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SESSION 3

Rehabilitation and Repair of Bridges

Modification et réparation des ponts

Sanierung und Reparatur von Brücken

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Maintenance of Reinforced and Prestressed Concrete Bridges in the Federal Republic of Germany

Entretien de ponts en béton armé et précontraint en République Fédérale d'Allemagne

Erhaltung von Stahlbeton- und Spannbetonbrücken in der Bundesrepublik Deutschland

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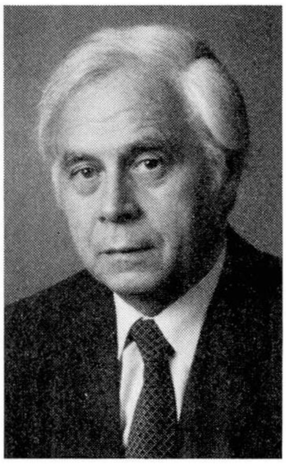

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SUMMARY
The maintenance of existing structures is gaining increasing importance in relation to new constructions. The funds for bridge and civil engineering constructions amount to approx. 50 mia. DM while the cost of their maintenance lies between 1,0 and 1,75 mia. DM per year. A more precise knowledge of expected costs is therefore absolutely necessary. For new building projects, more attention has to be paid in the future to the fact that economy is only guaranteed if costs are minimized for fabrication, maintenance, administration and utilization. Some examples of serious damage as well as interesting repair and renovation works, are given.

RESUME
L'entretien de constructions existantes gagne en importance vis-à-vis de la réalisation de nouvelles constructions. La valeur des ponts et ouvrages d'art est d'environ 50 mia. DM. Les frais annuels d'entretien de ces constructions sont estimés entre 1,0 et 1,75 mia. DM. La connaissance précise des frais futurs est absolument nécessaire. Pour de nouvelles constructions, il faudra se rappeler que la rentabilité n'est garantie que si un minimum de frais pour la réalisation, l'entretien, l'administration et l'utilisation est réalisé. Quelques exemples de dommages sérieux et travaux de réparation et de rénovation intéressants sont présentés.

ZUSAMMENFASSUNG
Die Erhaltung des Bauwerksbestandes gewinnt gegenüber dem Neubau zunehmend an Bedeutung. Das Anlagevermögen an Brücken- und Ingenieurbauwerken beträgt ca. 50 Mrd DM. Die Kosten für die Erhaltung dieses Bauwerksbestandes liegen schätzungsweise zwischen 1,0 Mrd DM und 1,75 Mrd DM jährlich. Eine genauere Kenntnis der zu erwartenden Kosten ist zwingend. Bei künftigen Neubauprojekten ist stärker darauf zu achten, dass die Wirtschaftlichkeit nur dann gegeben ist, wenn ein Minimum an Kosten für Herstellung, Erhaltung, Verwaltung und Benutzung verursacht werden. Einige Beispiele schwerwiegender Schäden sowie interessante Instandsetzungs- und Erneuerungsarbeiten werden aufgezeigt.

1. DEFINITIONS

The general term "preservation" stands for maintenance, repair and "renewal" (rehabilitation and replacement) and comprises the totality of all measures necessary to keep a structure in a safe and serviceable condition.

"Maintenance" comprises all current and partly periodical short-term and small-scale measures necessary for the preservation of the structure. They have no appreciable influence on the service balue of the structure. The measures include, for example, the cleaning of the drainage system or the routine maintenance of the bearings.

By "repair" we understand all measures of a larger scale for the restoration of the service value and the serviceable condition of the structure. It comprises, for example, painting, the repair of expansion joints, the renewal of pavement, the elimination of defects in concrete parts due to de-icing salt, and the injection of cracks.

"Renewal" comprises all necessary rehabilitation and replacement.

2. THE INCREASING SIGNIFICANCE OF MEASURES FOR PRESERVATION

2.1 Growing Bridge Stock

After World War II, the road network in the Federal Republic of Germany, especially the network of federal trunk roads, has been greatly developed. The federal motorways, and to some extent also the federal highways, were designed to have no intersections and were built with a generous allowance.

This contributed to the increase in the number of bridges, especially high and long valley bridges.

The development of the federal trunk road network and of the bridge stock during the past 15 years is shown in Fig. 1.

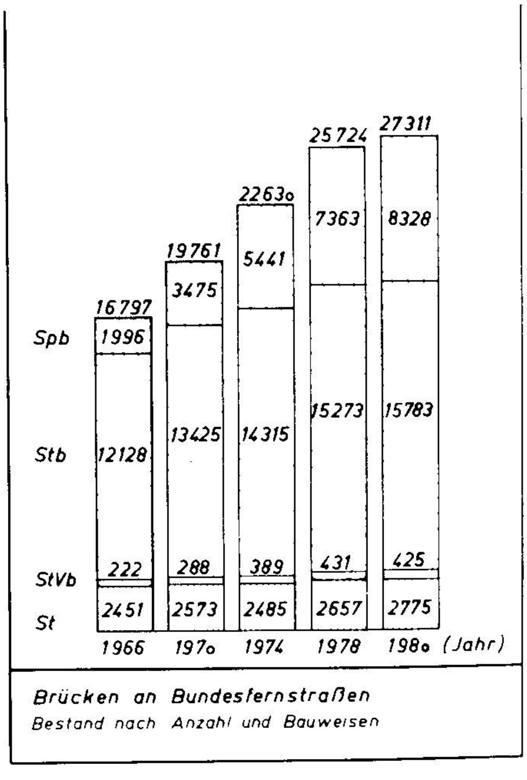


Fig. 1 Bridge statistics/number of bridges

2.1.1 Length of networks (km)

Road category / Year [1]	1965	1980	Increase 1965-1980
Federal motorways	3 204	7 292	4 088
Federal highways	29 906	32 248	2 342
Total	33 110	39 540	6 430

2.1.2 Number of bridges

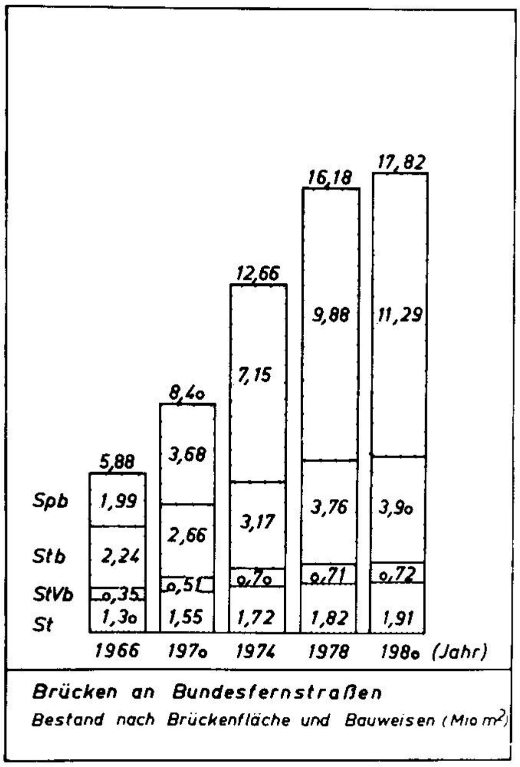
Construction method / Year [1]	1965	1980	Increase 1965-1980
Reinforced concrete	11 449	15 783	4 334
Prestressed concrete	1 725	8 328	6 603
Steel	2 357	2 775	418
Composite bridges	222	425	203
Total	15 753	27 311	11 558

[1] on 1 January of each year

2.1.3 Bridge deck area (in million m^2)

Construction method / Year [1)]	1965	1980	Increase 1965-1980
Reinforced concrete	2.19	3.90	1.71
Prestressed concrete	1.78	11.29	9.51
Steel	1.17	1.91	0.74
Composite bridges	0.32	0.72	0.40
Total	5.46	17.82	12.36

Approximately 11,500 bridges with a deck area of almost 12.4 million m^2 were built on federal trunk roads between 1965 and 1980. Thus, by 1980, the number of bridges carrying or spanning federal trunk roads increased to about 27,000, with their deck area amounting to 18 million m^2 (Fig. 2).



Almost 90 % of these (i.e. 24,000 bridges) consist of reinforced or prestressed concrete. The percentage of the deck area of these structures (approx. 15 million m2) to the deck area of all bridges on federal trunk roads is approximately 85 %.

Fig. 2 Bridge statistics/ bridge deck area

[1)] on 1 January of each year

2.2 Investment Value and Preservation Costs

There is a direct relationship between the preservation costs of engineering structures and their investment value and age. The investment value at present is estimated at 150 billion DM, about 50 billion DM of which accounts for bridges and other engineering structures.

To estimate the financial requirements for the preservation of the structures, we start with the assumption that at present, the annual costs of maintenance and repair - without rehabilitation and replacement - are between 0.75 % and 1.5 % of the investment value. Thus we obtain an amount of between 375 million DM and 750 million DM annually. If we further assume a service life of 50 or 100 years for these structures then we get additional costs for rehabilitation and replacement of 1 billion DM or 500 million DM annually. This sum will only be required, however, when the structures which today are still relatively new, have completed their technical service life and will then require approximately 10 % of the total highway budget for rehabilitation or replacement.

An exact assessment of the financial requirements of measures for preservation is not yet possible, since we are still lacking detailed bases for assessment in the form of records and evaluation procedures.

A particular difficulty in estimating the financial requirements for the preservation can be seen in the age structure of the bridges. About 40 % of all bridges are no older than 15 years! We thus lack long-term experiences with the new construction methods developed after the war, such as prestressed concrete, composite construction, orthotropic plates, cable-stayed systems, etc. Furthermore, the demand on bridges in recent times is growing due to the strong increase in heavy vehicle traffic or due to environmental factors such as de-icing salt and air pollution.

We consider it an urgent task, therefore, to develop - with the aid of a road data bank - an evaluation procedure which will enable us to draw up a requirement plan and to arrange the work to be done according to priorities.

At present, we are still lacking a uniform, i.e. a methodical, systematic, and data-oriented policy in the Federal Republic of Germany that would make it possible to keep the roads and all structures appertaining to them in a safe and efficient state at the lowest possible cost.

The yearly expenses for the preservation of bridges and structures on federal trunk roads has been recorded, for the first time, in 1981. Such data collection will, after a couple of years, permit at least a better estimation of the requirements. It must be remembered however, that expenses will not necessarily be equal to actual requirements, since actual expenses might be limited by financial restrictions, by technical and organizational difficulties or by difficulties due to staff shortages in connection with the preparation and execution of preservation measures.

In any case, within the framework of the envisaged data collection procedure we would want to obtain information on the following parts of and work on bridges:
- surfacing and water proofing
- expansion joints
- bearings
- concrete work
- structural steel
- corrosion protection
- guard rails, railings, drainage
- miscellaneous measures

3. CONSEQUENCES

3.1 General Consequences

The preservation costs depend mainly on the durability of the structure, that means upon its functionality, as well as on the quality of execution and the regular inspection and maintenance. The objective of the highway authorities must be to economize on the total expenditure for construction and preservation. When considering the matter from an overall economic point of view, the road user costs (e.g. on account of traffic jams and a higher accident rate caused by repair work) would have to be included in addition to the above-mentioned costs.

3.2 Consequences for Design and Construction

This general objective results first of all, in consequences for design and construction. Structures of inferior quality can be built cheaper, but they will as a rule, entail higher costs for preservation later. It is therefore often more economical to accept the higher initial costs of construction for more durable structures.

Only durable structures requiring the least amount of maintenance should be constructed ("easy-to-keep-in-good-repair structures"). According to the motto: "Prevention is better than cure" the following aspects should be kept in mind:
- Structural components should be generously dimensioned.
- The structures, construction methods, and materials should be suitable for site and working conditions (uncomplicated). (In the case of reinforced concrete structures, for example, sufficient space between reinforcement for placing and compaction of the concrete should be provided for.
- Continuous girders should be preferred to multiple single-span girders with numerous (often damaged) expansion joints.
- Large surfaces exposed to all weathers and requiring higher inspection and maintenance expenditure should be avoided. As an alternative to slab-and-beam bridges with multiple beam, slabs and hollow box girders which do not have these disadvantages should be used.
- Highly stressed and sensitive components such as expansion joints, bearings, and cables should be easily replaceable.
- Suitable means for the replacement of bearings should be provided for, because subsequent auxiliary means may involve enormous costs. The necessary, somewhat larger dimensions of the substructures do not significantly increase the cost of a newly built structure.
- All components should be easily accessible for the purpose of inspection and maintenance.

3.3 Consequences for the Construction Process

Our experience shows that the quality and thus the durability of structures can be influenced above all by careful quality control and construction supervision. At the centre of all efforts in this field must be the self-supervision of the manufacturer or contractor in his own responsibility. However, supervision and control by official agencies and by the owner cannot, as experience shows, be dispensed with, either.

4. SUPERVISION OF STRUCTURES

4.1 Inspection according to DIN 1076

The first and foremost precondition for the systematic preservation of a bridge is its regular and careful inspection. As early as in 1933, we had the standard DIN 1077 "Rules concerning the inspection of solid road bridges", and this was updated in 1959 by DIN 1076.

Fig. 3 contains the most important engineering structures which are inspected in conformance with this standard. Above all, noise barriers and trough constructions have increased considerably during the past years. The properties of the structures to be preserved are listed in the middle of the figure and on the right those documents that are needed for the inspection of the structures.

The next figure (Fig. 4) shows the types of inspection and the times and intervals at which these should take place. The most important inspection is the so-called main inspection, which is carried out every six years, during which the structure is put to the "acid test".

Fig. 3 DIN 1076; inspection structures/properties/ documents

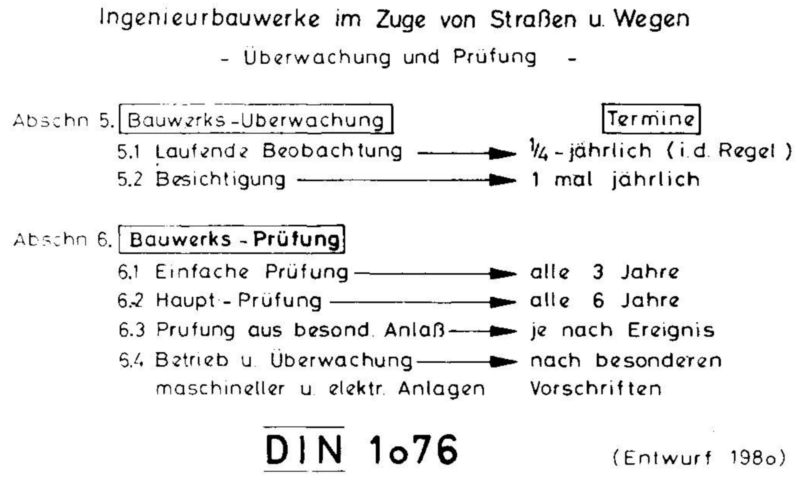


Fig. 4 DIN 1076; inspection intervals

Was ist zu prüfen ?

Gründungen

- Setzungen
- Unterspülungen, Auskolkungen

(2) Massive Bauteile

- Risse, Hohlstellen, Durchfeuchtungen
- Betondeckung, freiliegende Bewehrung
- Abplatzungen u. Risse parallel zu Spanngliedern
- Karbonatisierungstiefe

(3) Stahlkonstruktionen

- Risse, Beulen, Verbiegungen
- Schweißnähte, Anschlüsse
- Schrauben, Niete
- Korrosion (insbes. bei Kabeln, Seilen, Hängern)
- Ortsfeste Besichtigungseinr. (Stege Podeste, Leitern, Treppen)
- bewegl. Besichtigungseinrichtg (Bühnen)

Hauptprüfung nach DIN 1o76
(Entwurf 198o)

Fig. 5 DIN 1976;
subjects of inspection

Was ist zu prüfen ?

(4) Holzbauwerke

- Schrauben, Leimfugen, Anker
- Wassersäcke, Fäulnis

(5) Lager, Übergangskonstruktionen, Gelenke

- Beweglichkeit, Dichtigkeit
- Sauberkeit, Korrosion, Verformungen, Verankerung
- Planmäßige Stellung

(6) Abdichtungen, Fahrbahnen, Entwässerung

- Feuchte Stellen
- Vergußfugen
- Risse, Blasen, Hohlstellen, Verdrückungen
- Kappen, Schrammborde, Schacht-abdeckungen
- Straßeneinläufe, Entwässerungs-Leitungen

Hauptprüfung nach DIN 1o76
(Entwurf 198o)

Fig. 6 DIN 1076;
subjects of inspection

Was ist zu prüfen ?

(7) Wand- u. Deckenverkleidungen

- Befestigung
- Risse, Verformungen, Hohlstellen, Korrosion, Ausblühungen

(8) Lärmschutzwände

- Befestigung, Verankerung
- Risse, Verformungen

(9) Schutzvorrichtungen

- Schutzplanken
- Geländer
- Rauch-, Berührungs- Blendschutz-anlagen, Erdungen

Fig. 7 DIN 1076;
subjects of inspection

Was ist zu prüfen ?

(1o) Versorgungsleitungen

- Gas, Wasser, Öl, Schwach- u Starkstrom
- Befestigungen, Entlüftungen

(11) Vermessungstechn. Kontrollen

- Festpunkte, Meßpunkte
- Lichtraumprofile
- Verschiebungen, Neigungen, Biegungen am Bauwerk
- Gradienten-Lage

Hauptprüfung nach DIN 1o76
(Entwurf 198o)

Fig. 8 DIN 1076;
subjects of inspection

A survey of the focal points of the main inspection is given in Figs 5, 6, 7, and 8.

4.2 Organization of the Supervision of Structures

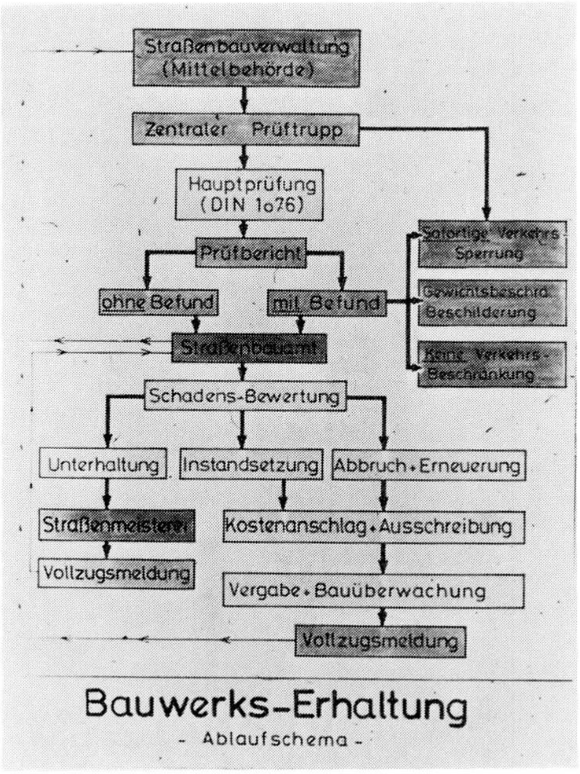


Fig. 9 shows how the supervision of structures is organized within our highway administration, taking the main inspection as an example. The main inspection is generally carried out by special inspection parties. Depending on the kind and the significance of the defects detected, the test results will set off different activities, which range from minor maintenance measures by the responsible road or motorway maintenance depot to the immediate closure of the bridge.

Fig. 9 Preservation of the structure - organization scheme

4.3 Inspection Equipment

A thorough inspection of all components of the structure above ground is only possible with appropriate equipment. Large and high bridges require equipment that can be driven as a vehicle on the road. Here we distinguish two different types, depending on their use:

- Figure 10 shows a double telescope with elevator and three joints with a working and inspection bucket for two men. The extension under the bridge is 9.10 m, the lifting height 28 m. Such equipment is particularly suitable for the local inspection of bearings, drainage installations, etc.
- For wide motorway bridges, inspection equipment with working platforms of up to 16 m extension under the bridge are necessary. Figure 11 shows such equipment in action. There can be no doubt that such platforms give much better access to underbridge areas and make it possible to do minor repair work.
- Other equipment, such as inspection trains from the German Federal Railways or an inspection ship from the Waterways and Shipping Administration, can be used for the necessary inspection of road bridges over electrified railway lines and across canals.

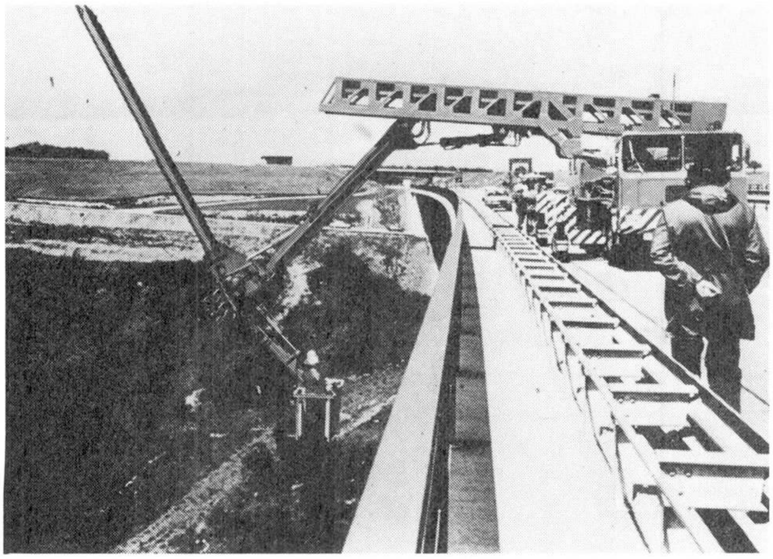


Fig. 10 Bridge inspection equipment Ruthmann double telescope with elevator and three joints US 260

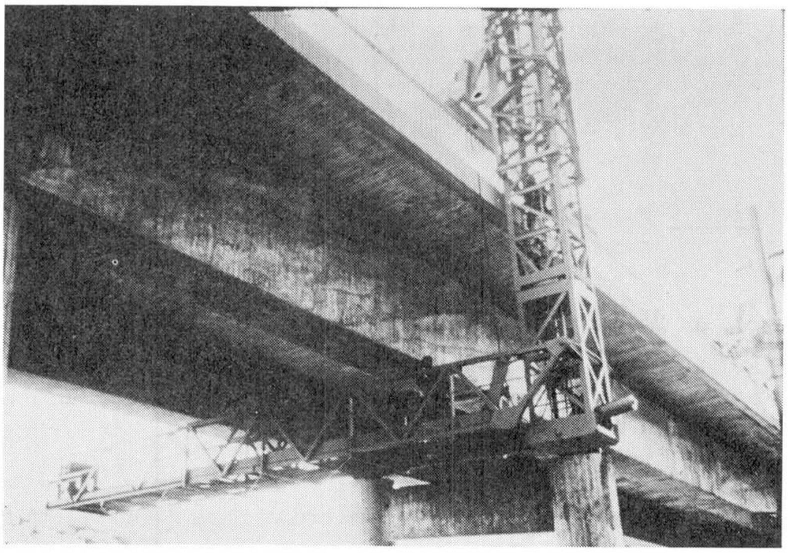


Fig. 11 Bridge inspection equipment Passarella

- However not only large equipment is needed for the inspection of bridges. About three quarters of all structures on federal trunk roads are bridges with a length of up to 30 m. A large part of these structures can be inspected with the aid of simple, often stationary means, such as ladders, platforms, catwalks, and through access openings.

5. DISCOVERY OF DEFECTS DURING INSPECTION

5.1 General

The regular employment of the bridge inspection equipment enables us to recognize damage as early as possible, thus ensuring immediate remedy. In fact, we have found that for the most part the structures are in a safe and serviceable condition, fulfilling the demands in this respect. We attribute this to the fact that highway authorities have always endeavoured to eliminate immediately all defects discovered in regular inspections.

5.2 Types and Distribution of Defects and Damage

The few defects that do occur can be broken down into the following types of damage, which have been found with the following frequency in one Federal Land during main inspections (according to standard DIN 1076) (Fig. 13):

- open reinforcement, rusting 17 %
- hollows/defects in the concrete of the superstructure 13 %
- cracks of more than 0.2 mm width in the superstructure 11 %
- damage due to de-icing salt along footpaths 14 %
- surfacing cracked, deformed 32 %
- joints in kerbs leaky 31 %
- waterproofing defect 21 %
- rails and posts rusting; sealing compound defect 58 %
- drainage intakes blocked 27 %
- bearings defect 14 %
- substructures: concrete deterioration, cracks 27 %

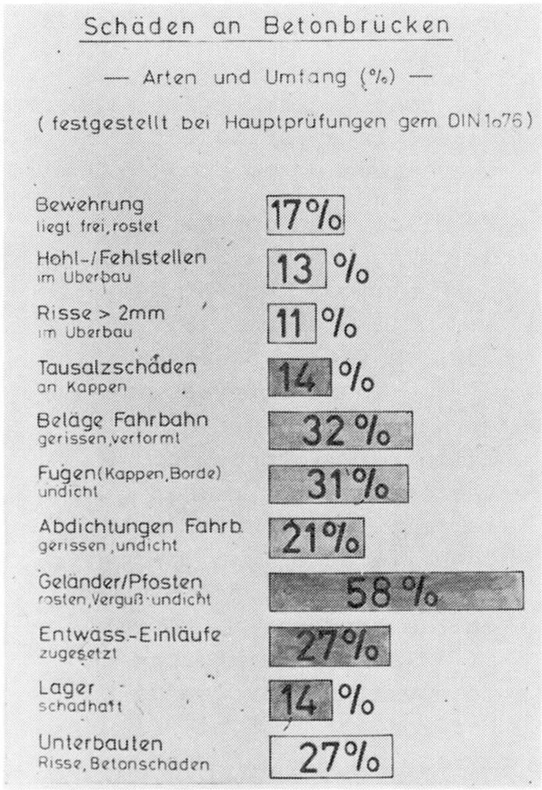


Fig. 13 Damage to concrete bridges; type and extent

These figures refer in each case to 100 inspected bridges, but they do not say anything about the extent and the degree of the damage, and are therefore limited in their statistical value.

According to the experiences gained by our highway authorities, we distinguish between three main categories of causes of damage:
- damage due to discrepancies in the design, incorrect assumptions in calculations, and faulty detailing;
- damage due to mistakes made during the execution of the constructions work;
- damage due to unforeseen or unforeseeable and thus unplanned stresses through utilization (traffic), operation (de-icing salt), and environmental influences.

Damage mostly results from a combination of these causes.

5.3 Serious Damage That Occurred in Recent Years

Let me show you now a few examples of damage to reinforced and prestressed concrete bridges that were found in recent years and which attracted much attention, but which should by no means be considered as representative of damage to bridges in general:

- About 10 % of the 8,300 prestressed concrete bridges on federal trunk roads have been built with coupling joints.

  Proof of the stresses due to non-uniform temperature through the cross-section of the superstructure not previously required, moments in the coupling joints not covered by reinforcement, as well as the lack of sufficient minimum reinforcement at the coupling joints caused serious cracks in some bridges, and in one case even the breaking of tendons at such a joint. It will probably be necessary in the long run to reinforce about 15 prestressed concrete bridges by means of additional tendons or bonded plates or bonded reinforced concrete plates.

  The costs of the repair and reinforcement work are not inconsiderable in these cases. In a particular case, e.g. the fly-over Prinzenallee in Düsseldorf, they amounted to about 5.5 million DM (Figure 14).

- We discovered another type of damage in the Schmargendorf bridge. This bridge was built between 1959 and 1961. A faulty assessment of the shear stresses together with insufficient ordinary reinforcement, the omission of reinforcement to counteract the effects of the indirect support of stringers at the cross-beam, as well as a lack of reinforcement to cover the forces due to horizontally curved prestressing tendons resulted, in this case, in severe cracking and corrosion of some tendons. Despite repeated rehabilitation and strengthening measures it was not possible to restore the service level of this bridge permanently. We decided, therefore, to dismantle the existing bridge and to build a new one at the same place for about 60 million DM (Figures 15 and 16).

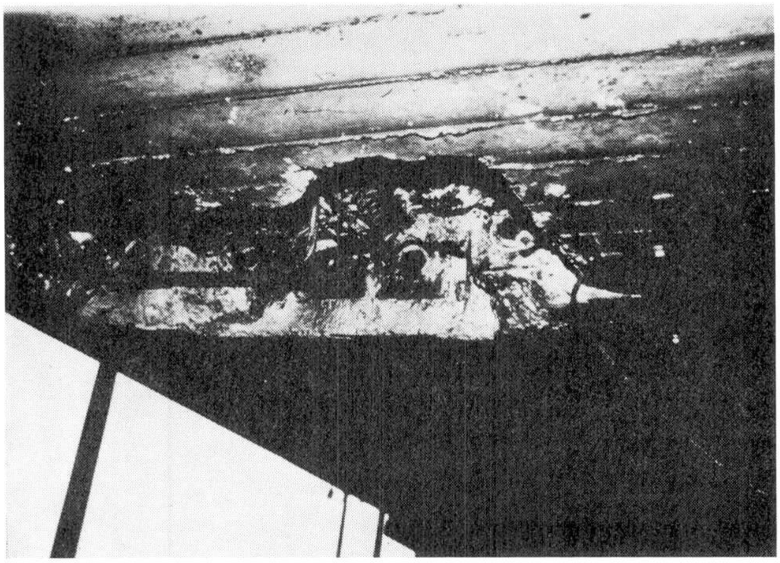


Fig. 14 Underside view of hollow box in the region of coupling joint - laid open (flyover Prinzenallee)

- Different causes led to extensive damage to three neighbouring 15-year-old valley bridges on the Krefeld - Ludwigshafen motorway west of the Rhine. Rehabilitation costs amounted to a total of 45 million DM. The bridges were built as a single-span girder system with prefabricated main girders. The numerous expansion joints required became leaky and to some extent broke under the high traffic loads. This enabled surface water containing de-icing salt to penetrate to the ends of the prefabricated girders and to cause severe damage there.

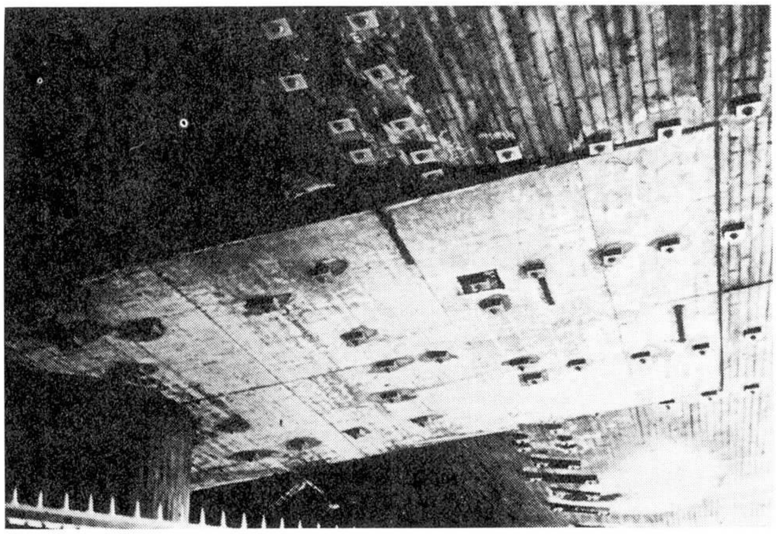


Fig. 15 Underside view of the region of pier with vertical additional tendons inside the structure (1st rehabilitation) (Schmargendorf bridge)

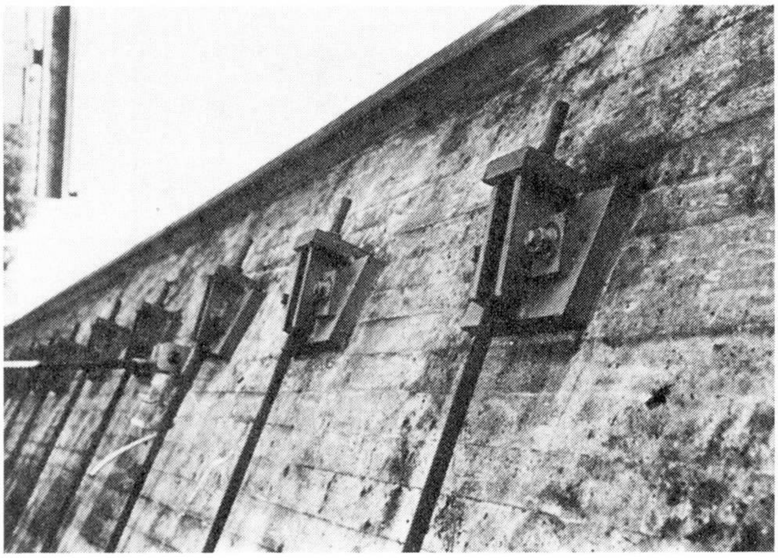


Fig. 16 Underside view with anchoring of outside additional tendons (1st rehabilitation) (Schmargendorf bridge)

A great number of cracks and surface scaling of the concrete, especially at the ends of the prefabricated girders and at the cross beams, constituted a risk to the stability at the points of support (Figures 17 and 18).

- As an example of severe damage caused by the failure of bearings I would like to mention the valley bridge Blockheide.

There, extremely low temperatures led to the roller bearings rolling off. The expansion movements were calculated in accordance with the code valid at the time the design was made. However they did not contain any calculational or constructional safety in view of the probable deformation values.

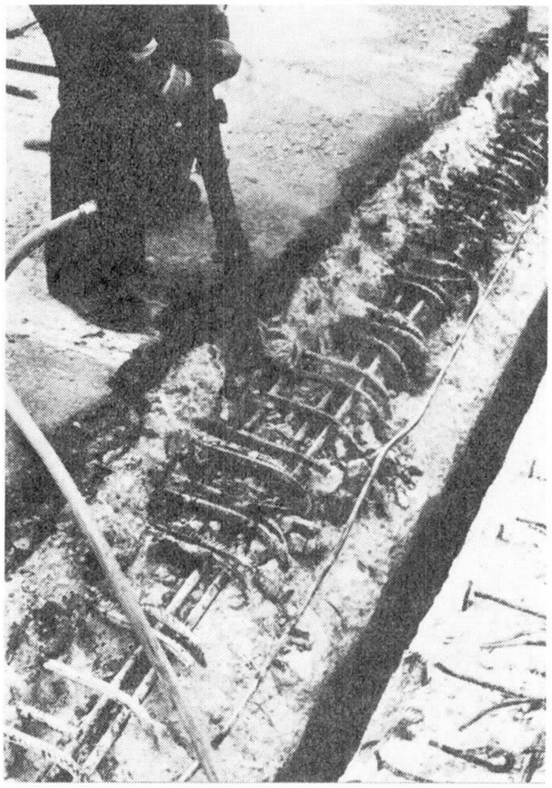


Fig. 17 Expansion joint removed (valley bridge Pfädchensgraben)

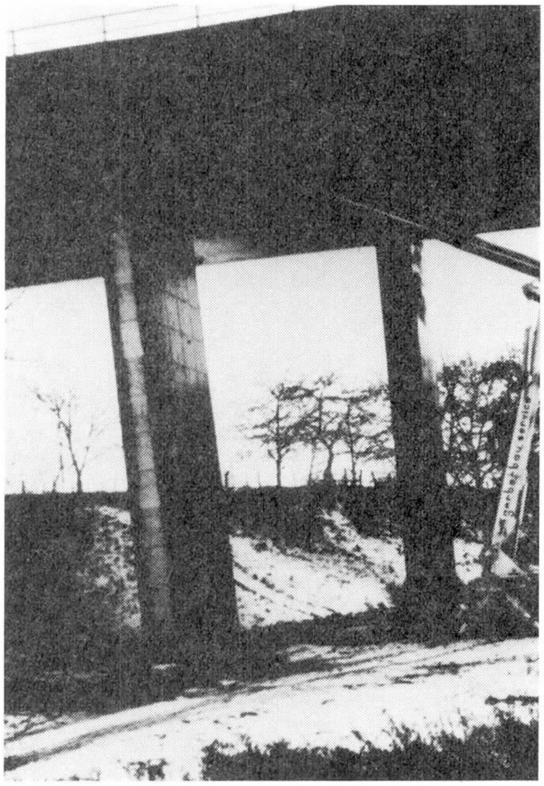


Fig. 19 Damage to the pier due to rolled off bearing (valley bridge Blockheide

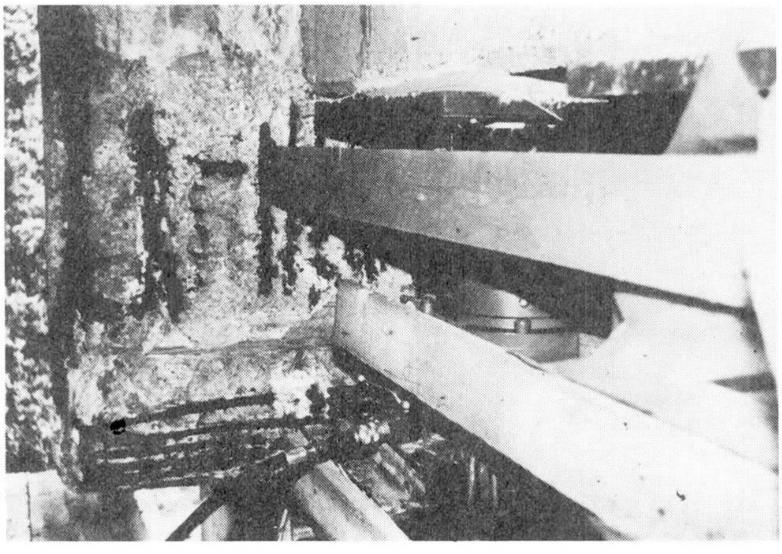


Fig. 18 Jacks under cross-beam to relieve longitudinal girder during repair work (valley bridge Pfädchensgraben)

Owing to the failure of a roller bearing the superstructure sagged by 27 cm which caused an eccentric load on the pier and entailed heavy damage to the super- and substructures (Figure 19).

5.4 Documentation of Damage

In order to be able to cope with the task of preservation of bridges both financially and organizationally, it is necessary - as was emphasized previously - to prepare an extensive survey and evaluation of the types and the frequency of damage. For this purpose, a first publication containing 84 cases of damage has been worked out by the Federal Ministry of Transport, and will be published shortly. (The above mentioned cases are contained in this publication.) It describes systematically and in the same pattern the structures concerned and the damage, specifies its causes, shows the rehabilitation, and gives possible advice for the design, its analysis, the work undertaken, the execution and supervision as well as bridge inspection. The purpose of such documentation, which will be extended as new cases of damage are recorded, is to sharpen the sensitivity of all parties involved in construction engineering - i.e. engineering offices, proof-engineers, construction companies, highway authorities, and students of construction engineering at institutes of technology, universities, and specialized institutes, to possible sources of damage. Such information is of particular importance to educational establishments, since the knowledge of possible damage to bridges, its causes, and its remedy has hitherto not been among the subjects taught.

I am also of the opinion, however, that analyses of the causes of damage should not only be exchanged at national level, but also at international level, and on a large scale and in good time, so that such damage can be avoided as far as possible in the future. A first approach in this respect can be seen in the work of Commission IX of the Comité Euro-International du Béton (CEB). This work should be supported and promoted by all countries.

6. EXAMPLES OF INTERESTING REPAIR AND REHABILITATION WORK

In the future, we will have to occupy ourselves increasingly with measures of repair, rehabilitation and strengthening. Suitable methods must be developed for carrying these measures through. These would have to take into account the fact that most of the work would have to be done while traffic is flowing, and that traffic should be interrupted as little as possible. In this context some research projects have already been commissioned by the Federal Ministry of Transport during the past years.

In case repair and rehabilitation are no longer possible, suitable methods for demolition must be developed before erecting a replacement structure, since in most cases explosives will be out of the question because of nearby traffic routes and buildings. Demolition procedures employing diamond saws, oxygen lances, or hydraulic loosening for large reinforced and prestressed components still have to be thoroughly tested.

Furthermore, difficulties often arise in connection with the tender and the award of the contract, since in most cases the exact extent of the damage can only be determined after the work has begun (leaky waterproofing, not grouted and broken tendons) and, unlike with a new bridge, the work involved is often difficult to estimate.

I would like to discuss now some examples of more frequently occurring or particularly interesting repair and rehabilitation work:

(a) Rehabilitation of defects due to chloride/scaling of concrete (Figures 20 - 23)

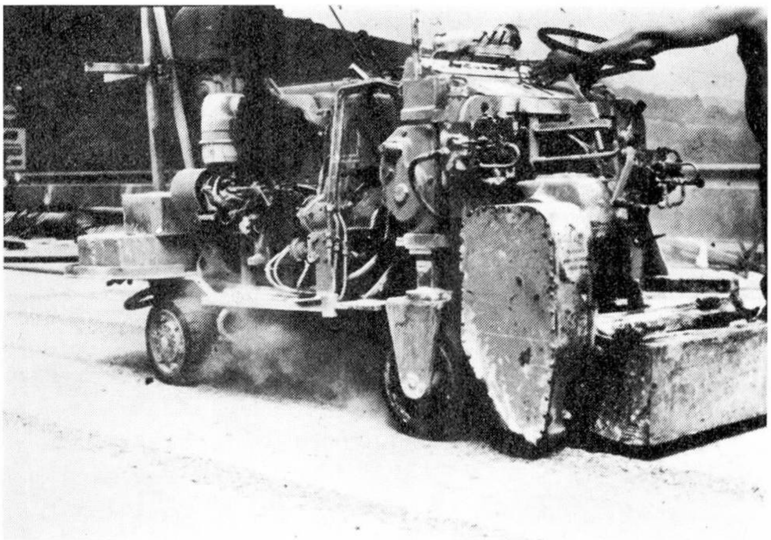


Fig. 20 Mechanical removal of carbonated top concrete layer from carriageway (bridge over the river Nahe at Dietersheim)

Fig. 21 Spreading of the bitumen epoxy resin waterproofing by hand (bridge over the Kocher valley at Geislingen)

- In the case of damage to the concrete deck slab due to chloride, both the pavement and the waterproofing must first be removed. In case of surface contamination the concrete is either treated mechanically or by high-pressure water. If required, deteriorated concrete should be removed. In case of contamination to greater depths, contaminated concrete is completely removed and replaced by new concrete with resin applied as a bonding agent. After that a bituminous waterproofing or an elastic plastic coating is applied by spraying. The advantage of the spraying method is that subsequently occurring cracks can be bridged and the structure thus be permanently protected.

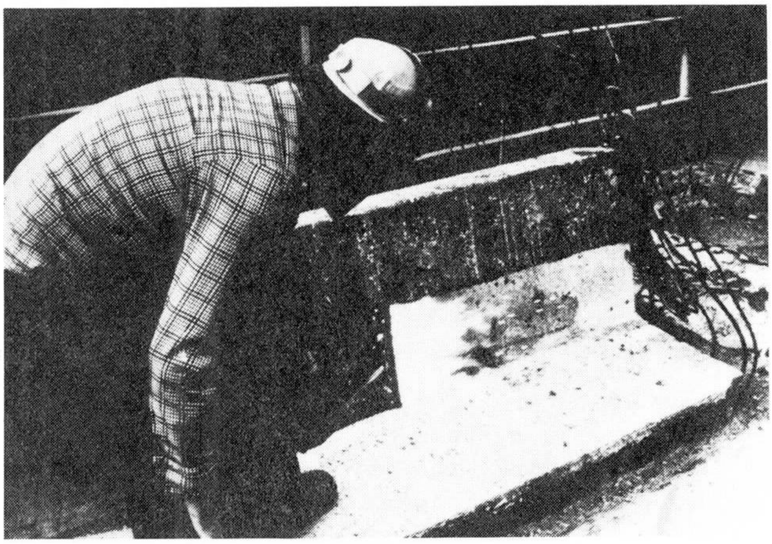


Fig. 22 Flame-cleaning of carbonated top concrete layer of concrete guard kerbs (valley bridge Pfädchensgraben)

- Chloride-contaminated surface concrete is often removed from small horizontal and vertical areas (concrete footpaths, kerbs) by flame cleaning; exposed reinforcement will receive a double corrosion protection.

  Damaged parts of the concrete will be filled with resin mortar, especially where the concrete protective covering of the reinforcement has to be strengthened. After that footpaths and kerbs receive, as a rule, a cloride-resistant coating.

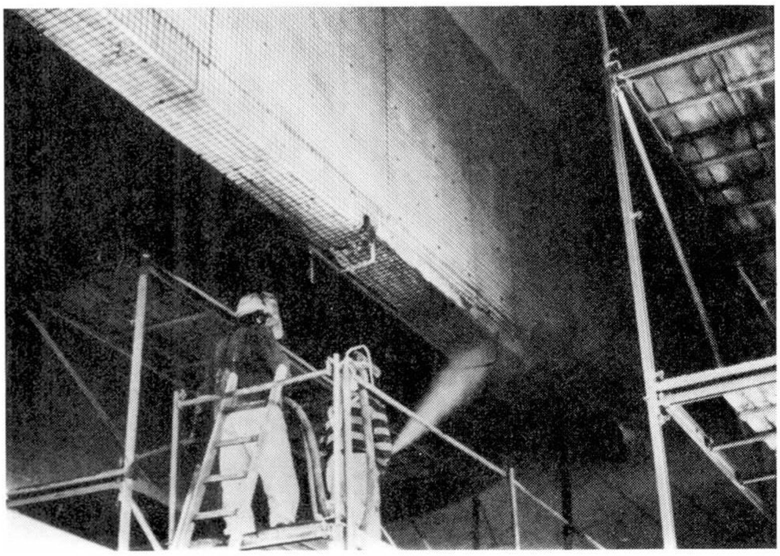


Fig. 23 Restoration of the required dimension by means of shotcrete (bridge over the river Lahn at the Taubenstein)

- In the case of defects in load-carrying concrete components, the loose and the chloride contaminated concrete is removed and the reinforcement sandblasted and corrosion-protected by a double coating. Then the concrete is replaced by shotcrete to the required dimension.

(b) Injection of cracks (Figure 24)

For the sake of durability cracks in reinforced concrete components should be sealed or injected if there is a potential danger of corrosion of the reinforcement. Besides providing corrosion protection, the objective of injection of cracks in load-carrying components is to close the cracks in such a manner as to enable the concrete cross-section to withstand tensile stress. (Restoring the condition of uncracked concrete).

Only cold-hardening, solvent-free epoxy resin systems are suitable for injection work. The crack width at the place of injection must have a minimum width of 0.1 to 0.2 mm in order to achieve a successful injection. Injection of cracks is then possible for crack widths up to less than 0.02 mm. Before injection the crack must be sealed at its surface.

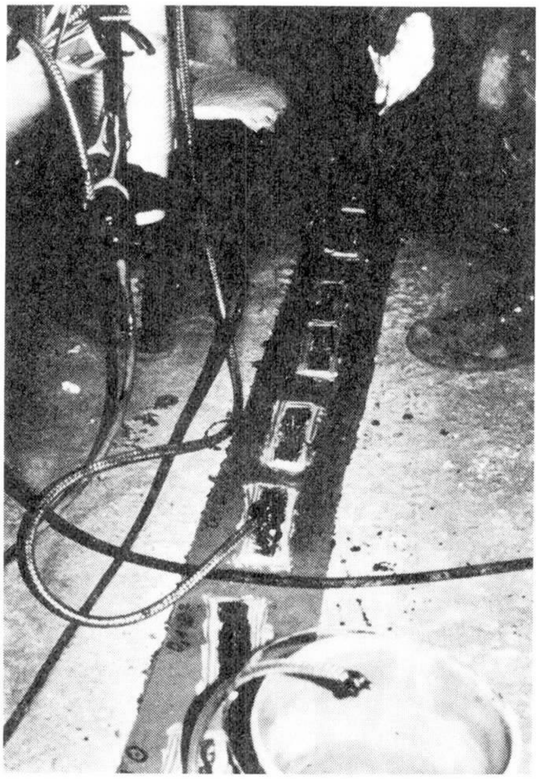


Fig. 24 Injection of joints with epoxy resin (bridge over the Lahn valley near Limburg)

(c) Rehabilitation of damage in coupling joints (Figures 25 and 26)

Besides the method of crack injection, as mentioned above, it may become necessary to increase the amount of ordinary reinforcement in the region of coupling joints in order firstly, to reduce the stress variation in the coupling elements and secondly, to minimize the crack width of new cracks under repeated loads. Two methods are under investigation (within the framework of research initiated by the Ministry of Transport): strengthening by bonded steel plates and strengthening by bonded reinforced concrete plates.

In the case of bonded reinforced concrete plates, the surface treatment is such that mechanical bond is achieved. Additional anchorage is provided by drilled-in threaded bolt and nut anchors. Reinforcement is laid and secured on the anchors and the concrete placed.

In the case of strengthening by bonded steel plates, the concrete surface is properly cleaned and prepared for bonding and the surfaces of the steel plate are sandblasted to a metallic clean surface condition. The steel plates are bonded to the concrete by epoxy resin adhesives.

Another possibility for strengthening prestressed concrete components in the region of coupling joints is the application of outside prestressing elements.

(d) Replacement of bearings (Figure 27)

A relatively frequent repair measure is the replacement of bearings. Damage to bearings could be the result of:

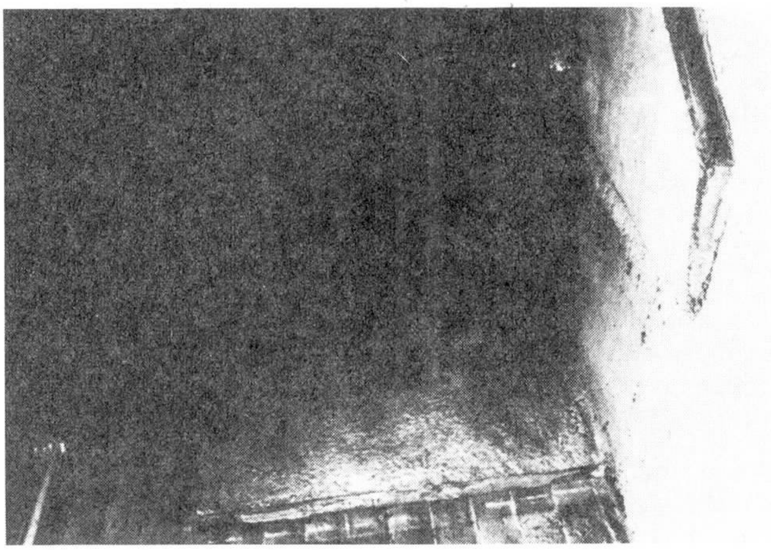


Fig. 25 Strengthening of coupling joints by means of bonded reinforced concrete plates in the hollow box (bridge over the Lahn valley near Limburg

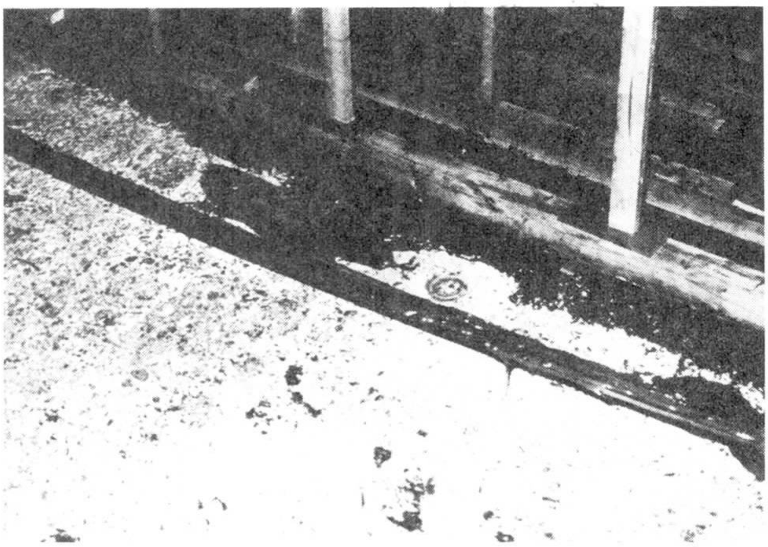


Fig. 26 Application of the steel plate (valley bridge Sterbecke)

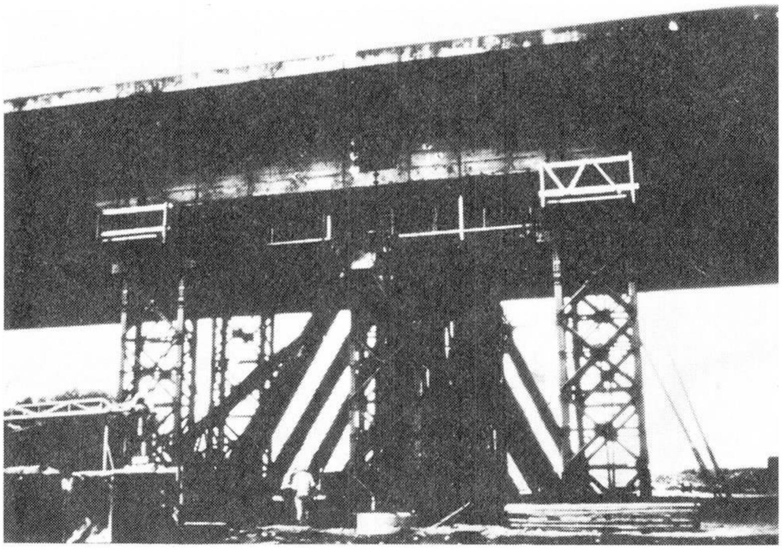


Fig. 27 Erecting tower for the replacement of the bearing (bridge over the river Main at Hochheim

- exceeding the limits of foreseen movement (rolling off the bearing plate),
- fracture of the rolls,
- overstressing and failure of elastomeric bearings,
- failure of neopreme pot bearings due to fracture of the sealing ring or insufficient lubrication or sliding surface area.

Lifting the superstructure and exchanging a defect bearing is a relatively simple operation if suitable means for placing jacks have been provided for. Extensive and costly provisional facilities are often required where jacks cannot be employed directly.

7. SUMMARY

To sum up:

- As the preservation of existing structures gains more importance over new construction in the future, attention should be paid to all questions of preservation.
- It is necessary to introduce and maintain on a wide basis, an exchange of experience on all types of damage and their causes as well as their rehabilitation. Both the administration and the building industry must have the necessary theoretical knowledge or must acquire it.
- Finally, it is indispensable that the necessary funds for the preservation of existing structures be provided.
- Only if the afore mentioned demands are met will it be possible to carry out the immense task of preserving the existing bridge stock in a satisfactory manner.

Experiences in Rehabilitation and Repair of Concrete Bridges

Expériences de rénovation et de réparation de ponts en béton

Erfahrungen in der Sanierung und Instandstellung von Betonbrücken

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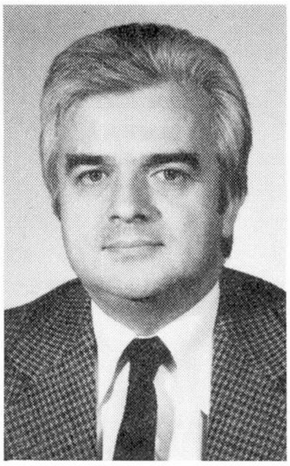

Aleksandar Pakvor, born in 1934, graduated in civil engineering from Belgrade University, where he also got his master's and doctor's degrees. As assistant professor of Concrete Structures, he is engaged in teaching, research work and professional activities.

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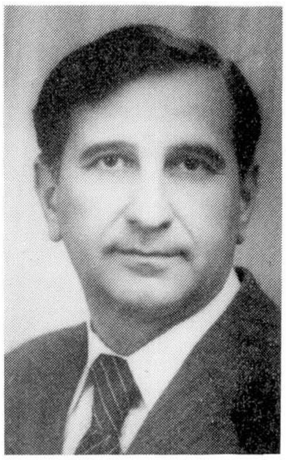


Živojin Darijević, born in 1928, graduated in civil engineering from Belgrade University in 1954. As assistant professor (1968) of Concrete Bridges, he is engaged in teaching, research work and professional activities.

SUMMARY
The serviceable value of a bridge, its durability, costs of maintenance and repairs are closely associated with the quality of the design and with the quality of the works executed. Proceeding from the basic principles of bridge designing, a prerequisite for a successful bridge, the possible causes of damage occurring on the bridge have been discussed. Besides the classification of damage, the gravity of its consequences have been commented. Some examples of rehabilitation of reinforced and prestressed concrete bridges illustrate the present considerations and viewpoints of this problem.

RESUME
L'utilisation et la durée d'un pont, les frais d'entretien et de réparation sont étroitement liés à la valeur du projet et à la qualité des travaux effectués. En partant des principes fondamentaux de l'élaboration d'un projet de pont, condition préalable de tout pont réussi, on a analysé des causes possibles de l'appariton des endommagements sur les ponts. On a également commenté leur classification ainsi que l'importance de leurs conséquences. Plusieurs exemples de rénovation de ponts en béton armé et précontraint sont présentés.

ZUSAMMENFASSUNG
Die Gebrauchsfähigkeit einer Brücke, ihre Dauerhaftigkeit, Unterhalts- und Instandstellungskosten stehen mit der Entwurfs- und der Ausführungsqualität in enger Verbindung. Nach den Grundprinzipien des Brückenentwurfs, Vorbedingungen einer erfolgreichen Brücke, werden die möglichen Gründe der erfolgten Schäden diskutiert. Neben der Schadenklassifikation ist die Schwierigkeit der Folgen analysiert. Einige ausgewählte Beispiele der Sanierung von Stahl- und Spannbetonbrücken werden vorgestellt.

Bridges are expensive and significant edifices that should serve for ages. When eliminated from use, consequences and damage are unpredictable. Their durability and costs of maintenance and repairs are connected with the quality of the project and the quality of work performance.

In the Belgrade school of concrete bridges, from the very beginning of its existence, emphasis has been put on the principles of the art of bridge designing - *objectivity, functionality, stability, rationality, originality and aesthetics.* The concept of stability implies not only stability in the narrower sense of the word, but the stability of everything that has an influence on change in the state of the structure as it was designed, associated with its purpose, functional requirements, traffic safety, durability and appearance, which is quite in agreement with the contemporary conception of the limit states. Successful can be considered only the bridge in which all the principles mentioned have simultaneously been achieved,

Defects (error is a heavy word) which may occur during the life of a bridge according to their cause are: *defects in designing, defects in construction, defects associated with maintenance,* and they may occur singly or in combination, or altogether.

Defects in designing are numerous: *inadequate choice of the static system, omissions and mistakes in static analysis and irregularly or badly solved structural details and fittings.*

Inadequately chosen static - structural conception in most cases is reflected on the rationality of the bridge, but it may also be the cause of serious functional defects - disturbance of functional, and even of general stability (girder or frame with cantilevers, shallow insufficiently rigid three-hinged arches, incorrectly solved cross-sections, etc.).

Omissions, overlooking and errors in static analysis may be very heterogeneous, from the most common ones, those of algebraic order, but with possible serious consequences, through glaring errors of static order, up to the badly estimated visco-elastic properties of concrete, which disturbs all the three aspects of stability. An experienced designer notices them during his work already, and corrects them in time, but they may be slipped by an inexperienced designer and be discovered only when the structure has been built, if the errors are of a somewhat more serious nature, or sometimes accidentally after longer use (when the bridge is tested for the passage of particular vehicles). Emphasis should also be put on the essential difference between the structures of reinforced concrete and those of prestressed concrete. In prestressed concrete structures one should know static influences much more precisely, especially in statically-indeterminate systems and bridges with a complex geometry in the plan (skewed and curved bridges). Computer technique has given a new dimension in designing and estimating structures. Besides the more precise and very fast computation, it offers the possibility of solving very complex, formerly inconceivable, problems. However, it requires programme testing, and in more complex structures also an adequate interpretation of the actual structure in a design model.

Incorrectly or badly solved structural details may occur as the consequence of not seeing the perspective development of traffic, of insufficient experience, or of excessive desire to present the structure as cheap as possible, as well as desire to overstimate the potential possibilities of the system. These incorrectnesses are reflected on the functionality and durability of the bridge, increased costs of maintenance and repairs, and even on the general stability (incorrectly solved cross sections, incorrect orientation of movable bearings of the bridge in a curvature, application of thin members, etc.).

Fittings are elements which do not involve the static-structural conception of the bridge, but essentially affect the regular serviceability and durability, What is the use of a beautiful, rational and grandiose bridge, if water drainage is not correct, so that various mishaps and accidents occur, or the solution of expansion joints is bad, which makes driving unpleasant and even risky, or water-proofing is left out or poorly solved, which shortens the life of the bridge, etc.

Defects in bridge construction may be numerous and heterogeneous, and they are generally: *nonachieved qualities of basic materials (mostly of concrete) and errors in construction.*

Under nonachieved qualities of concrete we imply not only lower strength, but all the irregularities in concrete placing (badly constructed break of concreting, local segregation, insufficient compaction, etc.). In prestressed concrete, this group also comprises irregularities in connection with the protection of cables from corrosion and their bonding with the basic section. The above mentioned defects need not be reflected directly on stability but on durability (resistance to frost or aggressive media, etc.) and vice versa (slender members and the like).

Errors in construction are all deviations from the design: *impreciseness of shuttering and falsework* which results in a change of structural geometry with all possible consequences (increase in steady load, displacement of the centre of mass, change in rigidity, undesirable eccentricity in high piers, inaccurate vertical alignement, etc.), *incorrect position of reinforcement and cables, insufficient protection layers, bad rectification of the bearing,* and the like, with far greater sensitivity of prestressed concrete structures. The gravity of consequences depends on the degree of deviation and may seriously disturb all the three aspects of stability.

Unfortunately, there are still opinions that concrete bridges are everlasting and that they do not require maintenance. It is not worthy of comments. However, *irregular and poor maintenance* may also be dangerous. Irregular maintenance, first of all, affects the serviceable value of the bridge, which must not be underestimated. It may be directly reflected both upon the state of structure (damage) and upon its stability - joint blocking, expansion action of ice in the joints, vehicle bumping into the structural members due to unrepaired bumper rails, scouring of piers due to change in the watercourse regime (sinking of vessels in the vicinity of the bridge, and the like), soil displacement caused by subsequent actions, etc. This group of defects is worthy of exceptional attention, because the problem of maintenance is often underestimated, and is connected with administrative difficulties - financing. The number of bridges requiring intervention is not small. Such a state is worthy of attention and requires a serious, expert and even scientific approach to the problem.

Dominant moments in *rehabilitation* of bridges and other structures are: *prompt discovery of deterioration,* which is not at first directly noticeable, and when observed - the consequences are more serious, and it may also be too late (rigid fractures, specificities of prestressed concrete structures, etc.), *determination of actual cause of damage* (possibly also of the history of the occurrence). These elements are an essential prerequisite for the successful rehabilitation of damage on bridges - an analogy to diagnosis/therapy in medicine.

The presented considerations will be illustrated by examples of rehabilitation of some bridges from the authors' practice. For understandable reasons the names of bridges, names of designers and contractors will not be mentioned, because our intention is not to judge, but to learn a lesson and to get new experiences.

Bridge A (Fig.1) is from an older generation of prestressed bridges in Yugoslavia, classified into a group for detailed control. The low building height induced the solution of the arch system above the prestressed two-way ribbed deck structure, being a tie at the same time. Suspenders are precast and prestressed. Such solutions are abandoned today. The arch is not indicated for small spans, and the conception with a suspended deck is a forced solution and belongs to the past.

During the detailed surveying vertical cracks almost on all suspenders were observed. In view of the vital significance of these members, possible causes of deterioration, the actual state and the necessity of interventions were analysed in detail. The results of the examination of concrete showed that the required

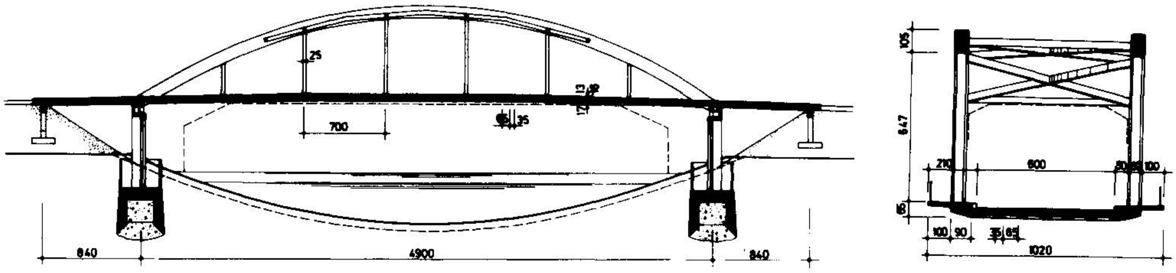


Fig. 1 Bridge "A", general arrangement

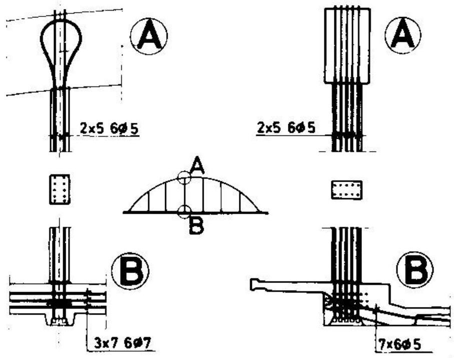


Fig. 2 Suspender cables

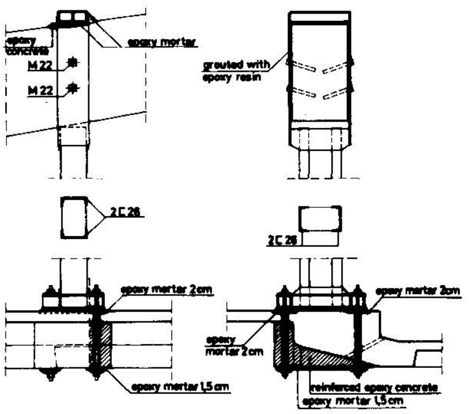


Fig. 3 Suspender rehabilitation

strength was achieved. The examination of cable corrosion showed that the protection of cables (grouting) was not satisfactory. Cables were seized by severe corrosion, with a considerable reduction of cross section. A considerable contribution to this state was made by a detail to which not sufficient attention was paid during the construction (Fig.2). After the grouting, control pipes on the arch extrados were cut off without particular treatment of these places. The insufficient compaction of the grouting mass enabled with time the penetration of weathering waters into the lower parts of suspenders, and the corrosion cables.

When cables were opened, expansive action due to corrosion of frost was not detected. The cause of the appearance of longitudinal cracks was the low percentage of reinforcement, particularly of stirrups, at high stresses of pressure in the concrete.

The dense arrangement of cables in the deck structure, as well as the impossibility of simultaneous release of the existing suspender cables did not allow the incorporation of new cables. Therefore, the suspenders were strengthened by rolled steel U-bars (Fig.3), which were partially activated by prestressing of high-strength bolts on the connection with the deck structure and with time they will completely overtake the role of corroded cables.

In Bridge "B" (Fig.4) all the basic forms of the above mentioned defects occurred (inadequate static system under today's conditions of traffic, defects in construction and defects connected with maintenance).

In a detailed examination of the bridge, it was established that the cracks of the main girderabove the medium supports, registered and injected already during the inspection of quality of work (20 years ago), were 0.1 - 0.3 mm wide and were still effective. Also, cracks were discovered in the central part of the middle span in the lower zone of the upstream box, as well as vertical cracks on the walls of piers through which cables were embedded. By detailed examination and analysis it was found that vertical cracks of piers were the result of the low percentage of reinforcement, particularly of transverse one. Cracks of the main

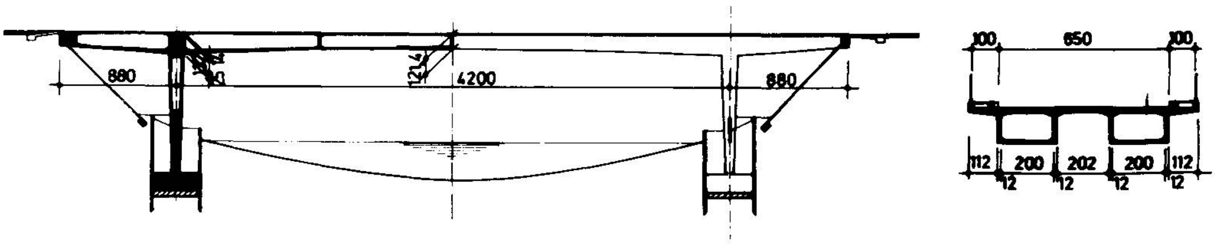


Fig. 4 Bridge "B", general arrangement

girder in the central part of the span indicated a far more serious situation, also after heavy vehicles crossings the bridge, it vibrated for a long time. As during the bridge construction openings for box inspections were not left, they were now immediately made. On entering the upstream box, the surprise was not small. One of the four groups of all bridge cables running directly close to the pier, was almost completely broken due to corrosion (Fig.5). Although cable protection, practically along the whole length, was well and efficiently carried out, above the pier, where cables entered the carriageway slab (Fig.6), there was a small zone in which the cable protection could not be efficiently performed. However, the absence of waterproofing and the appearance of cracks in the carriageway slab were enough to produce serious damage which was discovered late, but still on time.

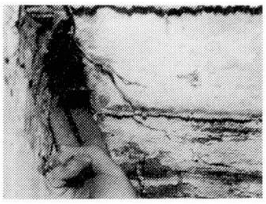


Fig.5 Cable damage

This is an example of two defects in structural details which could have been fateful: cable protection in the zone of piers is always very difficult to carry out and openings for the boxes were not constructed, which made periodical inspections and earlier discovery of damage impossible. The applied arrangement of cables, formerly used very often, has been abandoned throughout the wourld as well as in Yugoslavia.

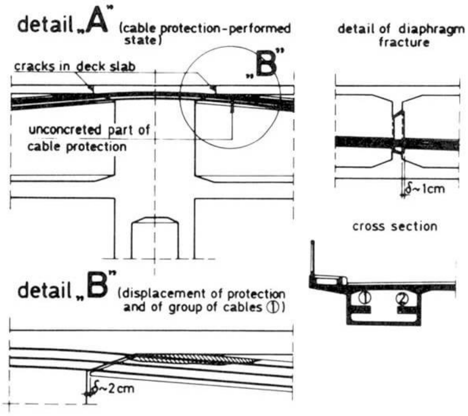


Fig.6 Cable damage arrangement

The bridge, because of seriously jeopardized stability, was excluded from traffic and rehabilitated according to a particular design. Cables of the girder were completely replaced, and supports were formed at either end of the bridge, so that the system for live load was transferred into a continuous frame with three spans. Cable replacement was made without a special falsework by temporary underpinning of the main girder by external cables.

Bridge C (Fig.7), across the channel, is a reinforced concrete skew frame with cantilevers. It was reconstructed during World War II on the modified foundations of a destroyed, similar bridge from 1939. The bridge conception, with comparatively large cantilevers and considerable deflections at the ends, with a small width of the carriageway, with steep bridge approaches in sharp horizontal curvatures and without transition slabs, does not correspond to the contemporary principles of bridge construction and to the conditions of today´s traffic.

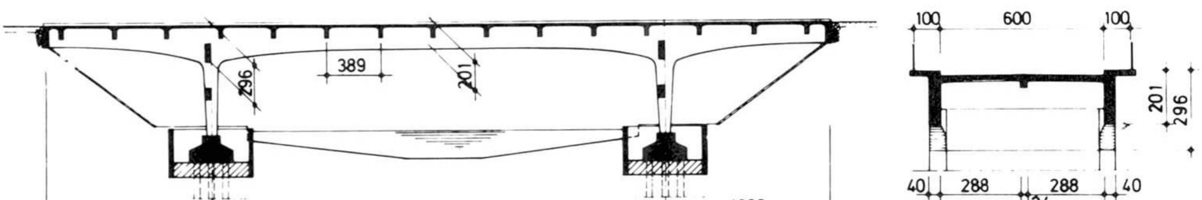


Fig.7 Bridge "C", general arrangement

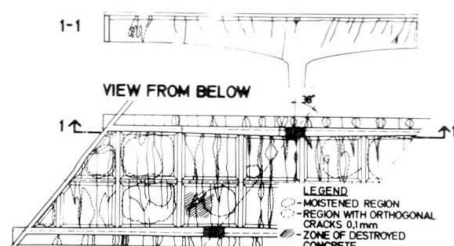


Fig.8 Detail of cracks

The bad state of the bridge, established by a former examination, required an urgent detailed examination and analysis. The state of cracks in the structure was observed, concrete quality examined and the general state of all the bridge members was established. A great number of cracks, mostly of nonpermissible width, were registered. A particularly bad state was found in the carriageway slab (Fig.8), made without waterproofing, where almost all cracks were moistened, and individual parts of the slab were so cut by cracks that only the reinforcement prevented the separated pieces of concrete to fall out.

This fact required an urgent intervention. Opposite to the author's opinion that the best solution was to construct a new bridge, nevertheless, for certain reasons (of administrative nature) it was concluded to rehabilitate the existing bridge. The old carriageway slab was removed and a new one made (Fig.9). The connection between the new slab and the existing main and transverse girders was achieved through the existing reinforcement, undamaged owing to very careful work, as well as through new anchors, placed in drilled holes and sealed with epoxy resin. The new slab was concreted step by step, taking into account symmetrical development of work. Cracks of the existing girders were injected with epoxy resin, according to a special procedure, and the structure was transversely and longitudinally prestressed by introducing moderate stresses of pressure. Longitudinal prestressing required a reconstruction of transverse girders at either end of the bridge in order to obtain anchor blocks. Finally, transition slabs were made, and before constructing a new pavement, waterproofing was carried out.

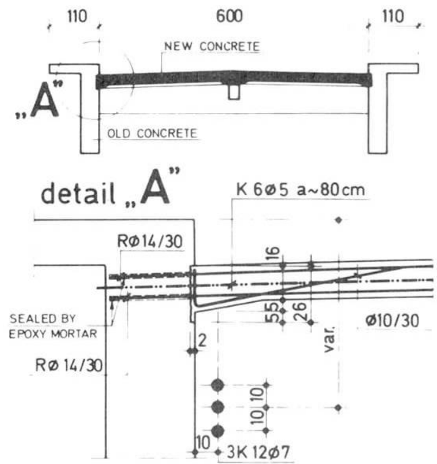


Fig.9 Replaced deck slab

Bridges D, at the approach to a city bridge across a large river, are prestressed continuous frames with slender columns. The cross-section consists of two boxes connected with a carriageway slab. These bridges belong to the first prestressed bridges in Yugoslavia and are loaded with all "childhood diseases", which is not the case only in our country. Here certain defects appeared in designing (thin elements at the cross-section, a small amount of reinforcement, cables inside the boxes, even without protection pipes, in construction (unattained design force of prestressing, insufficient cable protection), and in irregular maintenance.

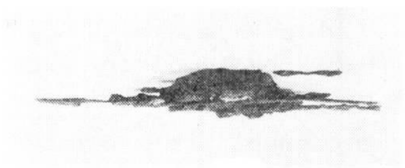
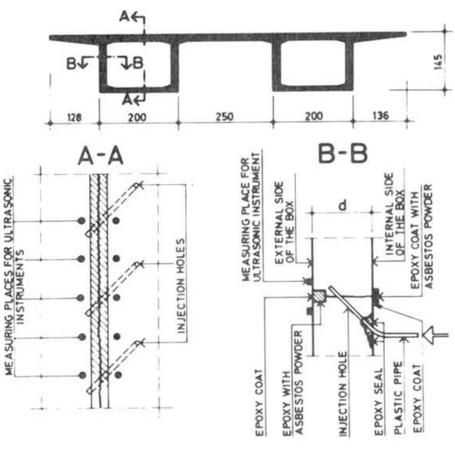


Fig.10 Cable destroyed by corrosion

Even before the bridges were put into operation, four structures, due to the appearance of cracks, were improved by additional prestressing with cables inside the box. Subsequent detailed observation revealed a comparatively large number of cracks, as well as an unattained prestressing force and badly protected cables, attacked by corrosion, and even completely broken (Fig.10).



The state established required urgent rehabilitation. Cracks were injected with epoxy resin, according to a special procedure (Fig.11). Bridge rehabilitation required additional prestressing with cables inside the boxes anchored in specially constructed blocks. Also, because of irregular maintenance, other repairs had to be carried out.

All these bridges were successfully rehabilitated and, after test loading, were put into the regular operation.

Our approach to this problem is from the aspect of designers, as well as from our experiences acquired in rehabilitation of bridges, with the intention to contribute to a more successful solution of this problem.

Repairs on the Givors Bridge

Réparations du pont de Givors

Instandstellungsarbeiten an der Givors Brücke

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Dr. Eng.
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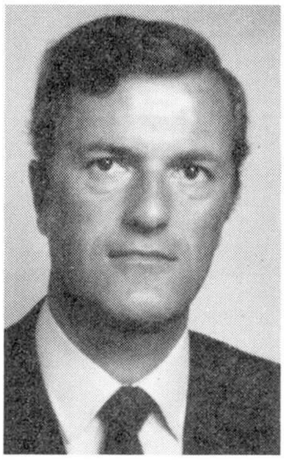

Engineer, C.E.T.E. Lyon, born in 1941, Engineering degree and Doctorate INSA Lyon. Engineer at C.E.T.E. Lyon for 13 years. Took part in the evaluation, design studies and follow-up of repairs on several major bridge structures in the Rhone-Alpes region of France.

Philippe MOREAU
Dep. Head
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Born 1940, in Paris, Master's degree in Engineering with Ecole Polytechnique de Paris in 1960. Associated with a big contractor firm, as Design Engineer, Head of Department, then Director of new techniques, he carried out studies relating to containment vessels in prestressed concrete, offshore platforms, LNG tanks and precast concrete bridges.

SUMMARY
The Givors motorway bridge exhibited certain disorders which were liable, with time, to reduce its bearing capacity and would have required it to be taken out of service in the longer term. Repairs consisted in providing additional prestressing and protecting the deck against the action of aggressive environmental agents.

RESUME
Le pont autoroutier de Givors présentait des désordres pouvant réduire, à terme, sa capacité portante et entrainer à plus long terme sa mise hors service. Les réparations ont consisté en la mise en oeuvre d'une précontrainte additionnelle et à assurer la protection du tablier contre l'action des agents agressifs extérieurs.

ZUSAMMENFASSUNG
Die Givors-Autobahnbrücke wies Zerrüttungserscheinungen auf, die die Reduktion der Tragfähigkeit und auf längere Sicht die Schliessung der Brücke hätten verursachen können. Die Instandstellungsarbeiten umfassten eine zusätzliche Vorspannung des Bauwerks und den Schutz der Fahrbahnplatte gegen agressive Mittel.

1. DESCRIPTION OF STRUCTURE

The Givors bridge, built in 1967-1969 allows the crossing of the Rhone river by the A.47 motorway providing a link between Lyons and Saint-Etienne. The bridge handles considerable traffic (25 000 vehicles/day, 30 % of which consist of lorries).

The structure is 300 m long and 18 m wide. It is made up of a traversely and longitudinally prestressed concrete deck. It has five continuous spans of 30, 110, 20, 110 and 30 m respectively.

1.1 Characteristics of structure

1.1.1 Deck

The deck includes two identical box girders of rectangular section and variable height secured by the upper slab forming a road pavement. The height of the girders varies from 5.5 m on the piers to 2 m at the key of the 110 m spans. The thickness of the webs is constant and equal to 0.3 m.

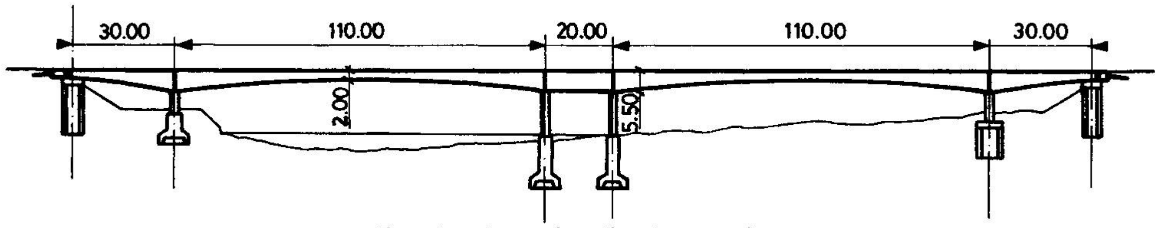

Fig. 1 _ Longitudinal section

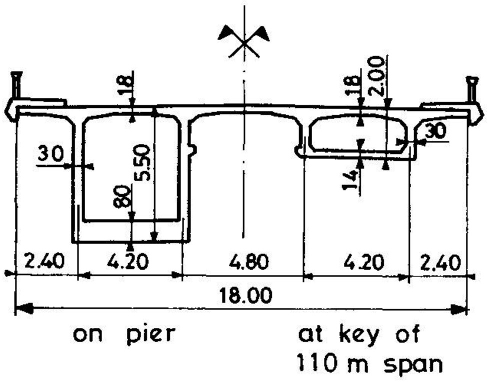

Fig. 2 _ Cross_section

1.1.2 Piers

The central piers and the right-bank pier are founded on steel caissons sunk by compressed air in the gravelly alluvia of the Rhone.

The left-bank pier is founded on a reinforced concrete caisson placed in the open.

The deck rests on the central piers via Freyssinet hinges and on the bank piers via plates in reinforced neoprene.

1.1.3 Abutments

The abutments are sunk into the access embankments and are each founded on open caissons. The deck-abutment connection has the particular feature of opposing the permanent raising of the bank span ends thanks to a tenon and mortise device, each tenon being in the projection of a web. Free expansion and rotations on the abutments are provided by bearing devices in reinforced neoprene. The abutment is transversely prestressed to withstand the forces applied to it, a lean concrete ballasting being used to prevent lifting.

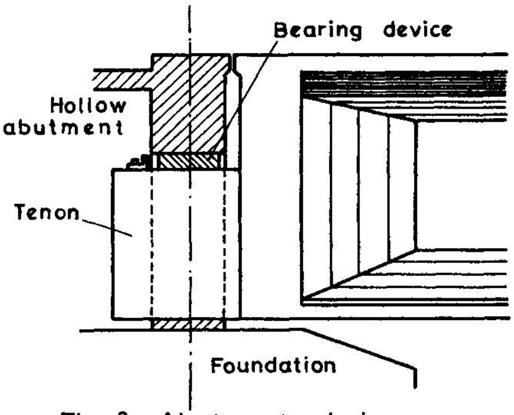

Fig. 3 – Abutment device
(longitudinal section)

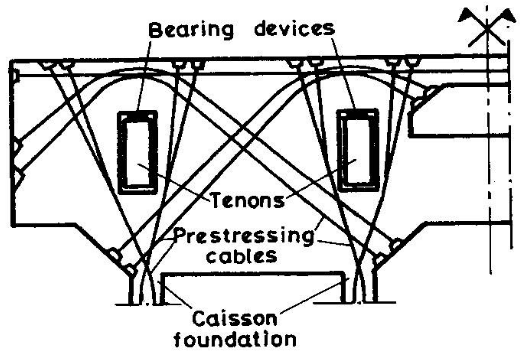

Fig. 4 – Abutment device
(cross-section for a caisson)

1.2 Construction of bridge

After completing the piers, the deck was constructed by contilevering from the piers with voussoirs 3,4 m long concreted in place by means of travelling formwork. The voussoirs on piers are 8 m long. Longitudinal prestressing was applied according to the STUP process using cables consisting of 12 strands of 31 mm diameter (12 T 13) and 8 mm diameter (12 Ø 8). In the transvere direction, prestressing is provided by 12 Ø 8 cables with an average spacing of 1,12 m. The waterproofing of the structure consisted of a "Colpont" type blanket, the road pavement being of the "Gussasphalt" type.

II DESCRIPTION OF DISORDERS

As part of the periodic inspection of bridges, the management department had a detailed inspection carried out on the bridge during the months of May and June 1977. This showed the existence of apparently serious disorders capable of jeopardizing the operating safety of the bridge and justifying in-depth investigations.

The damages affected :
- The road pavement : the pavement exhibited cracking and crazing of a widespread nature and of such a depth locally that the underlying concrete was visible.
- The inside of the caissons : the following were observed :
  . numerous leaks,
  . cracks in the lower slab and in the webs ; the affected zones in the lower slab being located in the central part of the large spans and were broken down into three distinct categories :
    .. bending cracks on the voussoir joints,
    .. cracks at the back of the anchorage bosses,
    .. prestressing diffusion cracks along the bosses.

The disorders in the web were of a more generalized nature, showing :
- voussoir joint openings,
- cracks following the path of the cables,
- many zones of segregated concrete (bad facings, rock pockets),
- distortions and splitting in bearing devices and excessive or abnormal migration on the abutments.

III. ANALYSIS OF CAUSES

A priori, after the detailed inspection, two major causes offered an explanation for the disorders :
. inadequate construction,
. Insufficient prestressing.

In order to better assess the situation and to determine the required repairs, an additional series of investigations was carried out on the bridge itself and on its construction documents.

3.1 Additional investigations

The following investigations were conducted :
- Crack measurements under traffic and under the influence of tempereature and humidity.
- Measurement of bearing pressures on pier.
- Gammagraphy, along with the placement of inspection openings on certain cables.

At the same time, the different construction documents were examined, namely :
- design drawings and calculations,
- the results of inspections carried out during construction (results on concrete, tensioning records, etc...).

3.2 Results of investigations

These additional investigations yielded the following findings :
. Under traffic, crack movements reach 0,3 mm.
. 60 % of the X-rayed cables exhibited grouting defects sufficiently serious so that their protection was jeopardized.
. The damages observed do not come from the materials (steel and concrete) themselves.
. There is every indication that the specified prestressing was applied.
. Design calculations do not show any procedure not in accordance with regulations.

3.3 Diagnostics and assessment

After the examination of several elements, the obvious lack of prestressing was found to come from a series of events summarized briefly below.
At the time of construction, adaptation phenomena were only exceptionally taken into account in the design calculations ; this was however covered by a special note of the contracting firm and was taken into account to the amount of 50 % in the dimensioning of prestressing.
Moreover, jobsite difficulties led to a repair intended to "make up" the longitudinal profile. However, in the design calculations for verifying this new load, no account was taken of the results of the adaptation calculations. Hence, if the validity and the procedures used for the adaptation calculations are accepted, the structure as it was constructed exhibits tensions in the lower slab reaching 3,1 Mpa. Furthermore, since the time of construction, the influence of temperature gradients has been established.
All these accumulated factors bring about tensile stresses of 4,7 Mpa in the lower slab, largely sufficient to explain the cracking especially as the joints between voussoirs are not reinforced.

Let us finally points out that the results of bearing pressure measurements confirm those of the adaptation calculations of the contracting firm.

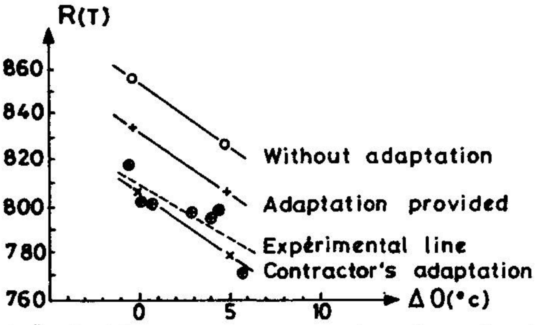

Fig.5 - Variation of bearing pressure on abutment with temperature gradient

Concluding, we can state that in the case of this bridge there was insufficient prestressing.
The problems are aggravated by the insufficient grouting accentuated by waterproofing defects. Proper waterproofing must consequently be provided on this structure at the level of the road pavement as well as to prevent the penetration of moisture at the level of the cables.

IV. DEFINITION OF REPAIRS

4.1 Waterproofing

The aim was to protect prestressing cables against corrosion. Two solutions were examined :
- Protecting the structure against any infiltration of water capable of reaching the cables.
- Protecting the cables themselves, i.e. regrouting 60 % of the ducts (10 litres of slurry to be used).

The second solution was found to be risky because :
- Regrouting had to be carried out under vacuum, hence practically requiring the waterproofing of all the cracked zones.
- The location of the ducts for feed borings was regarded as difficult if not dangerous, in view of the number involved.
- Gammagraphy showed that grouting defects were in most cases "dotted" along the path of the cables.

The first solution was consequently adopted with the following conditions :
- Removal of wearing course and of waterproofing layer.
- Demolition and reconstruction of cracked or ruined repair zones (and reconditioning of sidewalks).
- Treatment of joints between repaired blocks.
- Placement of new wearing course waterproofing system.
- Reinforced bridging outside of fine cracks (thickness $\leq$ 0,5 mm).
- Grouting of largest cracks (thickness $>$ 0,5 mm).
- Smoothening and blinding of degraded concrete zones.
- Application of a waterproofing layer under the cantilevers and on the external flanks of the caissons.

4.2 Prestressing

The additional prestressing was dimensioned according to the following criteria :
- Adaptation according to the initial calculations of the contracting firm.
- Temperature gradient equal to 5°C cumulated at the load locations.
- Dead weight of structure + repairs + new superstructures.
- Loads in accordance with 1971 regulation.

Under these conditions, the tensile stresses to be compensated were T,8 Mpa.

For the entire structure, the additional cabling includes :

- eight 12 T 15 cables
- four 6 T 15 cables
- four 12 T 13 calbes

running from abutment to abutment in each span of 110 m

These cables are deviated in order to improve the efficiency. The anchorage of the 12 T 13 cables is accomplished by means of bosses bearing on the spacers on the piers.

The anchorage of the 6 T 15 and 12 T 15 cables is accomplished, for each caisson, by means of a still (in prestressed concrete) bearing on the end tenons via Freyssinet type hinges.

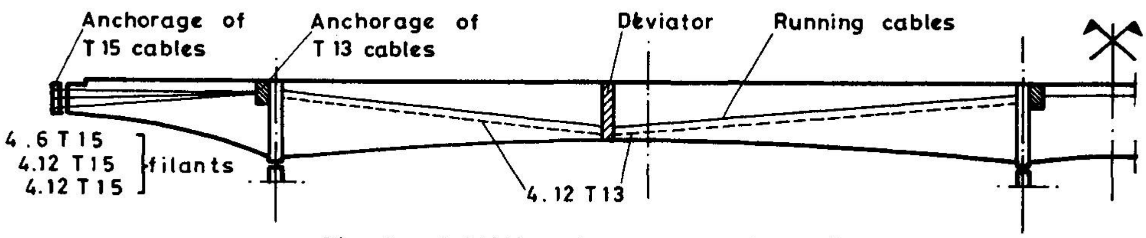

Fig. 6 – Additional prestressing diagram

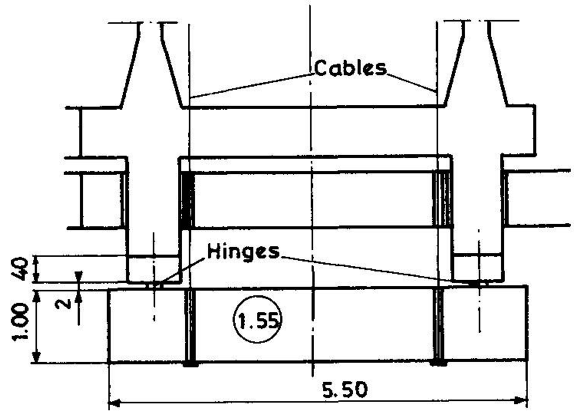

Fig.7 - Anchoring device on abutment

V . WORK PHASES

The different operations were carried out between May 1979 and September 1980. They required constant follow-up and inspection in view of the difficulties encountered :

- lack of room for the changing of bearing devices,
- necessity of surfacing the bearing zones,
- complexity of setting anchorage structures in zones of difficult access,
- coordination of crack grouting with prestressing,
- treatment of all cracks and general waterproofing.

5.1 Conclusions on work carried out

- Additional prestressing : Prestressing operations were followed by extensometry ; the analysis of the tests shows that prestressing transited into the sections according to the calculation model.
- Waterproofing : The adopted arrangements appear to have achieved what was expected ; in fact, the structure is now perfectly dry.

On the whole, it can consequently be considered that the repairs have allowed the bridge to go back into normal service. However, it is a bridge which must be kept under constant supervision owing to the lack of knowledge regarding the longterm behaviour of certain materials and notably of resins.

Strengthening of Bridges with Epoxy Bonded Steel Plate

Renforcement des ponts par des tôles d'acier collées

Verstärkung von Brücken mit angeklebten Stahllaschen

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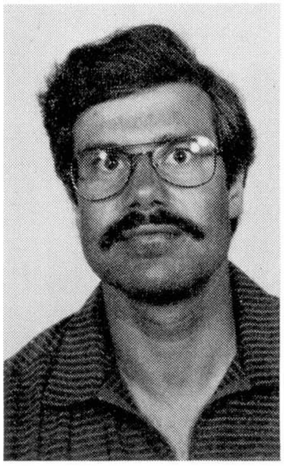

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SUMMARY
Numerous prestressed concrete bridges in the FRG, built by spanwise construction, show cracks in the working joints. For the remedy of these cracks the strengthening with epoxy bonded steel plate is a possible solution to reduce fatigue stresses of the prestressing steel and for crack control. After discussion of the efficiency the actual application of the method for a bridge is presented.

RESUME
De nombreux ponts en Rép. Féd. d'Allemagne, dont les travées ont été construites séparément, montrent des fissures dans les joints de construction. Des tôles d'acier collées représentent une bonne solution pour renforcer la construction. L'utilisation de ces tôles permet de réduire les contraintes de fatigue de l'acier de précontrainte et de contrôler les fissures. Après avoir discuté de l'efficacité du renforcement, un exemple d'application à un pont est décrit.

ZUSAMMENFASSUNG
Zahlreiche ältere Spannbetonbrücken in der BRD, die feldweise hergestellt worden waren, zeigen Risse in den Koppelfugen. Zur Sanierung dieser Risse eignen sich angeklebte Stahllaschen, mit denen die Ermüdungsbeanspruchung des Spannstahls und die Rissbreiten reduziert werden können. Nach Erläuterung der Wirksamkeit der Verstärkung wird deren Anwendung bei einem Brückenbauwerk beschrieben.

1. INTRODUCTION

During the past decades numerous prestressed concrete roadway bridges were built in the FRG employing the method of in-situ spanwise construction. These multi-span continuous bridges are mostly hollow-box beams. The working joints are in the points of contraflexure where usually all of the tendons are coupled. Many of the bridges exhibit more or less severe cracks at the joints. Usually, the bottom slab of the box girder is traversed by a crack of large width. This crack rises into the webs with diminishing width, thereby crossing the lower tendons and couplings. The main cause of these cracks is a temperature restraint, unaccounted for in previous designs [1]. In combination with other actions tensile stresses at the bottom arise and overcome the very low concrete tensile strength at the joint. As the reinforcement ratio of the bottom slab was often very low, yielding of the steel occurs and wide residual crack widths are formed. Due to transformation of the cross-section into the cracked state the durability of the reinforcement and of the tendons in connection with increased fatigue stresses is endangered. Thus, the necessity for repair arises, for which the strengthening with bonded steel plates, positioned in the interior of the box section, is a promising method and for which positive experiences were already gathered in several countries.

2. EFFICIENCY OF BONDED STEEL PLATE REINFORCEMENT

The repair of cracks in joints by epoxy resin injection alone is not a sufficient countermeasure because the highly probable repetition of thermal restraint may cause either new cracks or the opening of injected cracks. Therefore, besides crack injection also additional reinforcement - provided for by the bonded steel plates - is necessary to reduce the amplitude of fatigue stresses and the crack widths as well.

The efficiency of the bonded steel plate reinforcement with respect to fatigue stresses in the prestressing steel is shown by Fig. 1. The base moment M_o is superimposed by the moment M_{te} of the first high thermal restraint. Thereby, the cracking moment M_r is transgressed. The moments due to traffic loads cause a fatigue stress range $\Delta\sigma_p$ which exceeds the stress range in the uncracked section. After strengthening of the section the slope of the steel stress-moment-curve will be diminished. Thus, by choice of a suitable cross-section of steel plate the stress range may be reduced to the admissible value as predetermined by acceptance tests.

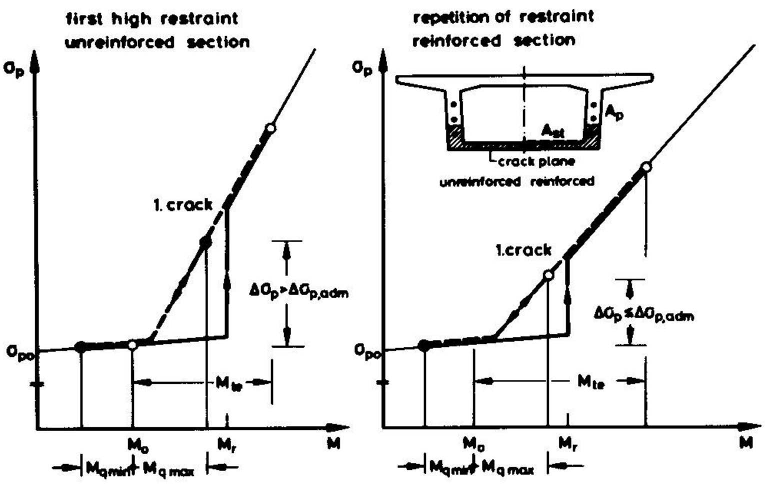


Fig. 1 Efficiency of bonded steel plate strengthening with respect to fatigue stresses in prestressing steel of joint

The efficiency of the additional reinforcement with respect to the steel stress of the regular reinforcement and to the crack width is depicted in Fig. 2. Due to the steel plate the steel stresses are relieved once a new crack is formed by repetition of restraint. After strengthening, both reinforcement and steel plate operate in the elastic range, thus, contracting the widths of possible new cracks without virtual residue.

3. RESEARCH WORK

Prior to application of this repair method extensive tests had to be carried out to assess the influence of the numerous parameters of strength and deformability of the composite steel plate - epoxy resin glue - reinforced concrete. The tests were financed by the Federal Ministry of Traffic [2],[3],[4].

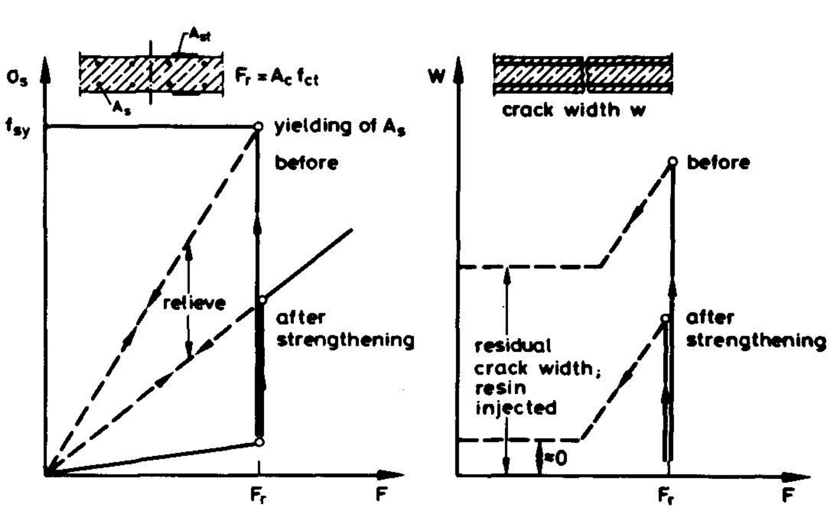


Fig. 2 Influence of strengthening on the steel stress of the reinforcement and on the crack width

The research consisted two parts. In the first part basic experimental work was conducted to study the following parameters of bond strength:

a) concrete
strength, carbonation moisture and planeness of surface

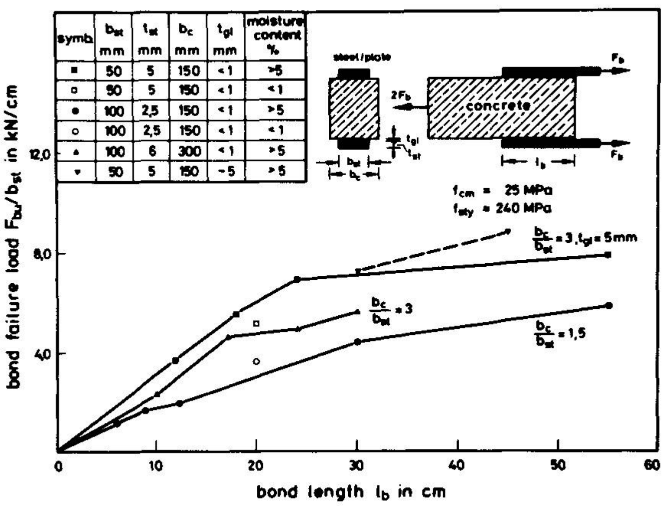
b) steel plate
strength, planeness and surface condition

c) glue
type, thickness of glued joint, ambient conditions during work and after

d) type of loading
short-time, long-time or dynamic loading

e) climate
humidity and temperature during service

f) geometry
relation between dimensions of concrete and plate



symb.	b_{st} mm	t_{st} mm	b_c mm	t_{gl} mm	moisture content %
■	50	5	150	<1	>5
□	50	5	150	<1	<1
●	100	2,5	150	<1	>5
○	100	2,5	150	<1	<1
▲	100	6	300	<1	>5
▼	50	5	150	~5	>5

Fig. 3 Bond failure load as a function of bond length and of b_c/b_{st}

Then, applied research followed in which the behaviour of reinforced concrete members of realistic size with and without strengthening was studied.

From the first part of research, Fig. 3 shows the influence of the bond length on the bond failure load per unit width of the steel plate (short time tests). The most important influence is exercised by the bond length. Up to a bond length of 30 cm the bond failure load shows an approximately linear increase. Beyond this length the increase becomes small with the tendency to approach a limit value of the bond strength. A thickness of steel plate of 5 to 10 mm and a geometry ratio $b_c/b_{st} \simeq 3$ seem to be optimal. Several epoxy glues, especially formulated for this application, were tested without showing significant differences in strength. With a suitable glue and adequate preparation and fabrication of the bonding joint, bond failure always occurs in the concrete several millimeter below the joint. Moisture and carbonation of the concrete surface region are of little influence. Up to a thickness of the glue of 5 mm the thickness exercises no significant influence. The dynamic strength proved to be satisfactory.

In the second part of the research work the composite action of reinforced concrete members strengthened with bonded steel plates was studied. At the onset of cracking the bottom slab of a box girder will be approximately subjected to axial tension. Thus, the specimens with a cross-section of 15 cm/50 cm and the length of 3,40 m had to simulate this situation. They were reinforced with 4 deformed bars of 10 mm diameter (grade 420/500). The cross-section of the steel plate was chosen to be twice and four times of the reinforcement's cross-section. The tests showed that the combined yield force of the two steel materials could be activated prior to bond failure. Up to failure the deformation and cracking of the strengthened specimens corresponded to r/c-specimens reinforced with same total steel area but as deformed bars. Fig. 4 shows typical results [2]. On basis of the test results rules for design and for the practical execution of the strengthening with bonded steel plate were developed. An alternating force during the hardening of the glue does not harm bond provided certain limits are not exceeded. This finding is of great practical consequence for the repair of bridges whose traffic flow must be kept up during repair.

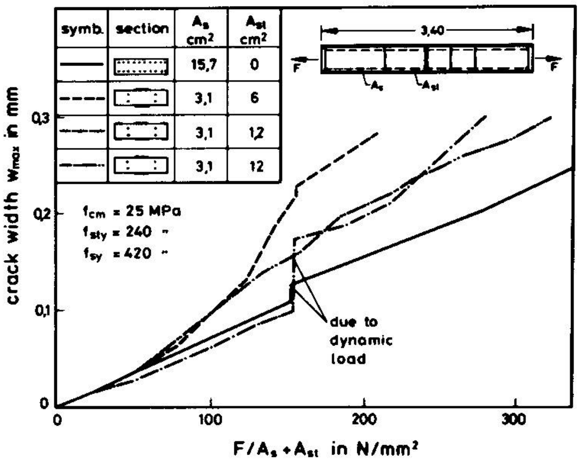


Fig. 4 Width of largest crack, depending on load and strengthening ratio

4. PRACTICAL APPLICATION

In order to gain practical experience with this method of strengthening its application was decided on for the Sterbecke Bridge near Hagen in 1980. This motorway-

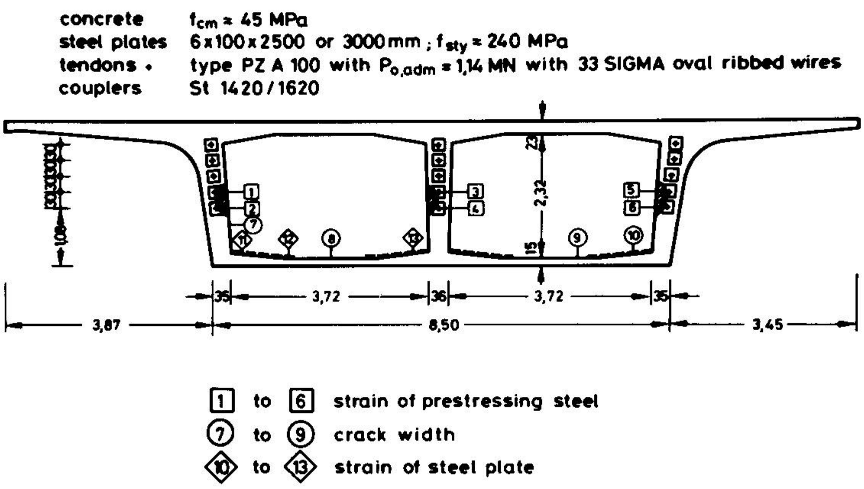


Fig. 5 Cross-section of bridge and other data

bridge consists of two separate superstructures with box sections as shown by Fig. 5. It is continuous over 7 spans of 40 m each. It was built spanwise with all of the 15 tendons being coupled at the joint near the points of contraflexure. In all working joints the bottom slab was cracked; the crack plane extended into the webs, passing the lower couplings. The necessary cross-section of the bonded steel plate reinforcement was chosen to reduce the calculated stress-range of the prestressing steel in the coupling to the admissible value of the prestressing system. The arrangement of the plates is depicted in Fig. 5. The length of the plates was chosen to cover the region of possible concrete tensile stresses at the bottom due to repeated restraint and other actions.

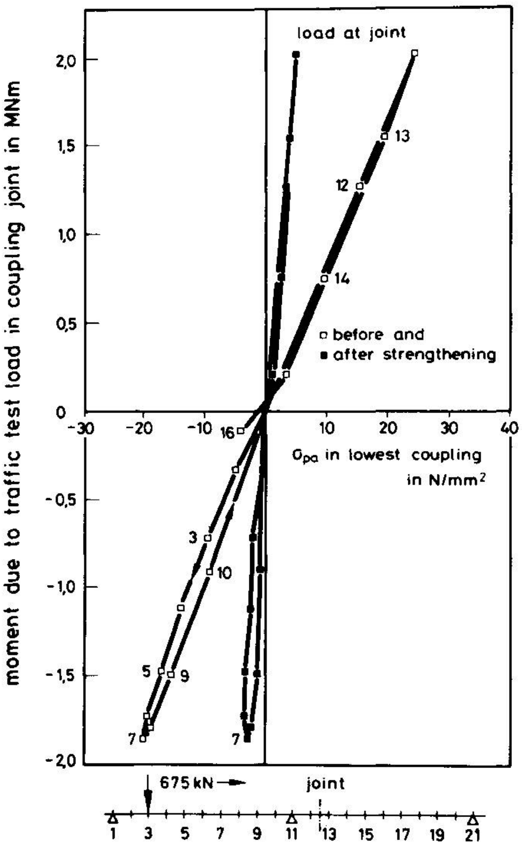


Fig. 6 Stress amplitude of prestressing steel of the lowest coupling, depending on test load moment

In order to assess the efficiency of the strengthening extensive on-site tests were performed prior and after strengthening. The test loading was executed with three 22,5 tons lorries which travelled along the bridge in different patterns. Fig. 6 shows the stress amplitude of the prestressing steel at one of the lowest couplings as a function of the moment at the joint due to the lorries travelling side by side. The stress amplitude is dramatically reduced proving the desired efficiency of the chosen strengthening method. The design loads will cause greater moment amplitudes, however, the admissible stress range of $\Delta\sigma_p$ = 55 N/mm² will not be transgressed. The complete results will be published soon.

The work on site consisted of several steps. Firstly, the cracks were injected with an approved epoxy resin. Then, the top surface of the bottom slab had to be prepared by vacuum grit blasting and by a shaper to the desired planeness. The grit blasted steel plates were glued to the concrete piecewise and pressed down by plane and stiff wooden distributing beams for the first 24 hours. Local deviations from the desired planeness of the concrete surface can be tolerated: they can be evened-out with thicker layers of glue ($\leq$ 10 to 15 mm). It is very important that the steel plates remain straight; tests have shown that transverse tension normal to the glued joint reduces bond strength. The glue must comply with the climatic conditions on site. During hardening of the glue the regular traffic prevailed. The gluing of 26 plates per box was performed within one day.

5. ACKNOWLEDEMENTS

The authors are indebted to Mr. Engelke of the Federal Ministry of Traffic and Mr. Voss, chief bridge engineer of the Landschaftsverband Westfalen-Lippe for their decision to test this method of strengthening.

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Post-Tension Strengthening of Composite Bridges

Renforcement de ponts mixtes par post-contrainte

Verstärkung von Verbundbrücken durch nachträgliche Vorspannung

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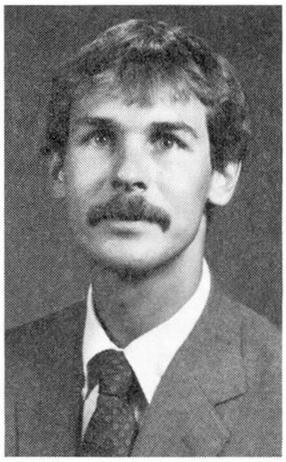

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SUMMARY
Because of changes in design specifications and increases in legal loads, a number of simple-span steel-beam, composite-concrete deck bridges in Iowa need the live load carrying capacity of the exterior stringers increased to meet current legal load limits. The feasibility of strengthening such bridges by post-tensioning was determined from laboratory tests. Reported herein are the results of strengthening by post-tensioning of two existing bridges.

RESUME
Faisant suite au changement des normes de projet et de l'augmentation des charges admises légalement, un certain nombre de ponts mixtes à une travée, en Iowa, USA, doivent être renforcés pour satisfaire aux règlements en vigueur. Des essais en laboratoire ont montré la possibilité de renforcer de tels ponts, par post-contrainte. L'article présente deux cas d'application de la post-contrainte à de tels ponts.

ZUSAMMENFASSUNG
Aufgrund von Änderungen in den Konstruktionsnormen und der Erhöhung der zulässigen Lasten müssen die Randträger einer Anzahl als einfache Balken ausgebildete Verbundbrücken im Staate Iowa, USA, verstärkt werden. Laborversuche haben die Möglichkeit aufgezeigt, solche Brücken durch nachträgliche Vorspannung zu verstärken. Der Beitrag bespricht die Verstärkung zweier Brücken die mit vier Längsträgern ausgebildet sind.

1. INTRODUCTION

Many simple-span, steel-beam, composite-concrete slab bridges built between 1930 and 1960 are not in complete compliance with today's bridge standards. In Iowa alone over 70 bridges of this type require "strengthening" to meet current standards. More specifically, the live load carrying capacity of their steel stringers must be increased. The simple-span steel-beam composite-concrete slab bridges (henceforth simply referred to as bridges) requiring such strengthening fall into two categories:

- those that should be posted because they do not satisfy allowable live load stress limits.
- those that presently are structurally and geometrically adequate but require additional load capacity to acccommodate resurfacing for extended life.

At present, no acceptable procedures for such strengthening have been developed. Thus, the purpose of this research program was to determine a technique for increasing the capacity of the steel stringers of these bridges, thereby increasing both their live load and dead load carrying capacity.

After an extensive literature review of the various methods for strengthening bridges, post-tensioning was viewed as the most economical and promising. Phase I of this study was, therefore, directed to determining the desirability of post-tensioning the stringers in the bridges. The objective of Phase I was fulfilled by testing a model bridge and applying known analytical procedures to determine the bridge's behavior under post-tensioning. Phase II of the study, which is reported herein, involved the strengthening of two existing bridges utilizing the post-tensioning schemes designed in Phase I.

The concept of prestressing steel structures is well over a century old. However, prestressed composite structures are a much more recent development. In the 1960s several USA researchers tested prestressed composite structures and developed methods for their analysis [1,2]. A few U.S. bridges have been built utilizing prestressed composite structures since 1960. Reference 3 is recommended for a more complete review of post-tensioning steel structures.

2. GENERAL RESEARCH PROGRAM

On the basis of the literature review, field inspection of several bridges, and the results of Phase I, a second testing program was planned which involved the strengthening and testing of two existing bridges.

One of the bridges selected for strengthening, henceforth referred to as Bridge #1, was a four stringer 15.24m × 9.14m I-beam right angle bridge which is similar to the model bridge of Phase I but twice as large. The other bridge, henceforth referred to as Bridge #2, was a four stringer 21.34m × 9.14m I-beam 45° skewed bridge.

As noted later, only the exterior stringers of each bridge were post-tensioned as they were the only ones significantly overstressed. In addition to the post-tensioning forces which were applied to the bridges, each bridge was subjected to an overloaded truck weighing slightly over 267 kN. The response of the bridges to the truck loading was determined before and after post-tensioning so that the effectiveness of the post-tensioning could be ascertained.
Although a variety of bridge deck analysis methods are available, such as grillage analysis, finite difference analysis, and finite element analysis, orthotropic plate theory was utilized for correlation of experimental results [4]. Although good agreement was obtained between experimental and theorectical re-

sults, post-tensioning forces can only be approximated with orthotropic plate theory. A finite element program is presently being developed for a more exact analysis; however, in the authors' opinion, the analysis developed to date may be used in design with little difficulty.

3. DESCRIPTION OF BRIDGES STRENTHENED

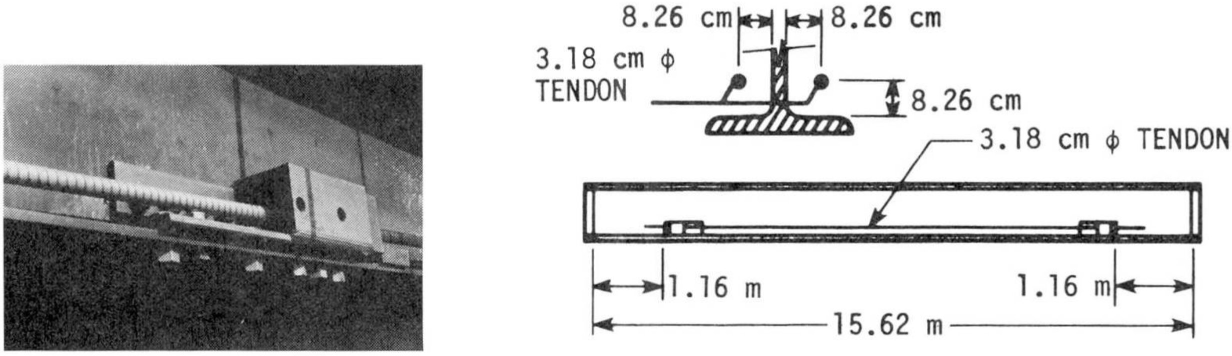
Bridge #1, built in 1948, is located 3.54 km north of Terril, Iowa, U.S.A. For composite action, all stringers had angle-plus-bar type shear connectors. Analysis indicated that for the bridge to carry additional live load, the exterior stringers needed additional shear connectors. Assuming the type of steel may be unknown in bridges requiring strengthening, it was decided to add shear connectors by bolting rather than welding. High strength bolts were tested as shear connectors in the laboratory and found to be more than adequate. Cores (10.16 cm to diameter) were drilled in the bridge deck above the exterior stringers and a total of 52 (26 per stringer) high strength bolts were double-nutted to the stringers; later the core holes were grouted. Member sizes, span lengths, etc. for Bridge #1 as well as Bridge #2 are presented in Table 1.

	Bridge #1		Bridge #2	
Span (Center line to Center line of Bearing)	15.62 m		21.72 m	
Deck Slab Width	9.72 m		9.72 m	
Deck Slab Thickness	23.50 cm		20.96 cm	
	Exterior Stringer	Interior Stringer	Exterior Stringer	Interior Stringer
Steel Beam	W27 × 94	W30 × 116	W33 × 130	W36 × 194
Beam Depth	68.38 cm	76.23 cm	84.05 cm	92.68 cm
Cover Plate	22.86 cm × 1.11 cm	22.86 cm × 3.18 cm	25.40 cm × 2.22 cm	27.94 cm × 3.49 cm

Table 1 Properties of Strengthened Bridges

Details of the post-tensioning system employed are shown in Figure 1. Due to uncertainty about the type of steel in some of the bridges to be strengthened, the connections were bolted. Figure 1a illustrates one of the 8 brackets, fabricated from 1.91 cm thick structural angle, bolted in position. The position of the 3.18 cm ϕ post-tensioning tendons is shown in Figure 1b.

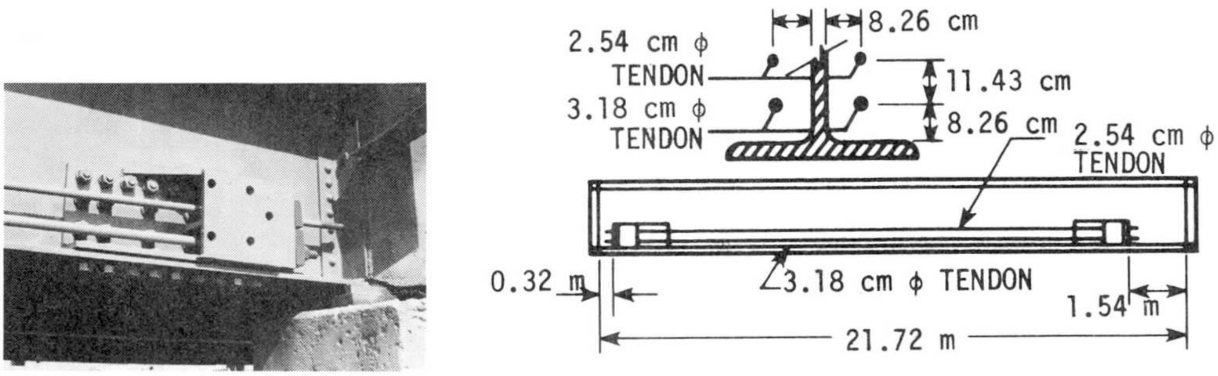
Bridge #2, built in 1947, is located on State Highway 144 approximately 24.14 km north of Grand Junction, Iowa, U.S.A. This bridge, like Bridge #1, has angle-plus-bar shear connectors and needed additional shear connectors on both the interior and exterior stringers to be able to carry additional live load. As on Bridge #1, cores were cut in the deck and 28 high strength bolts were added to each exterior stringer and 26 to each interior stringer for a total of 108 on the bridge.



(a) POST-TENSIONING BRACKET AND TENDON IN POSITION

(b) TENDON LOCATION

Fig. 1. Post-tensioning details - Bridge 1.



(a) POST-TENSIONING BRACKET AND TENDONS IN POSITION

(b) TENDON LOCATION

Fig. 2. Post-tensioning details - Bridge 2.

Details of the post-tensioning system employed are shown in Figure 2. Figure 2a illustrates one of the brackets bolted into place with high strength bolts; each bracket was fabricated from 1.91 cm thick plate. The position of the 3.18 cm ϕ and 2.54 cm ϕ post-tensioning tendons is shown in Figure 2b; note that the bracket is located at a great distance from the center line of bearing on the jacking end of the stringer.

4. FIELD TESTING PROGRAM

The testing program on each bridge included three parts: Part 1 was the response of the bridge to an overloaded truck (weight = 267 kN). Part 2 was the response of the bridge to post-tensioning, and Part 3 was the response of the bridge to the same truck after post-tensioning.

Parts 1 and 3 were essentially identical for both bridges. The truck was placed at a number of positions on the bridge, and the response of the bridge was recorded. Part 2 was different for each bridge and is noted below.

4.1 Bridge #1

For data collection, four electrical-resistance strain gages (two on the top flange and two on the bottom flange) were placed at the center line of each stringer. Additional instrumentation was used to measure deflections at the midspan of all stringers and the quarter points of one interior and one exterior stringer, and to measure the force in each tendon.

Hollow core hydraulic cylinders were used in post-tensioning the bridges. Since only two cylinders were available, it was necessary to post-tension the bridge in steps, post-tensioning one exterior stringer to approximately 1/3 of the desired force and then the other until slightly less than the desired force of 818.5 kN per stringer was obtained.

4.2 Bridge #2

As a result of the strengthening of Bridge #1, the instrumentation of Bridge #2 was increased. Electrical-resistance strain gages (two on the top flange and two on the bottom flange) were placed at the center line of each stringer, the quarter point of one interior and one exterior stringer, and close to one support of the same interior and exterior stringer. Strain gages were placed close to the ends of the stringers to measure the end restraint which was observed in the testing of Bridge #1. A step by step post-tension sequence similar to that utilized on Bridge #1 was employed except that twice as many steps were required because the exterior stringers had four tendons to post-tension. On the average, the desired force of 1361.2 kN per stringer was achieved.

5. TEST RESULTS

Space limitations allow only a portion of the results of the study to be presented in this paper. Details of the results of Phase I are presented in Reference [3]; those of Phase II will be presented in the final report which is currently being prepared.

5.1. Bridge #1

The desired post-tensioning force for each bridge was based upon the results of Phase I of the study. Thus, subjecting each exterior stringer to a force of 818.5 kN was supposed to cause a strain reduction in the lower flange of 218 $\mu\varepsilon$. This was based upon the assumption that the bridge was simply supported. However when the 818.5 kN force was applied in the field, a strain reduction of only 147 $\mu\varepsilon$ was obtained due to restraint of the ends of the stringers. Figure 3 compares the midspan strains measured with theoretical strains assuming the stringer to be completely restrained against rotation and completely free to rotate. Note that the measured strains are essentially midway between the fixed and the free condition. Thus, although the desired strain reduction was not obtained, it should be noted that the end restraint also reduces the effect live load has on the bridge.

5.2 Bridge #2

As in Bridge #1, the force applied to each exterior stringer (1361.2 kN) was determined assuming the stringers to be simply supported. The 1361.2 kN force was to cause a strain reduction in the lower flange of 212 $\mu\varepsilon$. When the desired force was applied in the field, a reduction of only 102 $\mu\varepsilon$ was obtained because of end restraint. Figure 4 compares the midspan strains measured with the theoretical strains assuming the stringers to be first completely restrained against rotation and second completely free to rotate. As may be seen, the strains in the exterior stringers approach the fixed end condition.

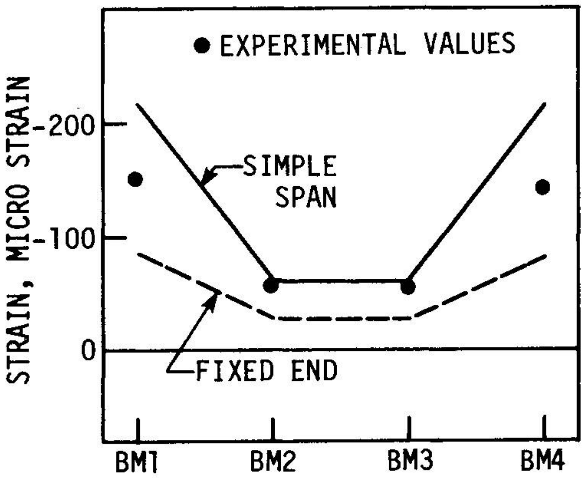


Fig. 3. Midspan, bottom flange strains resulting from post-tentioning: Bridge #1.

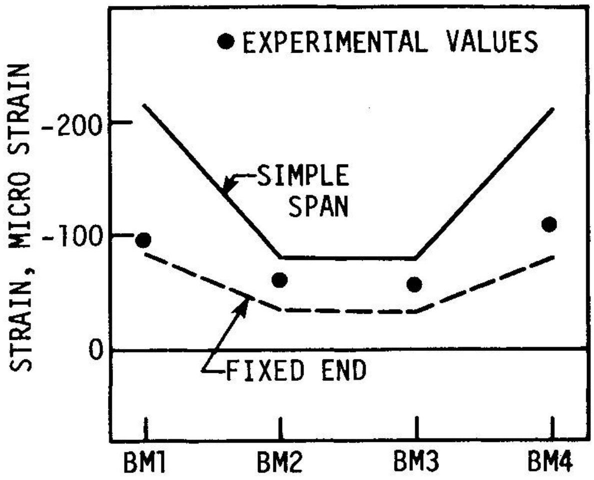


Fig. 4. Midspan, bottom flange strains resulting from post-tentioning: Bridge #2.

6. SUMMARY AND CONCLUSIONS

The results of the research outlined in this paper indicate that post-tensioning can be used for flexural strengthening of simple-span steel-stringer concrete-deck highway bridges. Each bridge to be strengthened should be reviewed to determine if additional shear connectors are required and what type and degree of end restraint exists as these variables are extremely significant in the strengthening procedure. Orthotropic plate theory, which has been established as a means to predict vertical load distribution in bridge decks, may be used to predict approximate distribution of post-tensioning axial forces.

7. ACKNOWLEDGMENTS

This program is being conducted by the Engineering Research Institute of Iowa State University and is funded by the Iowa Highway Research Board and the Highway Division, Iowa Department of Transportation. The authors thank the engineers of the Iowa DOT--especially Charles A. Pestotnik, Bridge Engineer, and John Harkin, Bridge Rating Engineer, for their support, cooperation, and counsel. Special thanks are accorded Assistant Professor Kenneth F. Dunker, Iowa State University, for his assistance during various stages of the program. The opinions, findings, and conclusions expressed herein are those of the writers and are not necessarily those of the Iowa Department of Transportation or the Iowa Highway Research Board.

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Erneuerung der Rheinbrücke Reichenau

Retrofitting of the Rhine bridge at Reichenau

Rénovation du pont sur le Rhin à Reichenau

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ZUSAMMENFASSUNG
Die Erneuerungsarbeiten an der Reichenauer schweisseisernen Brücke umfassen den Einbau einer Leichtbetonfahrbahn an Stelle der Schotterfüllung auf Zores-Eisen sowie eine K-artige Unterteilung der Querscheiben. Unter Berücksichtigung des überkritischen Knickverhaltens der Ausfachung bei den äusseren Fachwerkträgern ergibt sich dann eine genügende Sicherheit für die anfallenden Lasten dieser Strassenbrücke mit nur örtlicher Bedeutung.

SUMMARY
The retrofitting of the wrought iron bridge at Reichenau includes the replacing of the old deck, consisting of macadam on Ω-shaped profiles, by a slab of light-weight concrete and the local strenghtening of the cross bracings. Taking into consideration the postcritical buckling behaviour of the lattice for the external trusses, the structure will be safe under the loads acting on this highway bridge of purely local importance.

RESUME
Les travaux de rénovation du pont en fer puddlé de Reichenau comprennent le remplacement du tablier existant, en macadam sur fers Zorès, par une dalle en béton léger ainsi que des renforcements locaux des entretoisements. En tenant compte du comportement au flambement postcritique de la triangulation des poutres extérieures, on obtient une sécurité suffisante pour les charges sollicitant ce pont-route de vocation purement locale.

1. ZUR GESCHICHTE DER REICHENAUER BRÜCKE

Die Strassenbrücke über die vereinigten Rheine zu Reichenau befindet sich unmittelbar nach dem Zusammenfluss des Hinterrheines mit dem Vorderrhein, so dass sich für die erforderliche Ueberbrückung ohne Zwischenpfeiler eine Spannweite von rund 70 m ergibt. Die Flussüberquerung liegt im Zuge der "Italienischen Strasse", welche das Bündner Land über den Bernhardiner- oder den Splügenpass mit der Lombardei verbindet.

Im gleichen Jahr 1757, in dem Hans Ulrich Grubenmann seine berühmte Rheinbrücke Schaffhausen erstellte, baute sein Bruder Johannes die erste Brücke in Reichenau, welche mit ihren 66 m Oeffnung eine Rekordspannweite darstellte [1], [2]. Das Tragsystem dieser zwei Holzbrücken war ähnlich, handelte es sich doch um ein mehrfaches Hängewerk mit Bogenwirkung. Diese bemerkenswerten Bauwerke sind während der Franzosenkriege 1799 verbrannt worden.

Die neue an gleicher Stelle über den Rhein führende Brücke wurde durch das Hochwasser von 1817 zerstört, während die in 1819 von Stiefenhofer erbaute in der Nacht vom 31. Juli auf den 1. August 1880 ein Raub der Flammen geworden ist [3]. Im nachfolgenden Jahr wurde dann die eiserne Fachwerkbrücke ausgeführt, deren nach hundert Betriebsjahren 1980 fällige Erneuerung behandelt werden soll.

2. BESCHREIBUNG DER REICHENAUER EISENBRÜCKE

Vier 69,6 m weit gespannte Fachwerkträger bilden die Hauptelemente der Brücke. Die zwei 7 m hohen Aussenträger dienen gleichzeitig als Seitenabschluss des Verkehrraumes, während die um 1,75 m weniger hohen Innenträger die Fahrbahnabdeckung unmittelbar tragen (Fig. 1). Fachwerkartige Querscheiben, in 3,48 m Abständen angeordnet, verbinden die vier Hauptträger zu einem Trägerrost; sie dienen zudem der Lagerung eines axialen Längsträgers und von zwei Saumträgern, welche neben den Innenträgern die Stützung für die 5,4 m breite Fahrbahn (Strasse 3 m, Gehwege je 1,2 m) gewährleisten. Die Schotterabdeckung, später mit einem Bitumenbelag versehen, ruhte auf Zores-Eisen, die wegen der fehlenden Abdichtung mit der Zeit stark verrosteten und in den dreissiger Jahren ausgewechselt werden mussten.

Fig. 1 Seitenansicht der Eisenbrücke Reichenau

3. GRUNDLAGEN FÜR DIE ERNEUERUNGSMASSNAHMEN

3.1 Werkstoffeigenschaften

Proben, die an wenig beanspruchten Stellen entnommen wurden, haben ergeben, dass das Bauwerk aus *Schweisseisen* besteht. Bei einer Probe betrug die Streckgrenze 270 N/mm^2, bei der anderen 240 N/mm^2; der Berechnung wurde ein Wert von 215 N/mm^2 zugrunde gelegt. Beim Nietwerkstoff liegt dagegen unberuhigt vergossener Flussstahl vor (Festigkeitswerte vergleichbar mit denjenigen eines hochfesten Stahles).

3.2 Belastungsannahmen

Die Verkehrslasten entsprechen grundsätzlich den Angaben der Norm SIA 160 (1970) für Brücken mit herabgesetzter Belastung (2,5 kN/m^2 + Achslast 2x60 kN), weil das Tragwerk heute nur noch dem lokalen Verkehr dient. In Abweichung zur Norm wurde keine gleichzeitig wirkende Gehwegbelastung berücksichtigt, wie dies wegen der abgelegenen Lage des Bauwerkes zulässig erscheint.

3.3 Sicherheitsfaktor

Der Tragfähigkeitsnachweis wurde weitgehend nach der Norm SIA 161/1979 (Stahlbauten) durchgeführt, wobei wegen der Verwendung von Schweisseisen Anpassungen vorgenommen werden mussten. Insbesondere hat man der grösseren Streuungen wegen den Widerstandsfaktor zu 1,3 (statt 1,15) festgelegt, so dass der globale Sicherheitsfaktor 1,8 (statt 1,6) beträgt.

4. ERNEUERUNG DER FAHRBAHNABDECKUNG

Als Ersatz für die Schotterfüllung auf Belageisen ist eine 160 mm starke Leichtbetonplatte (1,8 t/m^3) eingebaut worden. Diese Massnahme führte zu einer Verminderung der ständigen Last um 8 % auf 20 kN/m sowie zu einer kleineren Streuung gegenüber dem alten Zustand mit unbekannter Schotterdicke.

Um die Plattenstärke möglichst klein zu halten, konnten die Belageisen nicht als verlorene Schalung benützt werden. Vor dem Umbau haben aber diese Zores-Eisen die seitliche Versteifung fast allein übernommen: das Bauwerk besass nämlich keinen wirksamen Windverband, weil die schlaffen Diagonalen eine geringe Querschnittsfläche aufweisen und zudem die Gurte der Aussenträger mehr als 1 m oberhalb der WV-Ebene liegen. Während der Fahrbahnerneuerung wurde deshalb das Tragwerk mittels an den Ufern verankerter Seile zweifach seitlich abgespannt.

5. VERSTÄRKUNG DER HAUPTTRÄGER (AUSSENTRÄGER)

5.1 Lastverteilung auf die vier Hauptträger

Die Querscheiben dürfen als praktisch starr angenommen werden. Die höheren und daher bei gleichem Biege- und Schubwiderstand steiferen Aussenträger nehmen rund je 30 % der symmetrisch zur Brückenachse wirkenden Lasten auf, die Innenträger je 20 %. Für die einseitig angeordnete Achslast beträgt das Verhältnis Aussenträger/Innenträger ebenfalls rund 1,5. Beim damaligen Entwurf hat man offensichtlich eine Verteilung zu je 25 % vorausgesetzt, so dass die Aussenträger tatsächlich ungünstiger beansprucht sind und einer sorgfältigen Untersuchung bedürfen.

5.2 Seitliches Knicken der Obergurte der Aussenträger

Die Druckgurte der Aussenträger liegen mehr als 1 m oberhalb der Fahrbahnscheibe und sind alle 3,48 m elastisch durch Pfosten T-145/100/10 gestützt. Der Knickwiderstand wurde einerseits mit einem Eigenwertprogramm bestimmt; dabei ist berücksichtigt, dass die Gurte im unelastischen Bereich knicken und somit nach [4] den reduzierten Modul $T = E\cdot\bar{\lambda}^2\cdot\sigma_K/\sigma_r$ aufweisen, die stabilisierenden Pfosten dagegen elastisch bleiben und daher eine relativ höhere Steifigkeit besitzen. Die Wirkung der gemessenen Vorverformungen wurde zudem mit einer Berechnung 2. Ordnung verfolgt, wobei ein elastisch-starrplastisches Materialverhalten vorausgesetzt wurde.

Diese Kontrollen zeigen, dass die Sicherheit auch bei vermindertem Fahrbahngewicht nicht ganz ausreicht. Zudem waren unterhalb der Fahrbahn nur waagrechte Verbindungen zwischen den Pfosten vorhanden. Der obere Bereich der Querscheiben wurde deshalb K-artig ausgefacht (Fig. 2): man erreicht eine einwandfreie Stützung der T-Pfosten sowie eine Verminderung deren Einspannlänge, daher mit bescheidenen Kosten eine fühlbare Erhöhung der Tragkraft der Obergurte.

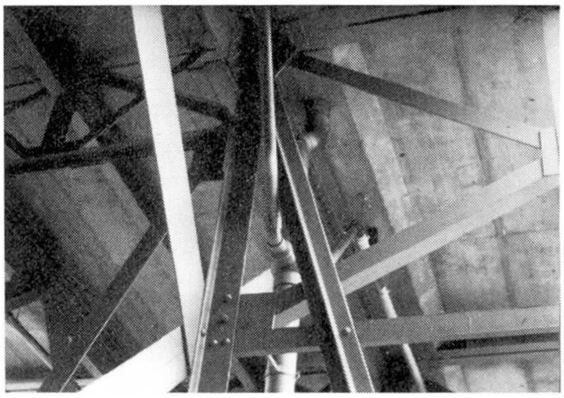

Fig. 2 Verstärkung der Querscheiben mit K-artiger Unterteilung

5.3 Füllungsglieder der Aussenträger

Fig. 3a zeigt die Ausfachung der Aussenträger, mit vier Streben in jedem Vertikalschnitt. Die Stabkräfte wurden elektronisch bestimmt, wobei einerseits die Biegesteifigkeit der durchgehenden Gurte und der Endpfosten, andererseits die Exzentrizitäten infolge der veränderlichen Anzahl Lamellen berücksichtigt wurden. Dagegen betrachtet man die Strebenanschlüsse als gelenkig und führt deren Kreuzungspunkte nicht als gemeinsame Knoten ein.

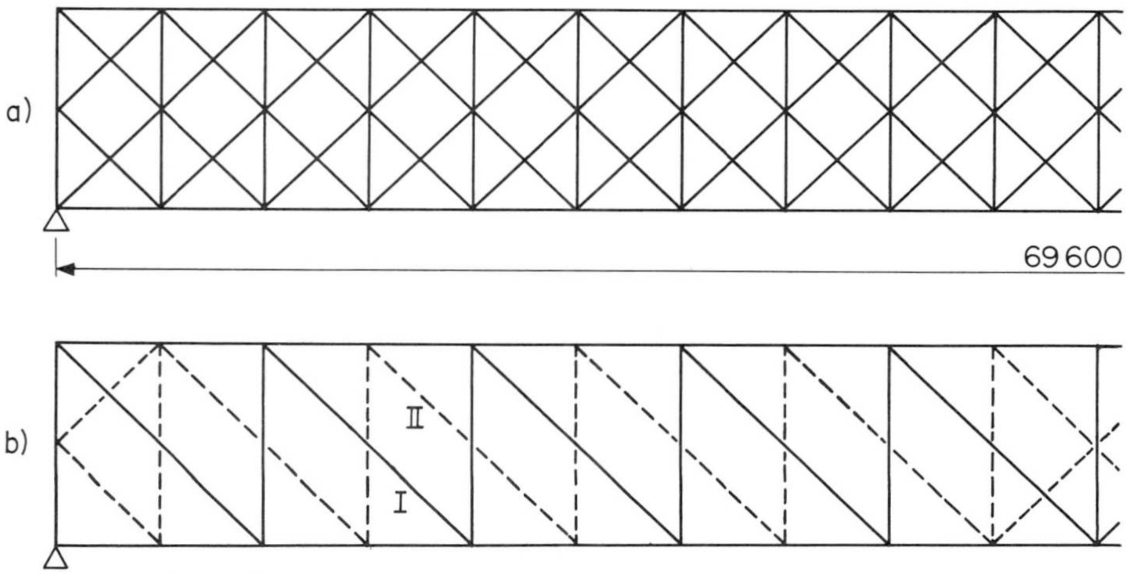

Fig. 3 Ausfachung im unterkritischen a) und im überkritischen b) Knickbereich

Bei der Bestimmung der Knicklänge der Druckstreben wurde berücksichtigt, dass diese Stäbe in ihrer Mitte durch die in Fig. 2 abgebildeten K-Verbände seitlich gehalten sind und zudem in den Viertelspunkten durch die Zugdiagonalen elastisch gestützt sind. Die Ausbildung als aussenliegender Doppelwinkel bedingt allerdings Exzentrizitätsmomente, welche den Knickwiderstand abmindern. Die Sicherheit beträgt somit nur ca. 1,4.

Zu Beginn des Knickvorganges nehmen aber die Druckstreben ihre Traglasten weiterhin auf, so dass nur die darüber hinausgehenden Querkräfte (entsprechend 1,8 - 1,4 als Sicherheitsmass) einem überkritischen Tragsystem zuzuweisen sind: es handelt sich um ein doppeltes Ständerfachwerk (Fig. 3b) mit den gleichen Zugdiagonalen wie in der unterkritischen Ausfachung (Fig. 3a) und mit den Druckpfosten, die im primären System nur wenig beansprucht sind und daher genügende Tragreserven besitzen. Auch hier wirken sich die neuen K-Verbände bezüglich des Knickens der Ständer aus der Ebene günstig aus. Was die Zugstreben anbelangt, so genügen sowohl die Querschnittsflächen als auch die Nietanschlüsse. Da die Ausfachung II nach Fig. 3b als K-Feld endet, musste die entsprechende Druckstrebe leicht verstärkt werden.

Nach diesem Tragmodell soll sich die Ausfachung ähnlich wie der dünnwandige Steg eines Blechträgers im überkritischen Beulbereich verhalten, mit einem Schubwiderstand $V_u = V_\tau + V_\sigma$. Der Anteil V_τ resultiert aus den Schubspannungen, d.h. aus zu 45^o geneigten Hauptspannungen, welche den Stabkräften der Zug- und Druckstreben der Ausfachung a) entsprechen. Der Zugfeldanteil V_σ ist mit demjenigen aus dem doppelten Strebenfachwerk (System b) zu vergleichen.

Die Berücksichtigung der überkritischen Umlagerung der Strebenkräfte zur Erzielung einer genügenden Sicherheit scheint bei einer kaum der Ermüdung ausgesetzten Strassenbrücke unbedenklich (Pumpeffekte im Gebrauchszustand ausgeschlossen).

5.4 Innenträger

Die Innenträger wurden 1880 zur Aufnahme von je 25 % der Gesamtlast bemessen. Wegen ihrer gegenüber den Aussenträgern um 1/4 verminderten Höhe erhielten sie entsprechend stärkere Gurte und Streben. Da diese Träger in Wirklichkeit nur 20 % aufzunehmen haben (vgl. 5.1), reichen hier sowohl der Biegewiderstand (Druckgurte durch die Fahrbahn stabilisiert) als auch der Schubwiderstand aus.

5.5 Einflusslinien für die Füllungsglieder des doppelten Ständerfachwerkes

Beim doppelten Ständerfachwerk nach Fig. 3b bzw. Fig. 4 hängt der Verlauf der Einflusslinien der Füllungsglieder stark von den Berechnungsannahmen ab. In Fig. 4 bezieht sich der Linienzug a) auf ein Fachwerk mit über den Knoten durchgehenden Gurtungen (vgl. 5.3), wie dies der konstruktiven Gestaltung entspricht. Linie b) gilt für ein System mit gelenkigen Gurtanschlüssen (CULMANN), während bei c) das zentrale Andreaskreuz als inaktiv gedacht ist. Das mehrfache Fachwerk ist dann statisch bestimmt [5]. Der durchgehende Strebenzug mit der Diagonale 34 übernimmt direkt die in den geraden Knoten angreifenden Lasten und überträgt sie in üblicher Art zu den Lagern. Die in Trägermitte unterbrochene Ausfachung mit der Strebe 28 leitet die Belastung der ungeraden Knoten nur zum linken Lager, so dass der durchgehende Strebenzug den Ausgleich zu sichern hat.

Ein Andreaskreuz in Trägermitte koppelt die zwei Ausfachungen und glättet die Zacken der Einflusslinien zum Teil aus. Die Biegesteifigkeit durchgehender Gurte übt einen ähnlichen, sogar stärkeren Einfluss aus: jede der zwei Streben nimmt dann rund die Hälfte der anfallenden Querkraft auf. Beim doppelten Strebenfachwerk, wie auch beim Rautenträger, führt somit die CULMANNsche Annahme zu Ergebnissen, die von der Wirklichkeit stark abweichen können.

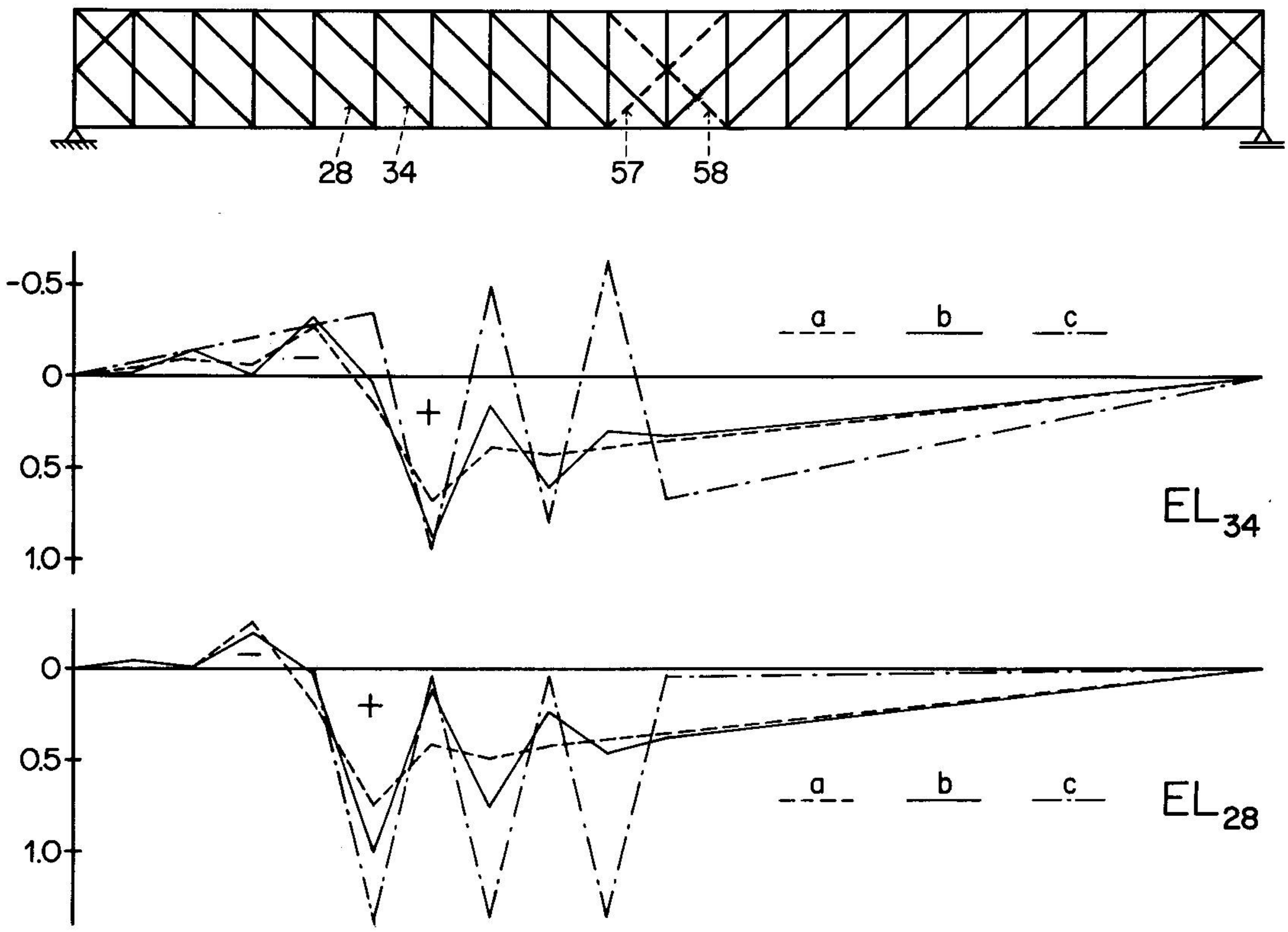

Fig. 4 Einflusslinien für zwei Streben eines doppelten Ständerfachwerkes. a) Gurte durchlaufend; b) CULMANNsches Fachwerk; c) Statisch bestimmtes Ständerfachwerk ohne zentrales Andreaskreuz (57 + 58)

6. SCHLUSSFOLGERUNGEN

Die Erneuerung einer alten genieteten Brückenkonstruktion ist nur dann wirtschaflich vertretbar, falls es gelingt, Verstärkungen auf wenige Elemente zu beschränken. Bei der Reichenauer Brücke konnte diese Bedingung dank genauerer Untersuchungen erfüllt werden, so dass ein Zeuge der technischen Entwicklung bei vernünftigen Erneuerungskosten für die Nachwelt erhalten bleibt.

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Wheeling Suspension Bridge, USA — Case Study

Le pont suspendu de Wheeling, USA — Etude de cas

Wheeling Hängebrücke, USA — eine Fallstudie

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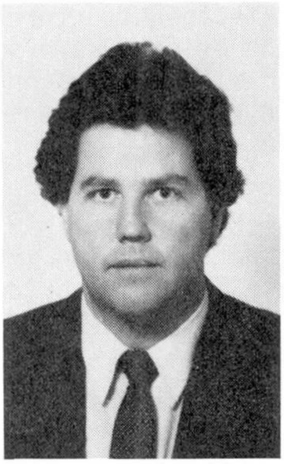

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SUMMARY
This paper reports on all aspects of the inspection, condition and rehabilitation of the Wheeling Suspension Bridge, WV, USA, one of the worlds oldest existing suspension bridges.

RESUME
Cet article présente en détail l'inspection, l'état réel et la rénovation du pont suspendu de Wheeling, USA, actuellement un des plus anciens ponts suspendus du monde.

ZUSAMMENFASSUNG
Dieses Manuskript erklärt im Detail die Inspektion, den Zustand und die Renovation der Wheeling-Hängebrücke, eine der ältesten, existierenden Hängebrücken der Welt.

INTRODUCTION

Spanning the Ohio River at Wheeling, West Virginia, is the historical Wheeling Suspension Bridge. The structure is one of the world's oldest existing suspension bridges and was the first bridge built with a span length exceeding 300 meters. In 1969, the American Society of Civil Engineers designated the bridge as a National Historic Civil Engineering Landmark, and in 1975, the National Park Service designated the bridge as a National Historic Landmark. It was designed by Charles Ellet, a civil engineer, who acquired much of his knowledge and experience in suspension bridges while studying in Europe. The suspension bridge was completed at an estimated cost of $250,000 and first opened to traffic on August 1, 1850. It is interesting to note the structure's original design live loads. According to Ellet, the bridge was designed for 2640 kN of live load or that represented by 16 six-horse wagons and 500 people occupying the bridge at one time. Other representations were that of 700 head of cattle or an army of 4,000 men.

On the afternoon of May 17, 1854, violent winds caused excessive vertical oscillations of the bridge which resulted in the cables breaking loose from their anchorages and the deck dropping into the river. This collapse provided engineers with quite a lesson on bridge aerodynamics.

The structure was rebuilt by Mr. Ellet using much of the original material and was reopened in 1856. In 1872 a system of radiating stay cables from the tower top to the deck was installed to stiffen the structure in accordance with a scheme designed by the famous Roeblings. Since that time, a number of repairs and modifications have been made. The last major modification was in 1956, when the entire timber floor was replaced by open steel grating supported on steel floorbeams.

GENERAL DESCRIPTION OF BRIDGE

The present day bridge has a main span of 307 meters between centers of masonry towers. The bridge roadway is 6.1 meters wide accommodating two lanes of automobile traffic and there are two-1.2 meter sidewalks. A general elevation and typical deck cross section of the bridge are given in Figure 1.

There are four main suspension cables supporting the bridge, two along each side. Each cable is approximately 19 centimeters in diameter and consists of 2200 wrought iron wires laid parallel. Each wire has a diameter of 3.5 millimeters. To exclude moisture and maintain compactness, each cable is tightly wrapped with 2 mm wire. The specifications under which the iron wire was furnished were not available. Tensile tests were therefore performed which indicated the wire to have an average breaking strength of 5780 N. This correspondends to an ultimate tensile stress of 600 N/mm^2 quite remarkable for its time.

The longitudinal stiffening trusses of the bridge are of timber construction. They are classic Howe trusses with counters in every panel. The diagonals are capable of resisting compression only. The chords and diagonals are fabricated from structural timber and the verticals are wrought iron rods. The truss depth is 1.9 meters which is only 1/160 of the span length. The combination of timber construction with small depth results in a rather small stiffness for a span over 300 meters long. The trusses are braced laterally by curved steel members extending out from the floorbeams.

The radiating stay cables are wire ropes which extend from the tower top to various points along the bottom chord of the stiffening truss. They have diameters varying from 25 millimeters to 44 millimeters and are clamped to the cable hangers at each intersection. There is a system of lateral sway cables consisting of wire ropes which are connected to the floorbeams and main cables and are anchored into the river banks with partially buried masonry deadmen.

The cable hangers are 2.9 millimeter diameter rods and are connected to the floorbeams at each panel point.

The deck system is composed of open steel grating supported by wide flange floorbeams spaced at 2.44 meter centers.

BRIDGE INSPECTION

The bridge inspection was conducted during July and August of 1978 and a supplemental cable inspection was made during January and February of 1979. The inspection consisted of an in-depth visual observation of all parts of the structure. A longitudinally moving scaffold was rigged beneath the superstructure so that the entire underside of the deck could be inspected. Due to the lack of a cable walkway, a hydraulically operated aerial lift platform was used to inspect the main cables.

The main cables were unwrapped and wedged open in thirteen different locations to observe the condition of the parallel wires. Cable bands were removed and inspected at four locations. The cables were inspected inside all four anchorages.

BRIDGE CONDITION

In general, the wire wrapping of the cables was found to be excessively corroded, loose and broken away in many locations, all of which allow deterioration of the underlying wires. Broken and severely corroded cable wires were discovered at a number of locations of which the most severe are described below.

The outer cable at Panel Point 16 South had approximately 50 broken wires and an additional 85 wires with up to 50 percent loss of section. The inner cable at Panel Point 123 North had 15 broken wires and approximately two hundred wires with excessive section loss. In the northeast anchorage, the inner cable wires were found to be seriously corroded. The corrosion was the result of water leaking through the anchorage ceiling. We estimated that 125 wires were broken or seriously corroded at this location.

In general, the cable hangers exhibited minor corrosion. As shown in Figure 1, one hanger from each cable is connected to the floorbeam at every panel point. At a number of panel point locations, one of the hangers was slack and carrying no load while the adjacent hanger was carrying the full load.

Many of the radiating stay cables were excessively slack and exhibited considerable corrosion. As previously mentioned, these stay cables are clamped to the cable hangers at each intersection. At many locations these clamps have become loose and worn. This has resulted in a constant rubbing action between the two members. Over a time span of many years, the rubbing action has created excessive section loss to both the cable hanger and stay cable.

The timber stiffening trusses were found to be in the advanced stages of decay. The upper and lower chords were considerably rotted throughout 90 percent of their length. More than fifty percent of the timber diagonals were loose and a number of chord splices were found to have failed.

The floorbeams, laterals and bridge grating were in generally good condition with only minor corrosion. The most significant areas of corrosion were at the two ends of the bridge. At these locations, salts and other debris which are carried onto the bridge by automobiles, pass through the open grating and collect on the flanges of the floorbeams.

BRIDGE REHABILITATION

Of prime importance in developing rehabilitation plans was to maintain as much as possible the historical appearance of the structure. The following paragraphs discuss the major items involved in the repair.

The wire wrapping will be completely removed and the cables rewrapped using a neoprene elastomeric wrapping system. This system consists of a first coat of liquid neoprene applied to the cleaned surface of the cable. Next a double thickness of 1.6 mm uncured neoprene is wrapped spirally around the cable with a 50% of overlap. Following this, two coats of Hypalon paint are applied to the neoprene sheet. After careful study, we found the system to be less costly, lighter, and to require less maintenance than the conventional wire wrapping. Another important reason for our recommendation of this system of cable wrapping was that due to the close proximity of the main cables at each side of the bridge, the proper installation of wire wrapping using a wrapping machine would be impossible. Installation of a wrapping system of this type has been performed satisfactorily on a number of bridges.

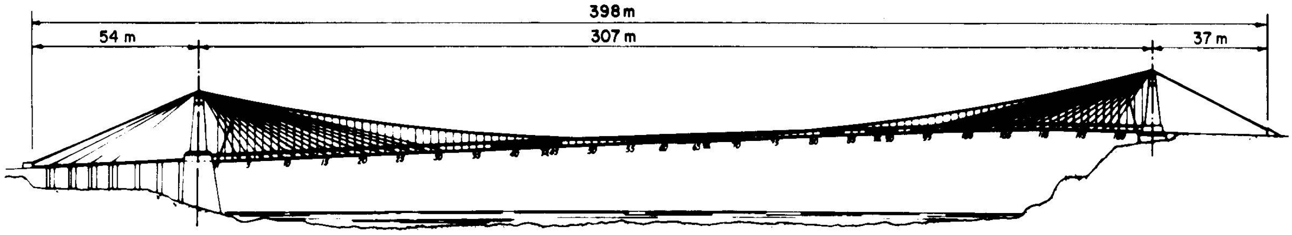
At those locations where broken or severely corroded cable wires occur, new wires will be spliced in. Two methods of splicing were investigated. First, a threaded coupling system was studied. Sample threaded couplers were fabricated and tested at West Virginia University using original bridge wire. The tests showed that the wires failed at an average load of only 60 percent of the ultimate wire strength. In every test the wire failed within the threaded length. Another drawback of this system was that the torque due to the threading process was causing the wire to delaminate. In addition, it would be tremendously difficult to thread the wires in the field. This system was therefore not considered acceptable. We next investigated a swaged type coupling. Couplings of this type have been used in the original construction of many suspension bridges including the Second Delaware Memorial Bridge and the Newport Bridge. This coupling is manufactured by CCL Systems of Leeds, England. Tests performed using these swaged couplers on the original bridge wire showed that the coupler could develop the ultimate wire strength. We found this system to be satisfactory and have therefore recommended it for the wire repair. Prior to performing any wire repairs, the timber trusses, representing fifteen percent of the total dead load on the cables, will be removed.

The cable hangers and stay cables with excessive section loss will be replaced. New clamps will be installed to connect the stay cables to the hangers. The timber trusses will be completely replaced using a dense graded select structural Douglas fir. The timber will be treated using a pentachlorophenol treatment. The chord splices will be made using modern split ring connectors. The stay cable connections to the bottom chord will be made using a steel anchorage plate connected to the chord with special shear plate connectors. All of the steelwork will be blast cleaned and painted using a three coat system.

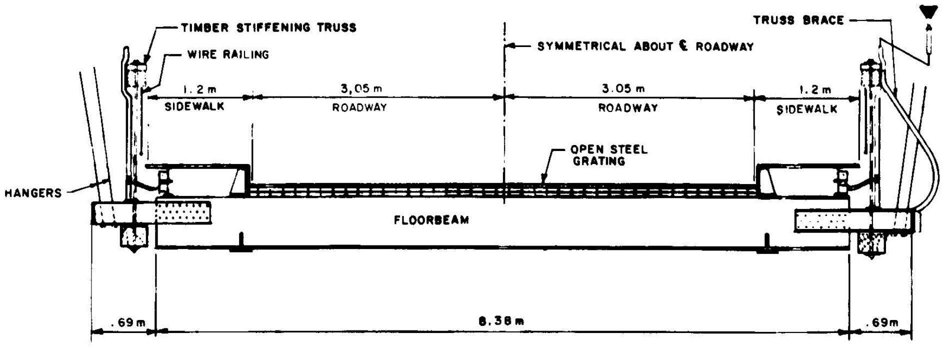
STRUCTURAL ANALYSIS

In order to prepare design plans and specifications for the rehabilitation of the Wheeling Suspension Bridge, an accurate analysis of the structure was essential. The analysis of the bridge presents a problem quite different from that of the typical suspension bridge. The highly indeterminate effect of the stay cables must be included in the analysis. Moreover, the non-linear behavior of suspension bridges is well known and is considered in design. Preliminary linear analysis of the Wheeling Bridge showed the deformations to be large enough to cause significant change in the geometry of the structure. Therefore, a non-linear analysis was required to solve for equilibrium in the deformed position under applied loads. In a typical cable stayed bridge, the horizontal components of the cable forces produce compression in the deck. However, for the Wheeling Bridge these horizontal cable forces produce tension in the stiffening truss. In addition, the stay cables in a typical cable stayed structure have a very large initial dead load tension and therefore remain in tension for all conditions of live load. The stay cables in the Wheeling Bridge have only a minimal dead load tension. As a result, various positions of live load will cause some stay cables to become slack. Therefore, an iterative analysis had to be performed neglecting those stay cables which become slack in the final solution.

The repair contract ($2.6 million) is presently underway and is scheduled for completion in early 1983. Were it not for the structure's great historical significance, the cost of the repairs could not be justified. Upon completion of the repairs, the bridge will be capable of carrying passenger vehicles only.



ELEVATION



TYPICAL DECK CROSS SECTION

WHEELING SUSPENSION BRIDGE
FIGURE 1