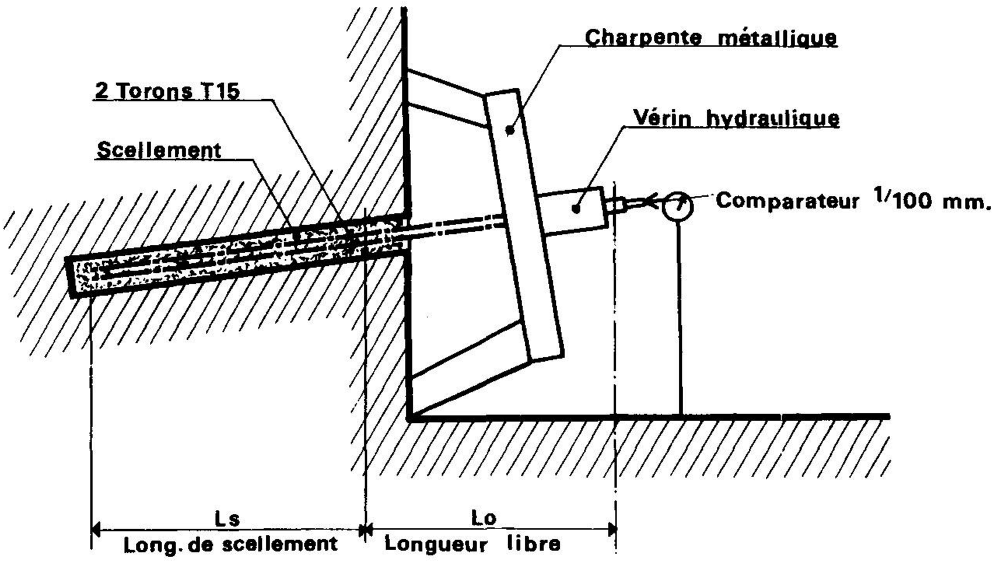
3.4 Parties libres du tirant

Les parties libres extérieures du tirant sont réalisées en double protection.
Le toron T15 (Ø 15,7mm, section 150 mm²) est logé dans une gaine plastique et protégé par une graisse anti-corrosive. Cette opération réalisée en usine permet une auto protection à l'origine du toron.
Lors du façonnage sur le chantier, les 2 torons formant tirant sont enfilés dans une gaine polyéthylène haute densité qui est injectée au coulis de ciment après mise en place du tirant.
L'armature T15 en partie libre se trouve ainsi sous double protection dans un film de graisse qui va permettre sa mise en tension et d'éventuelles reprises de tension ultérieures.
La partie active, ancrage coupleur 2T15 reste accessible dans sa réservation fermée et protégée dans un capot injecté à la cire.

4. ESSAIS PRELIMINAIRES

4.1 Principe

Les essais préliminaires réalisés en vraie grandeur sur le site dans des zones jugées représentatives des cas les plus défavorables ont permis de définir les caractéristiques de l'ancrage dans la maçonnerie. Ils ont également permis d'affiner la méthode de mise en charge des tirants avec contrôle de la fiabilité. Le principe de montage de l'essai est le suivant :



On mesure l'allongement Δl du toron en fonction de F effort appliqué.
F est appliqué par palier de 5 Tonnes de 10 à 40 Tonnes avec pour chaque palier la mesure des allongements dans le temps :
- toutes les minutes de 0 à 5 mn,
- toutes les 5 mn de 5 à 30 mn,

Pour chaque palier d'effort, la pression est maintenue constante pendant les 30mn de mesure d'allongement.

4.2 Interprétation des résultats

Les premiers essais réalisés ont mis en évidence la necessité de réaliser un prétraitement de la maçonnerie dans la zone d'ancrage. Ce prétraitement consiste à réaliser une injection à pression contrôlée de coulis de ciment en fond de forage, afin de colmater les vides existants dans les joints de pierre au voisinage immédiat de la zone de scellement.
Au total 14 essais d'ancrage ont été réalisés et l'analyse des tableaux de résultats de mesure a permis de définir que :
- le diamètre et le mode d'exécution du forage ne sont pas des facteurs déterminants.
- la longueur optimale du scellement pour 40 T est de 3ml.
- les torons doivent être positionnés et centrés correctement pour assurer un parfait enrobage du coulis de scellement.
- l'écrouissage de la maçonnerie est important lors de la première mise en charge.
- le fluage du scellement est le critère de contrôle à suivre lors de la mise en tension.

4.3 méthode de mise en charge

Chaque tirant est mis en tension par palier d'efforts et de temps en "diagonale" avec contrôle du fluage en tête de forage.
- pour 10 T. à 2 mn (avec correction d'origine) 15 T. à 5mn, 20 T. à 10mn, 25 T. à 15mn, 30 T. à 20mn, 35T. à 25mn, et 40 T. à 30mn.

Chaque valeur de mesure est immédiatement positionnée sur un graphique théorique preparé à l'avance où elle doit se situer à l'intérieur d'un fuseau admissible. Toute dérive à l'extérieur du fuseau entraine l'arrêt de la mise en tension si les 40 Tonnes ne sont pas atteintes.

5 MISE EN OEUVRE DES TIRANTS

5.1 Exécution des réservations

L'exécution des réservations comprend :
- l'amorce de saignée contre le parement,
- le forage de la maçonnerie pour ancrage des tirants et passage de la gaine.

Les techniques de base du forage sont celles du carottage à l'air avec lubrification à la mousse et la technique de forage destructif avec des marteaux fond de trou à très faible énergie de frappe, cette technique pouvant se révéler la seule efficiente dans les zones de maçonnerie présentant des pierres abrasives et de forte résistance.
- les saignées dans les dalles pour réservation du passage de gaine qui sont réalisées par sciage des limites à la scie de sol, puis piquage de la zone entre traits de scie pour obtenir une réservation d'environ 10x10 cm.

Après mise en place du tirant, la saignée est remplie au mortier de chaux avec finition de surface aux mortiers spéciaux.
- les chambres dans les dalles pour réservation de l'ancrage coupleur et des appareillages de contrôle de tension sont réalisées comme précédemment.

Les chambres devant rester accessibles sont ragréées au mortier de chaux sur leurs 4 faces latérales et le fond.

5.2 Préfabrication des tirants

Les 1/2 tirants sont préfabriqués sur le chantier le long d'une aire abritée aménagée à cet effet. A partir des plans d'exécution, après contrôle des dimensions in situ, les torons sont coupés à longueur et dégainés, puis dégraissés sur leur longueur de scellement. Sur cette longueur les câbles seront détoronnés pour permettre la mise en place d'une olive en extrémité et équipés de centreurs. Le tirant ainsi préparé est enfilé dans le forage avec un tube évent plongeur.

5.3 Scellement du tirant

Après enfilage du demi tirant on réalise un obturateur avec évent de purge en tête du forage. Le scellement se fait par le tube évent plongeur au coulis de ciment C/E=2 avec 2% d'intraplast Y en fin de remplissage le tube plongeur est extrait et on réalise par le tube évent un maintien en pression du coulis à environ 2 bars pendant 15mn.

5.4 Fixation des parties libres

Après scellement de l'ancrage, la partie libre est enfilée dans une gaine PEHD, soit posée dans les caniveaux aménagés, soit fixée le long des parois par des colliers supports. Le raccordement de 2 demi tirants jumelés se fait dans la chambre de tension grâce à l'ancrage coupleur FREYSSINET type 2T15 ; un équipement spécifique de mesure d'allongement est réalisé sur le câble en tête de chaque forage ; la gaine PEHD est alors injectée au coulis de ciment.

5.5 Mise en tension

La mise en tension des tirants se fait systématiquement par groupe de 2 ou 4 tirants au moyen de vérins hydrauliques FREYSSINET, type Twin Jack. Elle est réalisée par palier comme indiqué au § 4.3 avec contrôle strict du fluage par rapport à la courbe théorique d'étalonnage. Après mise en tension l'ancrage coupleur reçoit un capot inox en demi coquilles qui après fixation est injectée à la cire, les chambres de tension étant équipées de couvercles métalliques. Cet ensemble permet de réintervenir à tout moment pour modifier l'effort de tension.

5.6 Cas des tirants à scellement court

Pour environ 10 tirants l'épaisseur des parements de maçonnerie limite la longueur de scellement à 1,80m. Aussi des modifications techniques ont été nécessaires.

- augmentation du diamètre de forages à 100ml,
- centrage et frettage des torons par une spire Ø 6 au pas de 10 cm,
- scellement du tirant en mortier spécial sans retrait,

6. INTERVENANTS ET REMERCIEMENT

Je tiens à remercier tous les intervenants impliqués dans cette opération délicate.

- Cabinet Michel MAROT, Architecte des Bâtiments Civils et Palais Nationaux,
- Fernand TOMASINA, Vérificateur des Bâtiments Civils, Palais Nationaux et Monuments historiques,
- Bureau Michel BANCON, Ingénieurs Experts,
- CEBTP, Laboratoire d'Analyses,
- SOLETANCHE et RONTAIX, Sociétés partenaires de FREYSSINET sur cette opération
- PX CONSULTANT, Ingénieurs Conseils,
- Ainsi que le Ministère de la Culture et de la Communication.

Renforcement du silo à céréales du port de Safi

Instandstellung der Getreidesiloanlage im Hafen von Safi, Marokko

Reinforcement of Grain Silo at the Port of Safi, Morocco

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Né en 1936, Ecole d'Ingénieur de Marseille, Michel Laffray est Ingénieur d'Etudes de structures, bâtiments et ouvrages d'art chez Europe Etudes GECTI depuis 1961.

Ingénieur de l'Ecole Supérieure d'Electricité, chargé de mission chez SO. SI. PO, Mostapha El Khyari est maintenant Directeur de Techni-Controle et Expert près les Tribunaux.

RÉSUMÉ

Dès 1964 les premières fissures sont apparues dans la structure du silo de Safi, construit en 1957. A la suite d'un concours international, une opération de renforcement a été entreprise en 1987. Elle comporte un traitement local des fissures pour redonner au silo son étanchéité, une mise en précontrainte pour permettre aux fissures de rester fermées sous l'effet des sollicitations en service, une peinture de la surface externe du silo pour parfaire l'étanchéité des microfissures, protéger les aciers dénudés et redonner à l'ensemble l'aspect d'une structure fonctionnelle parfaitement adaptée à sa fonction.

ZUSAMMENFASSUNG

1964 erschienen die ersten Risse in der 1957 gebauten Getreidesiloanlage von Safi (Marokko). Nach einer internationalen Ausschreibungen wurden 1987 die Verstärkungsarbeiten durchgeführt. Diese bestehen aus einer lokalen Rissbehandlung für die Dichtigkeit, einer Vorspannung, damit die Risse unter Gebrauchslasten geschlossen bleiben, eines Aussenanstrichs zur Verbesserung der Dichtigkeit der Mikrorisse zum Schutze der Stahleinlagen und um dem gesamten Bauwerk wieder ein ansprechendes Aussehen zu geben.

SUMMARY

In 1964 the first cracks appeared in the structure of the grain silo built at Safi, Morocco, 1957. Further to an international competition, a reinforcement operation was undertaken in 1987. It consists of local treatment of cracks to restore the silo water-tightness, prestressing so that the craks remain closed during service, painting the external surface of the silo to improve the water-tightness in microscopic cracks, to protect the steel reinforcing bars and give to the whole structure an impression of being perfectly adapted to its function.

0. INTRODUCTION

L'activité du port de Safi, premier port de pêche du Maroc, est aussi pour une grande part consacrée à l'importation et à l'exportation de céréales. Le silo à céréales de Safi, ouvrage important construit en 1957 présentait de graves dégradations dans la structure en béton armé : fissurations verticales et horizontales dues à une insuffisance du ferraillage et à l'importance du gradient thermique. A la suite d'un concours international, l'Office National Interprofessionnel des Céréales et des Légumineuses (O.N.I.C.L.) a passé un marché pour la réhabilitation de la structure qui comportait :
- traitement de la fissuration
- renforcement par précontrainte extérieure
- traitement de surface et revêtement étanche par peinture sur toute la surface extérieure du silo.

Participants aux travaux :

Maître d'Ouvrage	: Office National Interprofessionnel des Céréales et des Légumineuses (O.N.I.C.L.).
Maître d'Oeuvre	: Société des Silos Portuaires (SO.SI.PO).
Titulaire du Marché	: Campenon-Bernard.
Sous-traitants	: Freyssinet (précontrainte - étanchéité). Europe-Etudes (étude de la structure).
Bureau de Contrôle	: Socotec Maroc.
Laboratoire de Contrôle	: Laboratoire Public (LPEE).

Quelques chiffres indiquent l'importance du travail de réparation :
- 7.040 carottages biais
- 1.200 cerces de précontrainte
- 11.000 m de protection de gaine en micro-béton
- 12.500 m2 de traitement de surface et de revêtement étanche.
- 800 m de traitement local de fissures.

1. PARTICULARITES D'UNE INTERVENTION SUR UN SILO A CEREALES

Un silo à céréales est un équipement industriel assujetti à un rythme annuel précis imposé par la date de la récolte qui doit y être stockée. Le stockage de la récolte est alors relativement rapide, le déstockage plus ou moins lent selon les besoins du marché. Indépendamment de ce rythme annuel, un silo en zone portuaire (c'est le cas de celui de SAFI) doit faire face à des mouvements de céréales liés aux chargements ou déchargements de bateaux céréaliers d'exportation ou d'importation.

Intervenir pour des travaux de renforcement dans une pareille structure, implique une adaptation des méthodes et des moyens d'intervention qui limite au maximum la gêne d'exploitation, en sachant que le produit stocké est à la fois périssable et alimentaire. Le Maître d'Ouvrage et le titulaire du marché de travaux doivent alors établir un plan de travail détaillé tenant compte des servitudes de l'exploitation et de celles des interventions de réparation. De plus dans le cas de SAFI, le traitement des fissures - préalable à la mise en précontrainte - imposait le remplissage de la cellule correspondante pour les ouvrir et la vidange immédiate pour effectuer le reste des opérations de renforcement. Dans ce cas, l'ensemble des travaux s'est étendu sur une période de 12 mois d'avril 1987 à mai 1988.

2. DESCRIPTION DU SILO

Il s'agit d'un équipement important, d'une capacité de stockage de 25.000 t environ. La structure comporte deux batteries comprenant chacune 3 rangées de 5 cellules tangentes entre elles, réparties de part et d'autre d'un bâtiment central de services comportant les élévateurs à céréales et les accès pour le personnel (ascenseur, escaliers). Le stockage est fait à la fois dans les cellules circulaires, et dans les espaces entre cellules tangentes (cellules en as de carreau au nombre de 16). Les cellules ont une hauteur de 34 m.

La structure de stockage est constituée par des parois en béton armé d'épaisseur courante 12 cm. La fissuration importante, à la fois verticale et horizontale, a des origines multiples liées à la fois aux hypothèses de calculs et à la qualité de la réalisation. L'étude du renforcement a pris en compte toutes les sollicitations de fonctionnement (statiques et dynamiques) liées aux opérations de stockage et déstockage, de même que les sollicitations climatiques (gradient thermique) particulièrement importantes dans un site maritime et chaud, tel que celui de SAFI; certaines parties peuvent être soumises, dans un laps de temps très court, au brouillard, au soleil intense et au vent.

3. ETUDE DU RENFORCEMENT

3.1. Textes réglementaires

- Béton armé : règles BAEL 83
- Béton précontraint : règles BPEL 83
- Calcul du silo : Annales ITBTP juillet-août 86

3.2. Hypothèses de calcul

- Caractéristiques retenues pour la matière ensilée (blé) : masse spécifique 850 kg/m3, angle de frottement interne 26°
- Coefficient de majoration dynamique appliqué aux efforts statiques (premier état d'équilibre) 1,3
- Gradient thermique retenu : 15° C pour les faces extérieures avec un module de béton de 30.000 MPa (ensoleillement) et 9°C pour les faces intérieures avec un module de béton de 20.000 MPa.

3.3. Matériaux

- Béton : pour une première série d'essais, la résistance atteignait 36,5 MPa en moyenne soit 33 MPa en nominal.
- Armatures existantes : compte-tenu de la corrosion des armatures au droit des fissures, et du très faible pourcentage d'armatures verticales, le Maître d'Ouvrage a exigé que les vérifications soient faites sans tenir compte des armatures existantes.
- Armatures de précontrainte : câbles monotoron gainé-graissé T15 super, très basse relaxation. Le coefficient de frottement de 0,05 et la valeur de la déviation parasite pris en compte (1 degré par mètre) se sont avérés pessimistes (allongements obtenus supérieurs à l'allongement de calcul).

3.4. Caractéristiques particulières au projet

La difficulté spécifique de la structure provient du fait que les cellules ne sont pas indépendantes mais accolées, retirant ainsi le caractère "de révolution" aux efforts appliqués. La faible épaisseur des parois (12 ou 13 centimètres) et le parti de négliger les armatures existantes ont conduit à effectuer des calculs de pression de précontrainte extrêmement précis sur chaque portion de barre modélisée dans le programme.

Les calculs ont été menés pour une batterie de cellules à différents niveaux et par cellule pour le calcul de flexion des parois dans la zone du fond du silo, dans la zone de couverture et en partie médiane (efforts dûs à la mise en tension des câbles). La force de précontrainte a été ajustée en fonction du niveau des câbles, de manière à ne pas solliciter le béton au-delà de sa possibilité compte-tenu des faibles épaisseurs et du ferraillage négligeable. Un calcul itératif en section fissuré a permis d'arriver à des cerces groupées en 4 séries de tension, afin d'éviter toute erreur sur le chantier, allant de 0,8 Rg à 0,2 Rg. Les vérifications ont été faites dans les différentes phases de construction de manière à permettre la mise en tension d'une cellule complète avant début de précontrainte de la cellule voisine et remplissage ou vidange de la cellule suivante.

4. REALISATION DU RENFORCEMENT

4.1. Installation du chantier :

Elle comporte les installations au sol pour la préfabrication, notamment des câbles de précontrainte et les installations permettant d'accéder sur toutes les faces du silo internes et externes y compris dans les cellules en as de carreau. Pour ces dernières, le choix s'est porté sur des passerelles légères, manoeuvrées à la main par les opérateurs eux-mêmes (L.A.H.O.). Dans certaines zones d'activité internes (cellules as de carreau), on a parfois utilisé deux passerelles superposées pour la réalisation d'opérations successives (mise en place, puis mise en tension des câbles de précontrainte). Ce type de passerelle impose pour le déplacement vertical, la présence de deux hommes. Lors de certaines opérations (enfilage des gaines), il était parfois nécessaire de disposer 6 passerelles au même niveau, dans six cellules différentes, ce qui imposait la présence de douze personnes. Dans d'autres circonstances, l'utilisation de passerelles déplacées mécaniquement aurait permis de réduire l'importance de la main d'oeuvre :

4.2. L'injection des fissures :

Les fissures d'une ouverture supérieure à 0,3 mm ont été traitées. Le principe du traitement était le suivant : création d'une saignée de 5 x 5 mm au droit de la fissure, puis remplissage de la saignée à l'aide d'un mastic souple type SIKAFLEX 15 LM.

4.3. Le forage de trous pour le passage des câbles :

Le forage a été exécuté par couronne diamentée (carottage); chaque forage avait une longueur de 0,60 m. La difficulté principale résidait dans l'implantation de l'axe de forage, en effet, celui-ci devait sortir tangentiellement au voile cylindrique du silo. L'utilisation d'un gabarit de positionnement a permis de résoudre ce problème de façon satisfaisante.

4.4. Le renforcement par précontrainte :

4.4.1. Le mode de précontrainte

Pour satisfaire aux exigences de ce renforcement, il était nécessaire d'une part de bénéficier d'un effort de précontrainte le plus constant possible sur la périphérie des cellules. Le choix des points d'ancrage dans un même plan vertical dans les cellules en as de carreau, pour des raisons à la fois pratiques d'exécution et d'esthétique, interdisait le déplacement des points d'ancrages pour uniformiser la précontrainte; il permettait d'autre part de mettre en place une précontrainte extérieure la mieux protégée possible. La technique adoptée est une solution originale développée par FREYSSINET, consistant à utiliser le toron gainé-graissé mis en place dans une gaine en polyéthylène haute densité (PEHD) 28 x 32 mm, injectée au coulis de ciment préalablement à la mise en tension. Cette disposition permet d'enrober le toron gainé-graissé dans une gaine de coulis rigide - exactement adaptée à sa forme - qui recrée l'environnement pour lequel ce produit a été créé, les dalles de béton précontraintes par post-tension. Les conditions mécaniques de fonctionnement de la précontrainte - coefficient de frottement en courbe inférieur à 0,05, absence de détérioration de la gaine PEHD du toron - sont parfaitement respectées et la protection contre les aggressions extérieures - gaine externe en PEHD et coulis de ciment - est également excellente.

L'ancrage des deux extrémités d'une cerce est réalisé dans une pièce d'ancrage unique en forme de "X", le câble utilisé étant un toron gainé-graissé de diamètre 15,2 mm. Pour des raisons esthétiques, il a été décidé de disposer les câbles de renforcement de façon uniforme sur toute la hauteur (40 cerces); les sollicitations diminuant avec le niveau des câbles, les câbles ont été tendus avec des forces décroissantes du bas vers le haut du silo. Tous les ancrages sont regroupés dans les cellules en as de carreau. L'essentiel des opérations de précontrainte était donc réalisé à partir de ces zones exiguës de 2 x 2 m2 environ.

4.4.2. La mise en oeuvre de la précontrainte

La cadence élevée de mise en place de la précontrainte, l'exiguïté de l'espace en haut du silo imposaient une préfabrication des câbles et des gaines. Les câbles ont été découpés au sol, dénudés localement, enfilés sur l'ancrage (une seule extrémité), enroulés et transportés par l'ascenseur à la partie haute du silo. Les gaines étaient préparées de la même façon. La mise en place des gaines des câbles était réalisée à partir des cellules en as de carreau; toutes les passerelles situées sur le trajet d'un câble - en général six - sont alors au même niveau. La gaine est d'abord descendue, déroulée depuis le haut avec une corde, puis enfilée au travers de chacun des trous de parois, par poussage manuel par le personnel présent sur les plates-formes. Le toron est alors descendu de la même façon et enfilé sur l'ancrage X 15, puis tous les raccordements sont alors faits pour permettre l'injection de l'espace entre gaine extérieure et toron.

L'injection du coulis de ciment est alors réalisée avec une pompe à main, sous une faible pression; rappelons que le coulis d'injection n'a pas ici de fonction de protection. Sa fonction essentielle est de réaliser une forme rigide enveloppant le toron gainé-graissé et permettant d'exécuter la mise en tension sans détérioration de sa gaine PEHD, par suppression des points de contact singuliers. La qualité du coulis et sa mise en oeuvre sont donc beaucoup moins importantes que lorsque le coulis doit assurer la protection de l'acier. Après injection, le câble est réglé en position définitive. Le premier câble, au bas du silo, est réglé à la lunette; les autres sont réglés par rapport à cette référence à l'aide de piges de longueur constante. Le résultat sur l'aspect extérieur est tout à fait excellent. La mise en tension est alors effectuée simultanément par les deux extrémités du câble, afin d'éviter le déplacement de l'ancrage et de permettre d'avoir le maximum d'effort dans le câble.

Les finitions sont alors réalisées; elles comportent la coupe des torons, la mise en place d'un double capot de fermeture de l'ancrage pour éviter les fuites de graisse. On doit impérativement éviter toute fuite pour conserver l'ambiance "qualité alimentaire". L'ancrage est alors enrobé dans un cachetage en micro-béton, dessiné pour faciliter l'écoulement des céréales et éviter la constitution des voûtes de grains. De même la partie de câble interne aux cellules est revêtue d'un déflecteur en micro-béton ayant la même fonction. Toutes ces opérations demandent beaucoup de soins à la réalisation, pour que l'accrochage au béton de la structure soit d'excellente qualité afin de résister à l'abrasion des céréales et aux efforts de poussée.

5. ETANCHEITE EXTERIEURE

Le choix du complexe d'étanchéité a été motivé par la nécessité de répondre aux exigences suivantes :
- Grande souplesse pour reprendre les dilatations thermiques,
- Stabilité aux ultraviolets (U.V.).

Le complexe retenu est le suivant :
- 2 couches d'époxane (époxy-uréthanne) de 500 g/m2 chacune par résine Freyssi 450,
- 1 couche de résine polyuréthane de 350 g/m2 par résine Freyssi 620.

L'application était faite à la brosse et au rouleau.

6. CONCLUSION

Cette importante affaire de renforcement de silo est sans doute une première mondiale tant par son ampleur que par l'association de différentes techniques de renforcement (forage, injection de résine, précontrainte extérieure, peinture d'étanchéité, etc...) permettant de redonner à une structure quelconque, non seulement ses capacités initialement souhaitées, mais également de nouvelles possibilités tenant compte de l'évolution des technologies depuis l'époque de la construction. Cette remarquable réalisation a valeur d'exemple et invite les Maîtres d'Ouvrage à envisager sereinement les opérations de renforcement de structures - même très dégradées -, souvent préférables aux démolitions et reconstructions, à condition toutefois de s'adresser à des spécialistes capables de maîtriser simultanément tous les aspects de la question, depuis l'expertise jusqu'aux détails d'exécution.

Réparation du pont de Veauche sur la Loire

Instandsetzung der Loirebrücke in Veauche

Repair of the Veauche Bridge over the River Loire

Jean ARCHIER
Ingénieur INSA
Freyssinet STUP
Lyon, France

Jean Archier, né en 1946, obtient son diplome d'ingénieur T.P. à l'INSA de Lyon. Pendant une quinzaine d'années, il s'occupe de travaux d'ouvrages d'art. Puis, en 1980, il devient Directeur de Freyssinet STUP Torkret France pour la région Rhône, Alpes, Auvergne, Bourgogne.

RÉSUMÉ

L'originalité de cette réparation est d'avoir permis, sur une structure inhabituelle (arcs encastrés sur piles supportant un hourdis par l'intermédiaire de bracons et d'entretoises, le tout en béton armé) l'utilisation, dans un délai très court, de toutes les techniques disponibles aujourd'hui pour la réparation et le renforcement des structures.

ZUSAMMENFASSUNG

Die Besonderheit dieser Reparatur besteht in der Anwendung, innerhalb einer sehr kurzen Frist, aller heute vorhandenen Techniken für die Instandsetzung und Versteifung von Brücken, an einem ungewöhnlichen Bauwerk (auf Pfeiler eingespannte Bogen, die die Fahrbahnplatte über Stützen und Riegel tragen).

SUMMARY

On this relatively unusual structure (lateral arches restrained at both ends on piles and supporting a deck by means of reinforced concrete rakers and cross-ties), the original scheme was, in a very short time, to utilize all repairs techniques.

1 - INTRODUCTION

La structure de l'ouvrage à conforter est relativement inhabituelle et consiste en des arcs latéraux de 27,50 ml de longueur encastrés sur piles et supportant un hourdis par l'intermédiaire de bracons et d'entretoises en béton armé. La continuité de l'ouvrage est assurée par des dalles indépendantes fonctionnant en cantilever. L'ouvrage est long de 165 ml et large de 7 ml.

Construit dans les années 1930, le Pont de VEAUCHE présentait plusieurs désordres dus principalement à deux causes :
- Une d'origine structurelle : à savoir la capacité portante de l'ouvrage qui, calculée avec le réglement de l'époque (1927), devenait limite vis à vis des sollicitations actuelles du trafic moderne (réglement de 1971).
- Une due aux agressions du temps et à quelques défauts d'exécution de l'époque, (nids de cailloux, enrobage des armatures, mauvaise étanchéité de surface, notamment au niveau des joints de chaussée...)

1.1 - Des moyens d'exécution très importants dans un délai réduit

Pour pallier cette évolution, il était nécessaire, après restauration du monolithisme de la structure (fissures et béton des entretoises) de rajouter un effort de compression au moyen de câbles de précontrainte extérieurs au béton.
Les travaux de traitement des bétons, travaux maintenant classiques (ragréage, imperméabilisation, étanchéité) permettent de réparer les dégâts reconnus sur la structure et de la protéger grâce aux procédés et matériaux modernes (résines de synthèse, béton projeté).
Le projet de réparation, établi par EUROPE-ETUDES-GECTI, a prévu :
- l'adjonction de précontrainte sur la structure en béton armé,
- le traitement des fissures selon une méthode originale expliquée plus loin,
- l'association de travaux de résines et de béton projeté,
- des travaux importants de bétonnage réalisés par une Entreprise Régionale,
- la reprise de toutes les structures hormis le garde-corps qui devait être conservé.

Le délai d'exécution extrêmement réduit, compte tenu des contraintes locales de circulation sur le plan de coupure de circulation minimum, a nécessité la mobilisation de moyens d'exécution très importants dans un délai réduit.

1.2 - La maîtrise complète des techniques de réparations

FREYSSINET STUP a été déclaré adjudicataire de cette réparation sur la base d'un dossier technique très complet, mettant notamment en valeur la maîtrise complète de toutes les techniques nécessaires à la mise en oeuvre du projet, et les références importantes de réparations de structures effectuées dans la région.
Le chantier s'est déroulé en trois phases principales de travaux :
- les décapages des structures et la réalisation de nouvelles entretoises capables de recevoir les ancrages des câbles 19 T 15 qui seront mis en oeuvre dans des gaines en acier galvanisé extérieures au béton. Ces travaux ont été couplés avec les traitements des fissures de la structure et exécutés sous coupure de circulation de six semaines.
- les travaux préparatoires à la mise en précontrainte de l'ouvrage avec rétablissement de la circulation,
- la mise en précontrainte faite hors circulation et la fin des travaux de traitement des bétons (résines et béton projeté), d'étanchéité et de chaussée.

2 - MOYENS D'ACCES MIS EN OEUVRE

La quasi-totalité des travaux de réparation effectuée se situe en sous-face du tablier hourdis inférieur du tablier, consoles, hourdis sous trottoir.
Ils s'effectuent à partir de passerelles de deux types s'appuyant sur le tablier ainsi qu'un échafaudage particulier pour le béton projeté. Ces passerelles ont été spécialement conçues par ce genre de réparation sur ce type d'ouvrage.

Elles ont d'ailleurs déjà été utilisées sur de nombreux chantiers (Viaduc de la RICA-MARIE, du FAYET, de la CAILLE, etc...).
L'ensemble des échafaudages est composé de la manière suivante :
- une passerelle de 12,50 ml de portée desservant la sous-face du tablier,
- un ensemble d'échafaudage tubulaire arrimé sur la passerelle précédente qui permet d'atteindre de celle-ci le hourdis inférieur du tablier,
- une autre passerelle pour les travaux sur consoles de trottoirs et les hourdis en encorbellement,
- un échafaudage spécifique pour le revêtement extérieur des zones.

3 - MODE OPERATOIRE

3.1 - Renforcement de la structure

Pour la réalisation du renforcement de la structure par précontrainte, nous utilisonsle procédé de précontrainte par câbles 17 T 15 FREYSSINET.

3.1.1 - Ancrages
Les ancrages utilisés, au nombre de 12 unités, sont du type 19 T 15.

3.1.2 - Gaines de protection
Ces gaines de protection sont des tubes d'acier Ø 101 et de 2 mm d'épaisseur. Les gaines sont emboitées les unes dans les autres. Leur étanchéité est réalisée par la mise en place d'une résine époxydique type FREYSSI 504 tandis que leur protection est assurée par galvanisation des tubes en usine.

3.1.3. - Coulis d'injection
Le coulis utilisé pour ces injections est du type "Retardé STUP" afin de parfaire le remplissage des gaines de précontrainte et d'assurer une bonne protection des aciers.

3.2 - Travaux de réparation

3.2.1 - Sablage
Pour le sablage des parements du tablier et des piles, nous utilisons une sableuse avec projection d'abrasifs ne comportant pas de silice.

3.2.2 - Démolition de l'enduit des poutres
Le décapage de l'enduit est réalisé à l'aide de petits marteaux burineurs.

3.2.3 - Traitement des fissures
Afin d'être absolument sûr de faire transiter les efforts, non seulement dans les deux poutres extrêmes de l'arc, mais aussi dans l'âme, le traitement des fissures s'est déroulé de la façon suivante :
- Perçage par carottage de diamètre 100 des âmes des poutres, à cheval sur les fissures suivant la direction de celles-ci, chaque trou étant distant de 40 cm du précédent,
- Après coffrage de l'extrêmité de la zone carottée, remplissage des vides laissés par le carottage par un mortier de CONBEXTRA à retrait compensé, de façon à constituer des clés de blocage assurant la transmission des efforts de cisaillement,
- Les fissures elles-mêmes dans la zone libre entre carottage sont injectées après calfatage au moyen d'une résine époxydique.
Selon le type de fissure (fissure vivante ou morte), le traitement est différent :
- Fissure vivante : le produit injecté doit être élastique afin de ne pas entraver le fonctionnement de l'ouvrage et assurer une étanchéité de la structure dans le but de protéger les aciers.
- Fissure morte : pour restituer le monolithisme à la structure fissurée, le produit à injecter doit avoir une adhérence supérieure à celle du support : il s'agit d'une résine fluide non solvantée. Son injection se fait de la façon suivante :
. Scellement d'injecteurs à raison de trois par mètre,
. Calfatage des lèvres de la fissure à l'aide de la même résine époxydique,
. Injection de la résine fluide à l'aide d'une bouteille munie de manomètres de contrôle,
. Après polymérisation de la résine, enlèvement des évents et du calfatage et remise en état du support.

3.2.4 - Ragréages des parements

. Préparation du support : Les surfaces à ragréer sont bouchardées de manière à dégager les parties de béton non cohérentes et les aciers mis à nu sont sablés à blanc et immédiatement protégés de la corrosion par une peinture époxy zinc FREYSSI 604 EZ qui a la particularité de pouvoir être appliquée en très fine couche pour éviter une éventuelle cohésion du film.

. Ragréage au mortier de résine époxydique : On utilise une résine pure du type FREYSSI 306 FD. Ce type de mortier comporte une partie résine et un durcisseur prédosés. La charge est également pré-mélangée afin d'avoir une granulomètrie très suivie.

. Ragréage au mortier hydraulique additionné de résine thermoplastique. Après sablage, imprégnation d'une résine époxy pure type FREYSSI 306, puis application d'un mortier hydraulique adjuvanté (CEBOND)

. Ragréage à la pâte de résine : Pour les faibles défauts de surface (1 à 5 mm), un ragréage à la pâte de résine époxy chargée du type FREYSSI 201 dans le cas d'une température d'application basse et FREYSSI 202 dans le cas d'une température élevée, est effectué.

3.2.5 - Peinture de protection des bétons

Selon la note d'information technique du L.C.P.C. de Juin 1980 :"Mise en peinture des bétons du Génie Civil", le complexe du type n°15 est utilisé.

. Tablier et piles : L'application est faite après préparation du support par sablage ou dépoussièrage. Le complexe est constitué des couches suivantes :

- 1ère couche : FREYSSI 602 à raison de 200 g/m^2,
- 2ème couche : FREYSSI 602 à raison de 200 g/m^2,
- 3ème couche : FREYSSI 621 à raison de 250 g/m^2.

Sa mise en oeuvre est faite au pistolet AIRLESS ou à la brosse, ou au rouleau, selon les zones d'application. Le choix de la teinte finale a été fait par le Maître d'Oeuvre.

. Garde-corps : Ce complexe de peinture époxy-polyuréthane est constitué par :

- une première couche de FREYSSI 621 à raison de 200 g/m^2,
- une seconde couche de FREYSSI 621 à raison de 250 g/m^2.

Sa mise en oeuvre est réalisée de la même façon que le complexe trois couches.

. Hygrométrie : Aucune peinture n'est appliquée au-dessus d'un seuil défini d'un commun accord avec le Laboratoire de contrôle. Un hygromètre installé en permanence enregistre l'hygromètrie ambiante.

3.2.6 - Béton projeté

Après piquage de l'ancien revêtement des flancs de l'ouvrage, on a procédé à l'application du béton projeté dans le but de recouvrir les armatures faiblement enrobées et de réaliser un enduit d'aspect uniforme pour l'esthétique de l'ouvrage.

A cet effet, un échafaudage complet d'une demie travée a permis une mise en oeuvre sans reprise de la couche de finition.

La formule granulométrique des agrégats s'inscrit dans le fuseau de l'A.F.B. pour granulats de 0 à 8 mm.

Le dosage en ciment est de 300 kg de CPA 55 pour 1200 kg de sable 0/5 et 600 kg de gravillons 3/8.

La projection est réalisée par voie sèche avec utilisation d'une machine à projeter TORKRET type S3, équipée en tuyaux de 32 mm pour le transport des matériaux. L'eau est introduite sous une pression de 7 bars à la lance de projection dans une proportion de E:C = 0,5.

Réalisé en deux passes, le béton a d'abord une épaisseur de 1,5 cm avec un parement dressé à la régle, puis 0,5 cm brut de projection, sans remaniement, pour obtenir un aspect uniforme.

3.3 - Travaux de chaussée

On a effectué les travaux suivants :

- Décapage de la chaussée,
- Préparation du support,
- Mise en oeuvre des joints de chaussée du type SLN

- Mise en oeuvre des joints de chaussée du type FREYSSI P
- Mise en place de descentes d'eau

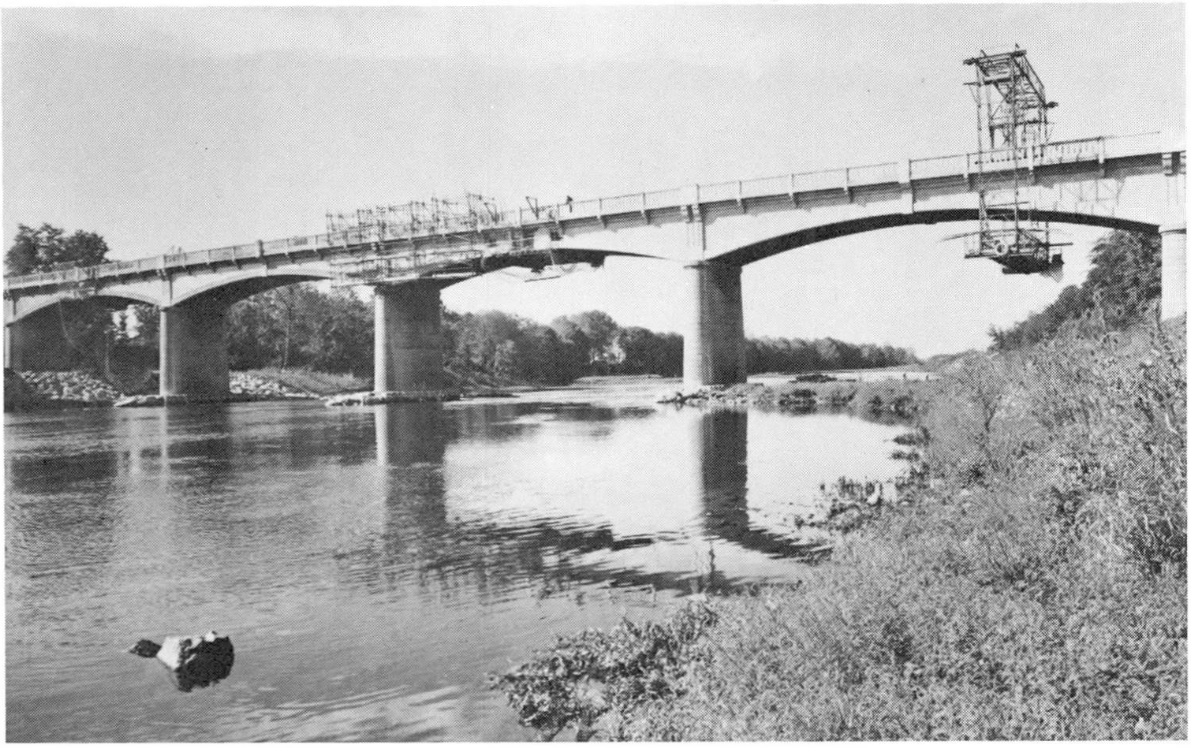
3.4 - Travaux divers

- Renformis en béton d'une épaisseur variant entre 7 et 10 cm.
- Etanchéité des trottoirs avec la résine brai époxy FREYSSI 401.

4 - QUANTITES

- Aciers précontrainte :	6 500 kg
- Ancrages 19 T 15 :	12 unités
- Joints de chaussée P :	10 ml
- Ragréages :	2 000 kg
- Peinture :	5 200 m²
- Injection de fissures :	300 ml
- Béton projeté :	420 m²
- Durée du chantier	Juillet 1984 à Juin 1985 avec un arrêt de 4 mois dû aux conditions climatiques de l'hiver.

Maître d'Ouvrage :	Conseil Général du département de la LOIRE.
Maître d'Oeuvre :	Direction Départementale de l'Equipement de la LOIRE Arrondissement territorial Subdivision de SAINT-GALMIER
Contrôle Technique :	C.E.T.E. de LYON
Bureau d'Etudes :	EUROPE ETUDES GECTI à LYON
Entreprises :	FREYSSINET STUP TORKRET FRANCE à LYON Sté Forézienne d'Entreprises et Terrassements à SAINT-ETIENNE.



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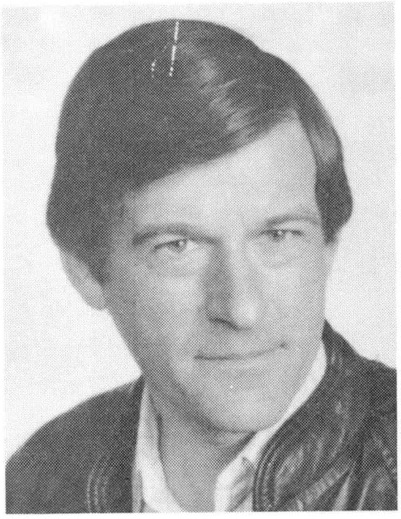
Entretien du patrimoine des ponts

Unterhaltung des Brückenerbes

Maintenance of Bridges Heritage

Ch. VAN BEGIN
Inspecteur général
Minist. Travaux Publics
Bruxelles, Belgique



Né en 1942, l'auteur est Ingénieur civil des constructions de l'Université Libre de Bruxelles. Il a, depuis plus de 10 ans, la charge, au Bureau des Ponts, d'un service d'études d'ouvrages d'art.

RÉSUMÉ

Le ministère belge des travaux publics gère quelque 5200 ponts importants. L'inspection systématique, depuis 10 ans a permis de mettre en évidence les dégradations les plus fréquentes, les plus graves, d'analyser les causes ainsi que les mécanismes de dégradation. Force est maintenant d'actualiser le processus d'inspection, d'optimaliser la maintenance du point de vue technico-économique et d'améliorer la durabilité des nouveaux ouvrages.

ZUSAMMENFASSUNG

Das belgische Bauministerium verwaltet ungefähr 5200 grosse Brücken. Die seit 10 Jahren systematisch durchgeführte Inspektion hat die schlimmsten Mängel und häufigsten Schäden aufgedeckt. Sie hat ferner die Ursachen und die Verfallprozesse der Bauwerke analysieren können. Es geht jetzt vor allem darum, den Inspekionsvorgang zu aktualisieren, die Unterhaltung auf technologischer und ökonomischer Ebene zu optimieren und die Dauerhaftigkeit der neuen Bauwerke zu verbessern.

SUMMARY

The Belgian Ministry of Public Works manages some 5200 important bridges. Their systematic inspection which has been carried out for 10 years now, makes it possible to bring out the most frequent and worst damages and to analyse the causes and the mechanisms of the damage. The thing to be done now is to actualize the inspection process, to maintain the structures in optimal technical and economical condition and to improve the durability of new structures.

SITUATION DU PROBLEME

Le Ministère des Travaux Publics gère quelque 5200 ponts importants situés sur le réseau des routes, autoroutes, voies hydrauliques et chemins de fer.

La moyenne d'âge de ces ouvrages est actuellement de 26 ans, une grande partie ayant été construits avec le développement du réseau d'autoroutes.

Les ponts sont des constructions sophistiquées soumises à usure, fatigue et à de multiples agressions d'ordre climatique (eau de pluie, gel,...) ou d'exploitation (surcharges, percussions, sels de déneigement,...).

Une inspection systématique des ponts effectuée depuis 10 ans a permis de mettre en évidence les dégradations les plus fréquentes, les plus graves, d'en analyser les causes ainsi que les mécanismes de dégradation (Figures 1, 2, 3, Tableau 1).

Force est maintenant d'actualiser éventuellement le processus d'inspection, d'optimaliser la maintenance (entretien, réparations, renforcement, remplacement) du point de vue technico-économique et de définir les moyens (budget, personnel).

Enfin, l'expérience acquise doit permettre d'améliorer la durabilité des nouveaux ouvrages à construire.

ENJEU

La sécurité des usagers, avant tout, justifie la vigilance. La sauvegarde d'un patrimoine de 200 milliards de FB représente une charge financière non négligeable.

Il faut se prémunir contre une dégradation accélérée de ce patrimoine, au risque d'hypothéquer l'avenir. Les ponts sont des noeuds particulièrement vulnérables dans les réseau des communications.

STRATEGIE

La gestion des ponts pose un large problème technico-économique que l'on résoud encore souvent de manière empirique.

Que faut-il réparer, quand faut-il réparer, à quelles charges budgétaires faut-il s'attendre dans les années à venir ?

Ces questions concernent à la fois l'économie, la technique, les probabilités et la statistique. Elles justifient une étude multi-disciplinaire devant nous doter d'un outil efficace d'aide à la décision. Un tel outil ne semble pas encore exploité dans le monde mais des recherches sont en cours dans divers domaines, pour optimaliser la gestion.

La figure n° 4 propose un schéma logique pour la gestion des ponts. Il peut servir de base pour l'organisation intégrée de cette gestion, l'étude et la mise au point de chaque étape.

Du point de vue économique, il se confirme que l'entretien des ponts nécessite un investissement annuel de 1 à 2% du capital de remplacement. Il faut y ajouter le remplacement d'ouvrages dont le taux est actuellement de l'ordre de 0.4% en Europe et supérieur à 1% en Amérique du Nord.

D'autres aspects méritent d'être étudiés et notamment :

- répartition optimale des crédits disponibles pour la gestion
- analyse coût /bénéfice
- rentabilité des investissements consentis pour la durabilité
- facteurs déterminants pour guider la décision de remplacer un ouvrage d'art au lieu de le réparer ou de le renforcer (comparaison des prix, incidence sur le niveau de sécurité et la durée de vie) (Figure 5)
- niveau d'entretien optimal (Figure 6).

CONCLUSIONS

Une bonne gestion du patrimoine répond à une nécessité économique impérieuse. On admet communément qu'elle est rentable.
Entretenir et réparer est moins prestigieux que de réaliser de nouveaux ouvrages. Une évolution des mentalités est toutefois décelée à cet égard qu'il faut favoriser.

D'importants programmes de recherches sont mis sur pied dans un souci d'optimalisation et d'intégration de la gestion, englobant les aspects techniques, administratifs, économiques, aide à la décision et formation du personnel. Il faut oeuvrer dans ce sens.

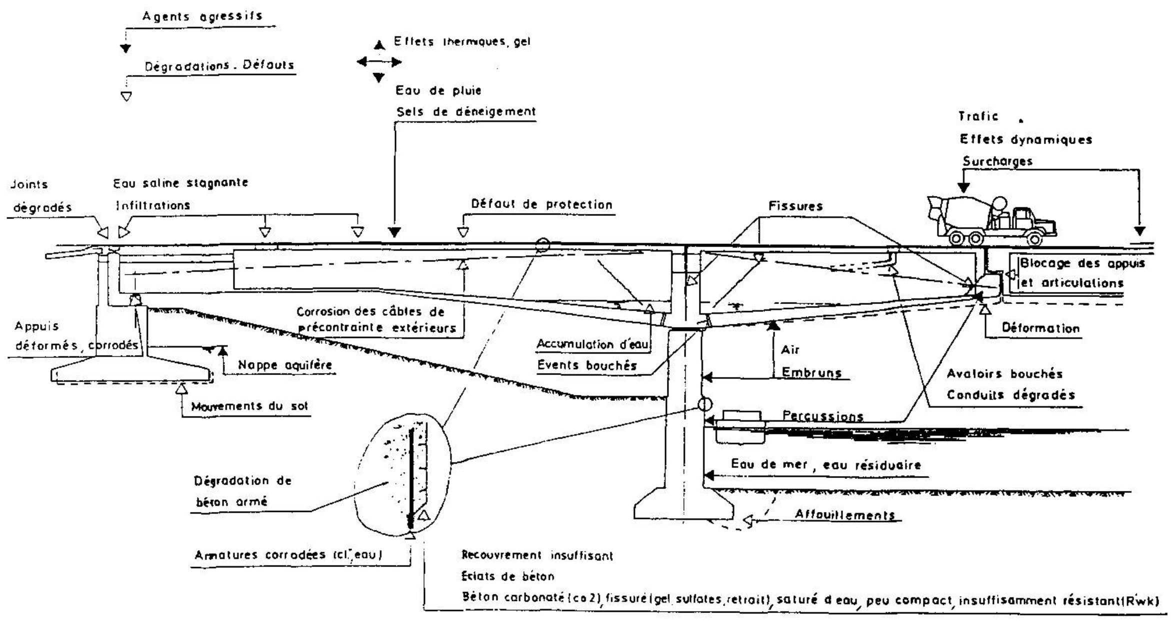


Figure n° 1

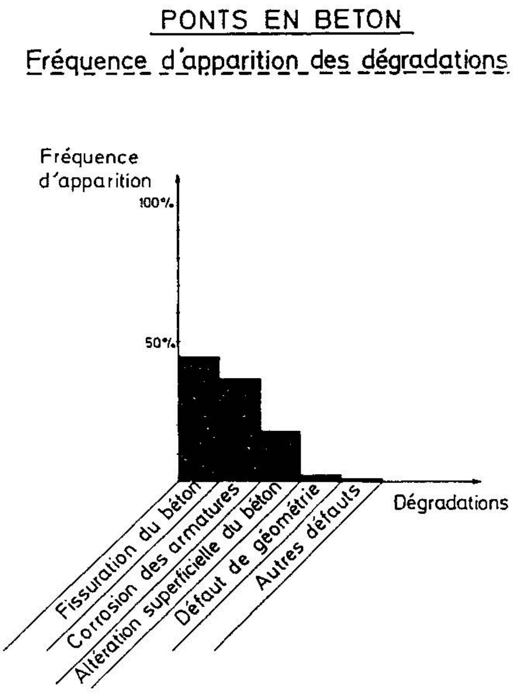


Figure n° 2

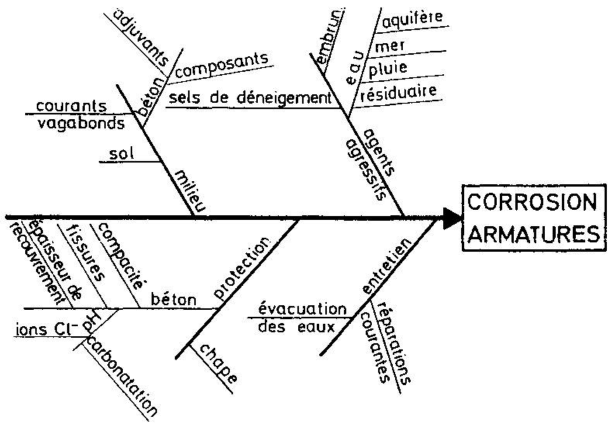


Figure n° 3

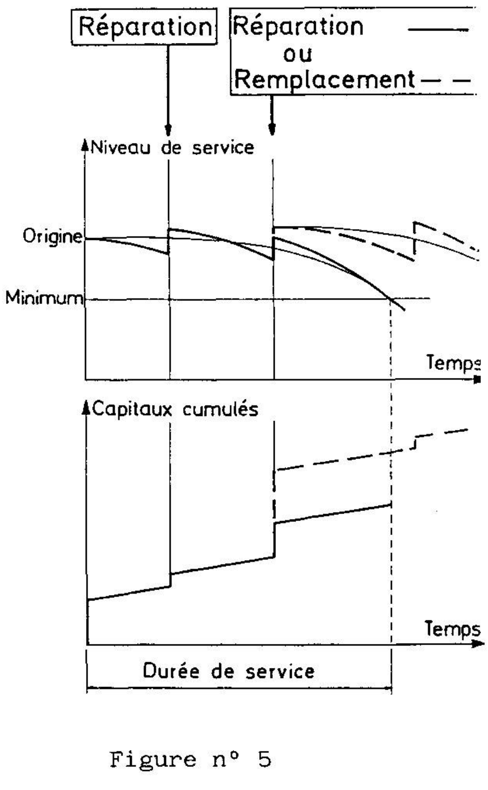


Figure n° 5

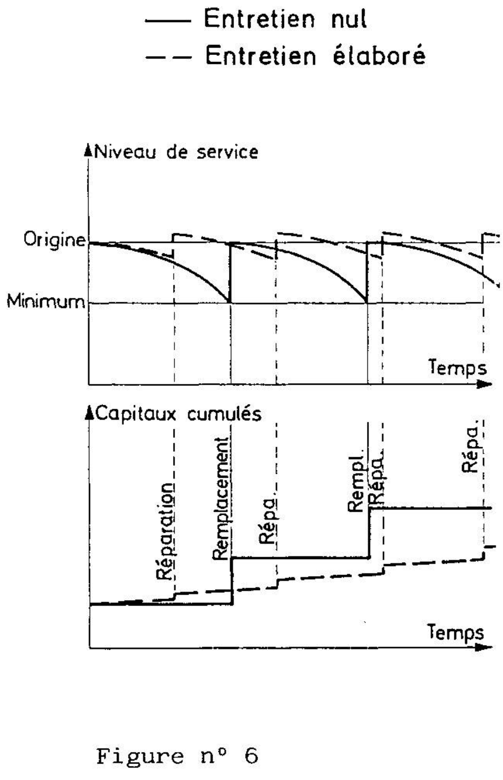


Figure n° 6

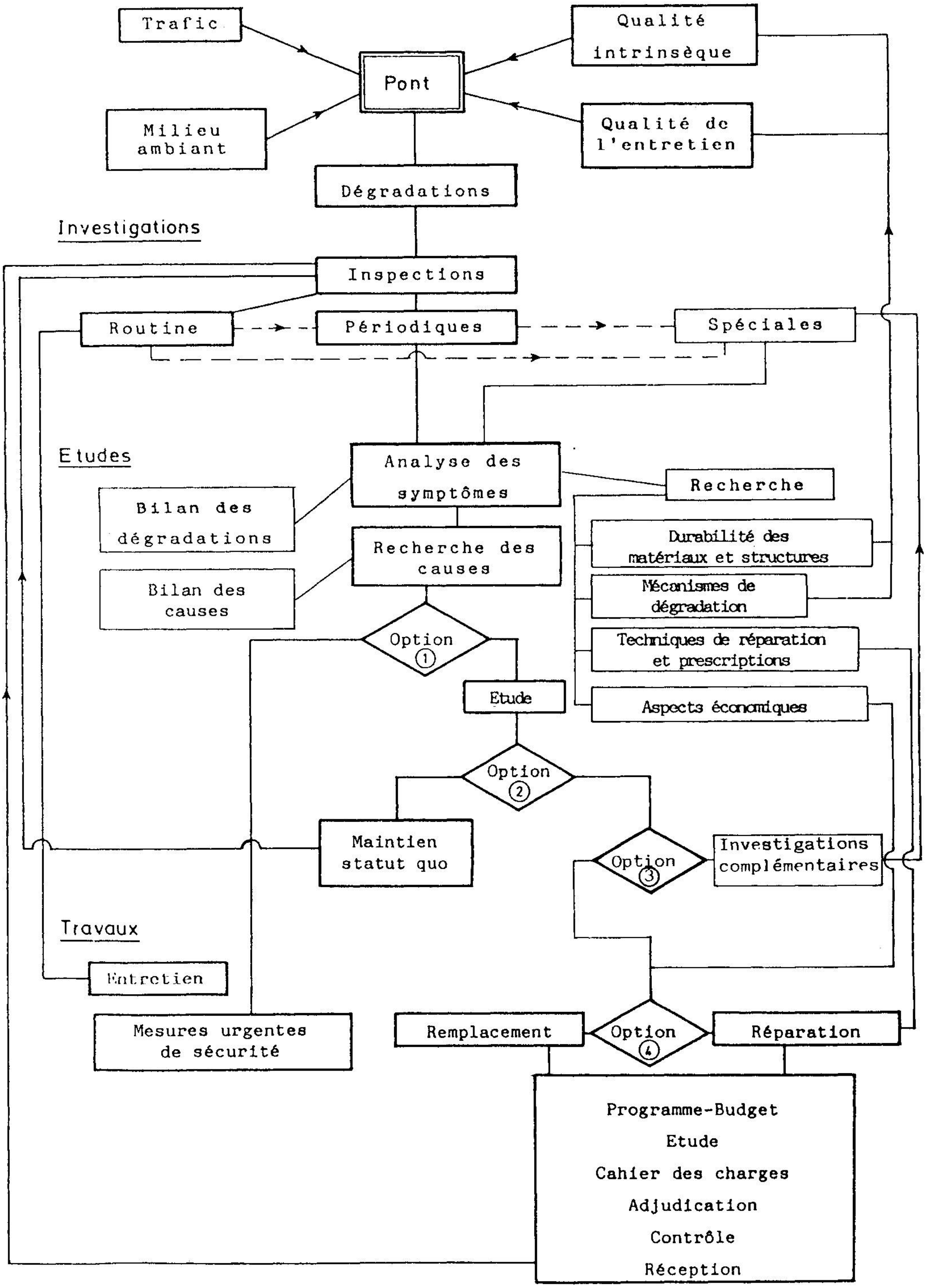


Schéma de la gestion des ponts. Figure n° 4

Tableau 1 Dégradations les plus fréquentes des ponts en béton et causes

Causes / Degradations – Fréquence (* *) – Importance (*)		Structures: flèche tablier	Structures: mouvements supports	Béton: fissures	Béton: dégradations gel/dégel	Béton: dégradations chlorures/sulfates	Béton: état de surface	Acier: corrosion acier B.A.	Acier: corrosion acier préc.	Acier/béton: dégradations interactives	Equipements: revêt.	Equipements: chape étanche	Equipements: joints	Equipements: appuis	Equipements: g.corps gliss.
		0	•	••	••	•	••	•••	•	•••	••	•	••	•	••
Conception															
- inadéquate	5			2					X			X	X	X	
- section arm. insuff.	5			1											
- enrobage insuff.															
- section béton insuff.															
Exécution/Matériaux															
- exécution non conforme au projet	3			4		3						X		X	
- exécution non conforme aux normes															
- qualité du béton insuff. (résistance/compacité)	5			7	2	2	X	2	X	X					
- retrait				5											
- enrobage/injection insuffisantes	6							3	X	X					
- gélivité	10				1										
- carbonatation								3		X					
- réactions alcali-silica	8			8											
Trafic															
- surcharge	7			3							3		X		
- fatigue	8												X		
- accidents/collisions															X
Environnement															
Eau de pluie						2		4	X	X					
- protection insuffisante	6				4	2			X	X					X
- drainage insuffisant	10														
- inondations/affouillements	8		2												
Tremblements de terre	9		3												
Vent	9														
Attaque															
- embruns/eau de mer	2							1		X					
- sels déneigement	1				3	1		1	X	X			X		
Effets thermiques	4			6											
Entretien	7						X				1		X	X	
Autres															
- usure											2		X	X	
- tassements du sol	10		1												
- graffiti							X								

* Importance des causes : 1, 2, 3, ... en ordre décroissant d'importance dans chaque colonne

x causes non classifiées, par manque de données

* * Fréquence des dégradations

0 nihil ou cas exceptionnels

• cas rares < 10 % des ponts inspectés

•• cas fréquents > 10 % < 50 %

••• cas très fréquents > 50 % des ponts inspectés