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PRESENTATIONS

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Zweckässigkeit und Umfang von Instandsetzungen

Utilité et étendue de réfections

Suitableness and extent of rehabilitations

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Tilman Zichner, geboren 1944, promovierte an der Technischen Hochschule Darmstadt. Er ist hauptsächlich auf dem Gebiet des Brückenbaus tätig, hier auch verstärkt beschäftigt mit Problemen der Sanierung und Instandsetzung.

ZUSAMMENFASSUNG

Sanierungsarbeiten an Brücken sollten — soweit dies möglich und sinnvoll erscheint — durch Wertverbesserungen und die Beseitigung von Schwachpunkten ergänzt werden. Ist der hierfür notwendige Aufwand zu hoch, muss über die Alternative "Bauwerksersatz" nachgedacht werden. An drei Brücken wird die grosse Bandbreite dieses Arbeitsgebietes erläutert.

RÉSUMÉ

Les réfections de ponts devraient être complétées — si c'est possible et raisonnable — par une augmentation de la valeur et par l'élimination des points faibles. Si les dépenses nécessaires sont trop grandes, il faut considérer l'alternative "remplacement de l'ouvrage". L'exemple de trois ponts révèle la grande diversité de ce champ d'activité.

SUMMARY

Rehabilitation of bridges should be supplemented — as far as it is considered to be possible and reasonable — by an increase of value and by the elimination of weak points. If the necessary expenditure related to these demands is too large, the alternative "replacement of the structure" has to be considered. Three bridges are taken as examples to explain the large variability in this field of engineering.

1. EINLEITUNG

Fehler bei Konstruktion und Bauausführung von Brückenbauwerken, aber auch Anfahrunfälle und geänderte Nutzungsvorstellungen führen zu Instandsetzungsmaßnahmen, die z.T. auch mit Bauwerksverstärkungen verbunden sein können. In solchen Fällen, in denen aus gegebenem Anlaß Sanierungsarbeiten erforderlich werden, sollte grundsätzlich die Frage aufgeworfen werden, ob über die Therapie des akuten Problems hinaus nicht sinnvolle Wertverbesserungen oder nützliche Eliminierungen von Schwachpunkten gleich im selben Arbeitsgang mit erledigt werden können.

Dies kann zwar zu umfangreichen Umbauarbeiten führen, liefert aber ein wartungsärmeres und dauerhafteres Bauwerk. Jedoch muß der erforderliche Aufwand - sofern keine anderen Gründe, wie z.B. Denkmalschutz vorliegen - in einem vertretbaren Rahmen bleiben. Läßt sich das nicht erreichen, so muß die Alternative "Abbruch und Neubau" mit in die Planung einbezogen werden.

An den drei nachfolgend geschilderten Beispielen, die sich durch ihr Schadensbild und die ausgeführte Problemlösung stark voneinander unterscheiden, soll die große Bandbreite dieses Arbeitsgebietes dargestellt werden.

2. BRÜCKE ÜBER DIE SAAR, BAUJAHR 1955

Das Bauwerk mit einer Gesamtlänge von 147 m besteht aus drei Zweigelenkbögen, die aus jeweils drei Bogenscheiben gebildet werden. Die Scheiben sind über die Fahrbahnplatte und mehrere Querträger miteinander verbunden (s. Bild 1).

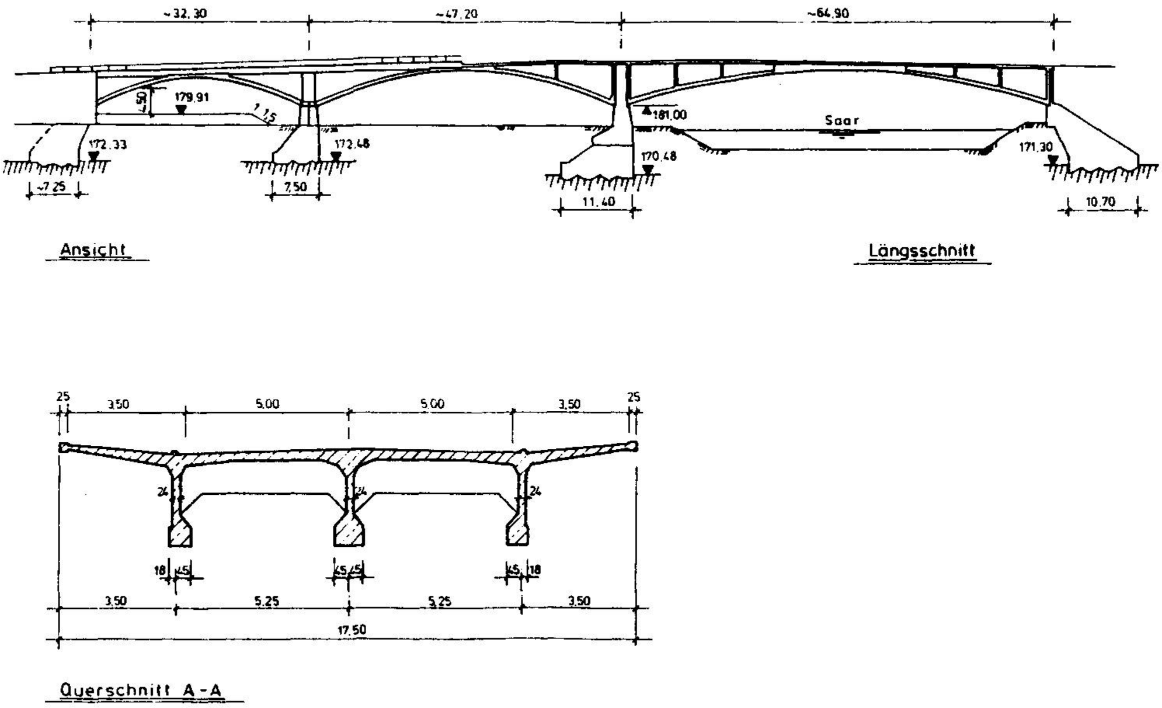


Bild 1 Von-Lettow-Vorbeck-Brücke in Saarlouis

Der Überbau ist längs und quer vorgespannt mit Spanngliedern aus Drähten von 7 mm Durchmesser. Als Stahlgüte wurde ein St 135/150 verwendet.

In den Scheitelbereichen aller drei Bögen wurden bei einer routinemäßigen Brückenprüfung Risse festgestellt, deren Breite mit größer werdender Spannweite der Bögen zunahm. Während die Rißbreite im kleinen Landfeld nur bei 0,2 - 0,3 mm lag, erreichte sie im Mittelfeld schon 1,2 mm und wuchs im großen Saarfeld auf 2,5 mm an.

Eine Überprüfung des Verpreßgrades der Hüllrohre ergab, daß einige Spannglieder nicht oder nur teilweise verpreßt waren. Die Spanngliedoberfläche zeigte Korrosionsansätze, die nicht nur in den Bereichen mit Verpreßmängeln, sondern auch in vollständig verpreßten Strecken zu beobachten waren.

Eine Nachrechnung des Überbaus führte zu dem Ergebnis, daß in den Bereichen, wo die Risse beobachtet worden sind, hohe Zugspannungen im Beton vorhanden waren. Die Breite der Risse (max. $w = 2,5$ mm) ist hauptsächlich auf den geringen inneren Hebelarm der Spannbewehrung und auf die sehr schwach ausgebildete schlaffe Bewehrung zurückzuführen. In den Untergurten der Bogenscheiben sind lediglich 6 Ø 8 = 3 cm^2 aus Stahl I vorhanden.

Unter diesen Umständen war es auch nicht weiter verwunderlich, daß die Bruchsicherheit im Scheitelbereich nicht im erforderlichen Rahmen nachgewiesen werden konnte.

Neben diesen gravierenden Mängeln bezüglich der Standsicherheit haben die Untersuchungen von Spannstahl und Injektionsmörtel einen weiteren, schwerwiegenden Mangel aufgezeigt, der die Dauerhaftigkeit des Bauwerk in Frage stellte.

So zeigten die an den Spannstahlproben durchgeführten Untersuchungen zwar, daß der Stahl hinsichtlich seiner mechanischen Eigenschaften kaum nennenswerte Einbußen erlitten hat, daß er aber durch die im Verpreßmörtel festgestellten korrosiven Bestandteile (Chlorid, Cyanid, Sulfid, Sulfat und Phosphat) bei Anwesenheit von Feuchtigkeit und Sauerstoff vor Spannungs- und Schwingungsrißkorrosion nicht geschützt ist.

Eine Sanierung des Bauwerks hätte neben der Gewährleistung der Bruchsicherheit auch die Verhinderung einer erneuten Rißbildung zur Vermeidung des Zutritts von Feuchtigkeit und Sauerstoff zur Spannstahloberfläche bieten müssen. Hierfür wäre nur eine sehr aufwendige Verstärkung mittels externer Vorspannung in Frage gekommen.

Unter Abwägung der zu erwartenden Schwierigkeiten und der erforderlichen Kosten hat man sich nicht für eine Sanierung entschieden, sondern dem Abbruch und einem Neubau den Vorzug gegeben.

3. BRÜCKE ÜBER DIE MOSEL, BAUJAHR 1963

Diese Spannbetonbrücke besteht aus vier Feldern mit Spannweiten von 35,70 m - 84,50 m - 42,25 m - 31,45 m. Im großen Flußfeld besitzt der Überbau in der Mitte ein Gelenk. Wegen des nur kurzen Feldes, das auf der einen Seite anschließt, mußte dort zur Sicherung gegen Abheben von den Lagern eine Zugabspannung vorgesehen werden (s. Bild 2).

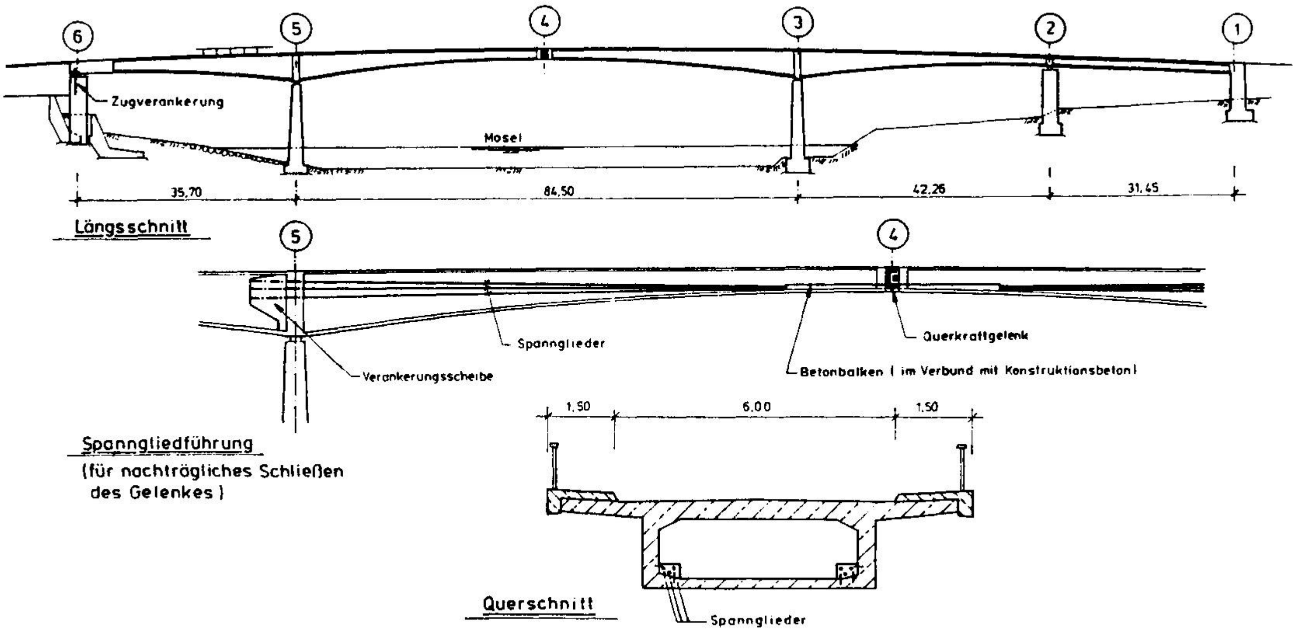


Bild 2 : Moselbrücke bei Thörnich

Dieses Bauwerk wies nun Anfang der 80-er Jahre zwei schwerwiegende Schäden auf: die stark korrodierte, nicht mehr bewegliche Zugabspannung zur Kippsicherung des nördlichen Überbauabschnittes und die starke Durchbiegung im Gelenk des Stromfeldes.

Durch die defekte Abspannung ist über fehlende Funktionsfähigkeit und Dauerhaftigkeit letzlich auch die Standsicherheit betroffen. Der Knick in Strommitte beeinträchtigt dagegen das Erscheinungsbild und durch die Verminderung des Fahrkomforts auch die Funktionsfähigkeit.

Durch die Sanierung sollte nun nicht nur eine Wiederherstellung von Standsicherheit und Gebrauchsfähigkeit durch Erneuerung der Zugabspannung und Ausgleich des Knicks erreicht werden, sondern es sollte eine wesentliche Verbesserung erzielt werden, was nur durch die Elimination des Schwachpunktes der Verankerung am Widerlager zu bewerkstelligen war. Zudem war zu bedenken, daß ein Ausgleich des Knickes durch Aufbeton zu erheblichen Schnittgrößen aus zusätzlichem Eigengewicht geführt hätte, die mit einer Abstufung der Brückenklasse nach DIN 1072 verbunden gewesen wären, da keine Tragreserven mehr vorhanden waren.

Das Instandsetzungskonzept sah daher, nach intensiven statischen Untersuchungen, vor, das Gelenk in Feldmitte zu schließen und das Stromfeld durch zusätzliche Längsspannglieder zur Aufnahme der beim Durchlaufträger auftretenen positiven Feldmomente zu ertüchtigen. Die noch fehlende Auflast am Widerlager zur Gewährleistung der Lagesicherheit wurde durch Ballastbeton im Hohlkasten erzielt.

Bei dieser Maßnahme mußte, da die Bewegungsmöglichkeit in Längsrichtung durch das Schließen des Gelenkes verhindert wurde, das Lagerungssystem vollständig neu konzipiert werden. Dies fiel jedoch nicht weiter schwer, da durch die Anhebung des Überbaus zum Ausgleich des Knicks die Lager auf jeden Fall hätten versetzt werden müssen.

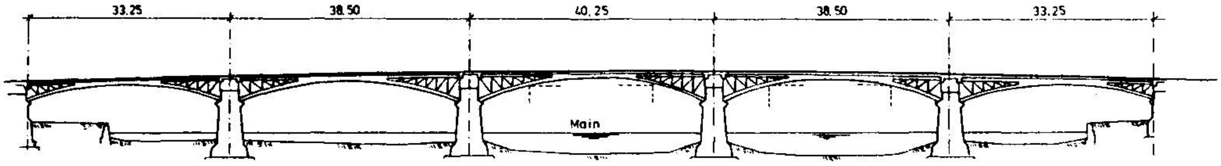
Zusammenfassend sind zur Verbesserung von Standsicherheit und Gebrauchsfähigkeit folgende Maßnahmen ausgeführt worden:

- Anheben des Überbaus
- Änderung des Lagerungssystems mit Austausch der Lager
- Einbau zusätzlicher Spannglieder im Stromfeld
- Schließen des Gelenks
- Ausbau der Abspannung und Einbau von Ballastbeton
- Einbau neuer, wasserdichter Übergangskonstruktionen
- Einstiegsöffnungen in der Bodenplatte
- Neue Leichtbetonkappen, neues Geländer
- Neue Abdichtung und Belag
- Betonsanierung, Anstrich

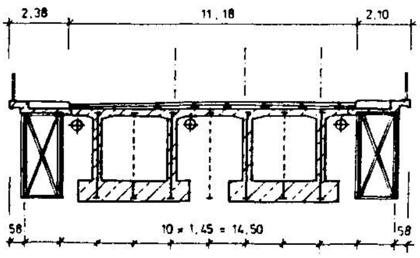
Durch diese Maßnahmen konnte das Bauwerk auch weiterhin in BKL 30 nach DIN 1072 eingestuft werden.

4. BRÜCKE ÜBER DEN MAIN, BAUJAHR 1878

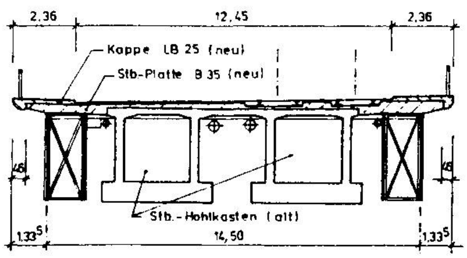
Diese Brücke über den Main mit 5 Bögen von i.M. 35 m Spannweite war 1878 zunächst als Stahlfachwerkbrücke mit 11 nebeneinanderliegenden Trägern ausgeführt worden. Nach dem Krieg wurde der mittlere Teil im Bereich der Fahrbahn zur Erhöhung der Tragfähigkeit durch eine Stahlbetonkonstruktion ersetzt. Hierbei wurden die Stahlfachwerke z.T. in die Konstruktion integriert. Während der Bauzustände bildeten sie das Traggerüst für die Betonierlast. Im Randbereich unter den Gehwegen blieben die stählernen Fachwerke erhalten (s.Bild 3).



Längsschnitt



Querschnitt (vor der Instandsetzung)



Querschnitt (nach der Instandsetzung)

Bild 3: Obermainbrücke in Frankfurt a.M.

Dieses Bauwerk wies stärkere Schäden im Bereich der Fahrbahnplatte auf, die alle von Undichtigkeiten an den zahlreichen Fugen herrührten. An jedem Pfeiler waren 2 Querfugen und zwischen Massivteil und Stahlfachwerk war jeweils eine Längsfuge über die gesamte Bauwerkslänge vorhanden.

Das Instandsetzungskonzept sollte nicht nur zu einer Behebung der aufgetretenen Schäden, sondern auch zu einer weitgehenden Elimierung der Schwachpunkte führen. Zusätzlich sollte in der Vorplanung untersucht werden, ob nicht eine Verbreiterung der Fahrbahnplatte um ca. 1,50 m möglich sei, damit drei vollwertige Fahrstreifen gewonnen werden können.

Zunächst wurde eine vollständige Umgestaltung der Pfeilerbereiche vorgenommen, um die Anzahl der Querfugen pro Pfeiler von 2 auf 1 zu reduzieren und gleichzeitig Besichtigungs- und Wartungsmöglichkeiten zu schaffen.

Dazu gehörte auch die Umsetzung der Abspannmaste, die außerhalb der Fahrbahnplatte angeordnet worden sind, um Durchdringungen durch die Abdichtung zu vermeiden.

Die Fahrbahnplatte wurde ohne Ausbildung einer Längsfuge verbreitert, was einerseits zur Behebung des Schwachpunktes Fuge, andererseits aber aus statischen Gründen notwendig war. Zur Abtragung der Verkehrslasten wären die Stahlfachwerke allein nämlich nicht in der Lage gewesen; hierzu bedurfte es der Mitwirkung der steifen Stahlbetonkastenträger, die nur durch die monolithische Verbindung über die Fahrbahnplatte zu aktivieren waren.

Für den Anschluß der neuen Platte an die bestehende Konstruktion mußten die Kragarme und die Gehwege abgebrochen und die Gradiente um einige Zentimeter angehoben werden.

Eine Überprüfung der Standsicherheit der neuen Konstruktion unter Belastung nach BKl 60/30 und Straßenbahnverkehr unter Berücksichtigung der räumlichen Tragwirkung ergab, daß die bestehende Stahlbetonkonstruktion ausreichend dimensioniert war, auch die erhöhte Belastung aufzunehmen. Die Stahlkonstruktionen hingegen mußte an einigen Stellen verstärkt werden, z.B. durch zusätzliche Bleche oder Austausch von Niete gegen HV-Schrauben.

Durch die ausgeführte Maßnahme wurde nicht nur die Tragfähigkeit des Bauwerks auf BKl 60/30 nach DIN 1072 angehoben, sondern auch die Gebrauchsfähigkeit - nicht nur gewährleistet, sondern darüber hinaus - verbessert. Durch die Vermeidung unnötiger Fugen und die Verbreiterung auf drei volle Fahrstreifen wurde eine langfristige Verbesserung der Dauerhaftigkeit und Funktionsfähigkeit erzielt. Nicht unerwähnt soll bleiben, daß durch die architektonische Gestaltung der Instandsetzungsmaßnahmen auch das Erscheinungsbild des Bauwerks gewonnen hat.

Causes of Damages and Rehabilitation of the Pancevo Bridge

Causes des dommages et restauration du pont de Pancevo

Schadenursachen und Sanierung der Pancevo-Brücke

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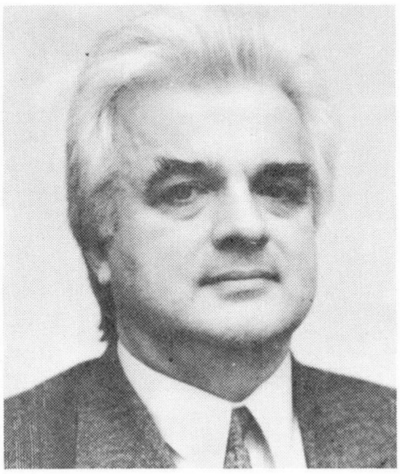

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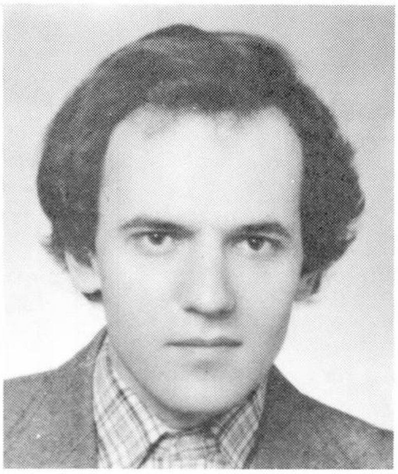

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SUMMARY

Access parts of the Pancevo bridge across the river Danube in Belgrade were built at the beginning of 1960s among the first prestressed concrete bridges in Yugoslavia. Numerous cracks in the concrete and in places very corroded prestressing wires have resulted from insufficient prestressing force, poor corrosion protection, poor drainage and by other failures in design, execution operation and maintenance. The rehabilitation covered injecting of the cracks, installation of new prestressing tendons inside the box cross-section, improvement of drainage and removal of other failures.

RÉSUMÉ

Les structures d'accès du pont de Pancevo sur le Danube à Belgrade ont été construites au début des années soixante parmi les premiers ponts en béton précontraint en Yugoslavie. De nombreuses fissures dans le béton et à certain endroits une corrosion très avancée des fils de précontrainte sont apparus à cause d'une force de précontrainte insuffisante, d'un mauvais enrobage des fils de précontrainte, d'une mauvaise évacuation des eaux de pluie et d'autres défauts dans les phases de projet, d'exécution, d'exploitation et de maintenance. La restauration comprend l'injection des fissures, la mise en place de nouveaux câbles de précontrainte à l'intérieur de la section en caisson, la réparation des systèmes d'évacuation d'eau et l'élimination d'autres défauts.

ZUSAMMENFASSUNG

Die Zugangsbrücken der Pancevo-Donaubrücke in Belgrad sind Anfang der sechziger Jahre als erste Spannbetonbrücken Jugoslawiens gebaut worden. Zahlreiche Betonrisse und stellenweise stark korrodierte Spannstähle sind die Folgen ungenügender Spannkraft, schwachen Korrosionsschutzes, schlechter Entwässerung und anderer Entwurfs-, Ausführungs-, Anwendungs- und Unterhaltungsfehler. Die Sanierung bestand aus einer Rissinjektion, dem Einbau neuer Spannstähle innerhalb des Hohlquerschnitts, der Instandstellung der Entwässerung sowie der Behebung von anderen Mängel.

1. GENERAL

Access parts of the Pancevo bridge across the river Danube in Belgrade had been made at the beginning of 1960s. They were among the first prestressed concrete bridge structures in Yugoslavia. The total length of 1.269 m is divided to 16 independent bridge structures interconnected by expansion joints, Fig. 1. The structures are continuous frames with two, three of five fields, of the span ranging between 20 and 40 m, with slender posts. The width of the bridges is (1.50+7.00+0.80) m with two carriageways to the same direction. The cross section is the constant height box of 1.45 m, Fig. 2.

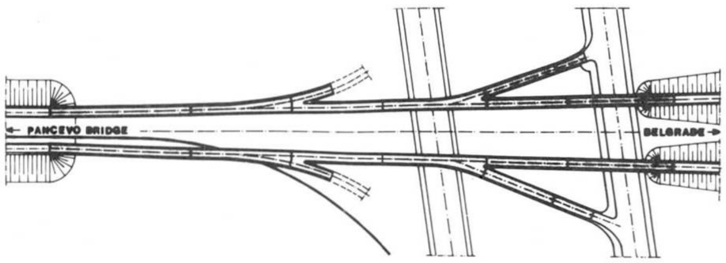

Fig. 1 - The lay-out of the access bridges

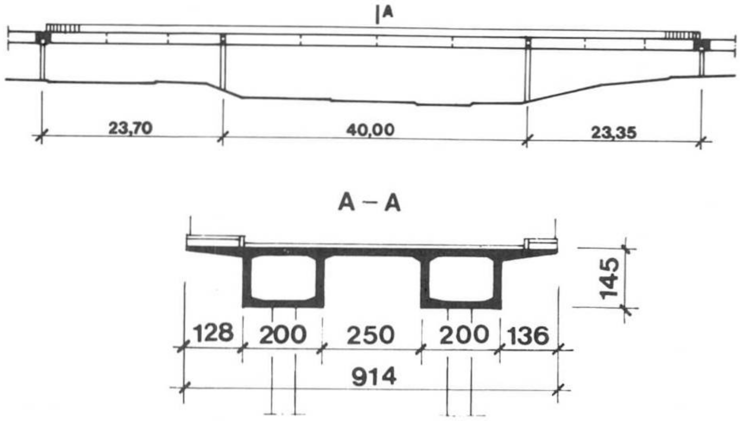

Fig. 2 - Disposition of a bridge

The prestressing of the main girders was made according to the Yugoslav IMS system. The tendons of 6Ø7 mm were conducted through steel tubes placed in the webs and the upper and lower chords. The deck of the bridge was partly prestressed in the lateral direction, too.

The medium posts are of circular cross section and have monolith connection with the main girder. The girders are supported at the ends, by sleeper beams where expansion steel bearings are placed via short elements of the reduced height.

Waterproofing of the structures has, unfortunately, not been either designed or erected.

2. CAUSES OF DAMAGES

Already during the test examination of the structures, before it was used for traffic, cracks were noticed on some of the structures and remained after the test loading was removed. That was why the majority of 40 m span structures were immediately subjected to interventions by adding a certain number of new tendons inside the box of the main girder.

During the subsequent inspections, a large number of cracks was noticed on the majority of structures. The cracks were mainly perpendicular to the main girders direction and distributed in the zones around the medium supports and about the middle of the span, Fig. 3. A detailed inspection of the structures has been made including the "survey" of cracks, both from the outside

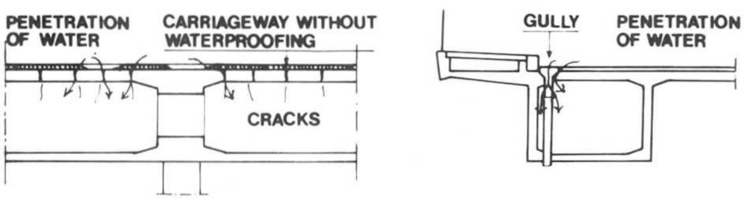

Fig. 3 - The penetration of water to the inside of the box

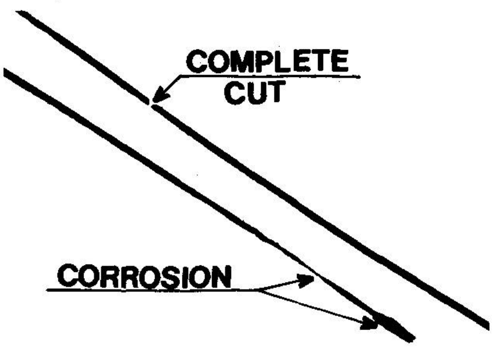

Fig. 4 - Corroded tendon

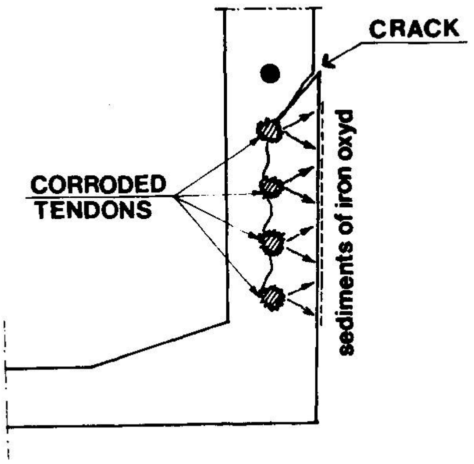

Fig. 5 - Irregular injected tendons

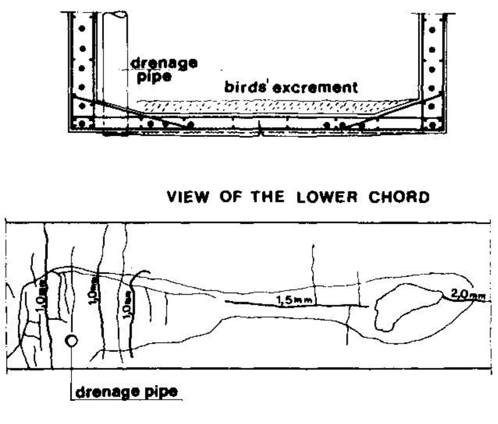

Fig. 6 - Damaged lower chord

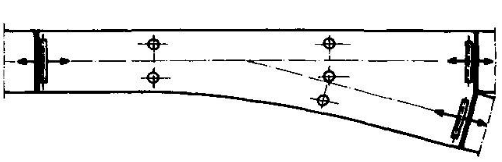

Fig. 7 - Irregular disposition of expansion bearings

and the inside, after washing the inside of the box. All the cracks have been registered, classified and inserted into the corresponding plans. All weak and suspected spots on the surface of the structures have been registered, too. The areas with a large number of cracks, with the sediments of iron oxide or lime and the areas which, under the blows of the hammer, demonstrated the existance of bubbles or sepatation of the concrete cover, had been opened in order to establish the level of the damage.

It has been found out that prestressing tendons were corroded on some of the places, from slight corrosion to complete cut, Fig. 4. Cement grout for injecting tendons was not of the required quality and on some of the places it did not exist, at all, Fig. 5. By measuring the existing force in the tendons, it was established that on a number of wires the designed prestressing force was not achieved. The results of measuring their own free oscillations and damping pointed to the decrease of general rigidity of the main girders.

Large damage and concrete deterioration zones have been tested and checked by ultrasonic, Fig. 6. The quality of the built in concrete and of the prestressing wires was examined, too.

Beside the structures, the other elements of the bridges have also been examined:bearings, expansion joints, gullies, drainage system, carriageway, fense and the like. The expansion joints were incorrect and broken, Fig. 7, the bearings contained empurities and were corroded, some of them outside the bearing slab or turned out and the gullies were either blocked or out of operation due to some other reasons. The carriageway was unsmooth and wavy with a large number of impact holes, Fig. 3. The posts and the foundations were in a good state.

On the basis of the results of the detailed inspection and examination as well as of the analysis of the complete design and technical documentation, the causes and the level of damages have been established together with the required rehabilitation measures.

The damages of the structure came as a result of various failures in designing, execution, exploitation and maintenance.

One of important design and execution failures is that waterproofing has not been planned. That has caused penetration of water and salt used for rubbing the bridge deck during the winter time. The penetration was to the inside of the box, through the bridge deck, and especially through cracks appearing in the upper zone, above the medium supports, Fig. 3. Incorrect and damaged gullies were another way of water penetration to the inside of the main girder box. Their designed position in the middle of the span is very inconvenient as that is the place where the tendons are concentrated in the lower zone and the lower chord through which the drainage tubes penetrate, Fig. 6.

The possibility of subsequent intervention in case that due to any reason, the designed prestressing force is not acquired, has not been anticipated by the design.

A certain number of crackes has appeared in the zones around 1/4 of the span due to incorrect anchoring of the tendons in the lower chord and webs as no care was taken of the necessary overlapping of the tendons at the places of anchorage, Fig. 8.

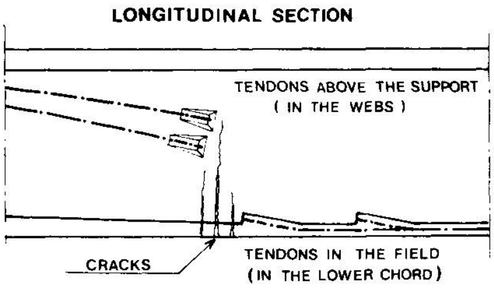

Fig. 8 - Irregular end of tendons

Long presence of water in the inside of the box has also contributed to the corrosion of tendons as no openings on the lower chord for drainage of possibly penetrated water have been designed. No wire net was placed at the openings for entering the inside of the main girders and for aeration so the inside of the box made a good "home" for birds. In time, a thick layer (10 cm and more) of birds'excrement, eggs and similar impurities was formed which had extensively increased aggressivity of the environment, Fig. 6.

The incorrect position and undesigned moving possibilities of expansion bearings have contributed to incorrect operation and undesigned stressing on some of the structures, Fig. 7.

The increase of weight and spead of vehicles in exploitation, frequently above the permitted limits, together with undesigned dynamic action due to driving of vehicles along the unsmooth and damaged carriageway and inadequate and damaged expansion joints have significantly contributed to the appearance of cracks and to further progressive increase of the level of damage suffered by the structures of this traffic route whose frequency is very high.

Incorrect and irregular maintenance of the pavement, of the gullies and expansion joints together with the application of salt for rubbing the carriageway have also considerably contributed to the appearance of the described damages. Incorrect expansion joints made the penetration of water and mud from the carriageway to the expansion steel bearings possible, which caused their damages and the cessation of the correct function.

3. REHABILITATION

On the basis of the detailed inspection, examination and establishment of the level of the damages, the rehabilitation designs for each separate structure have been made according to which the works were started with.

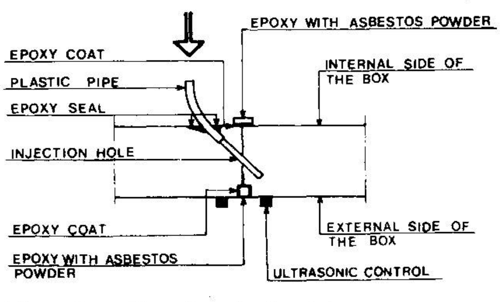

Fig. 9 - Cracks injecting

Within the rehabilitation of the structures, injecting of all cracks over 0.1 mm wide, by the application of special procedure, using low viscosity epoxy resin under pressure has been done. There was ultrasonic control of injecting, Fig. 9. During the execution of those works, the traffic on the bridge was completely stopped.

On one of the structures on which greater deterioration of the concrete, due to permanent wetting, and action of frost and salt took place, the whole damaged zone of the lower plate was replaced - several square metres, Fig. 6.

For improving the state of stress and in order to achieve the required prestressing force, supplementary prestressing has been performed using new tendons of the IMS system. The tendons 4Ø15.2 mm in polyethylene tubes are placed in the inside of the main girder boxes, Fig. 10. On the places on which the tendons

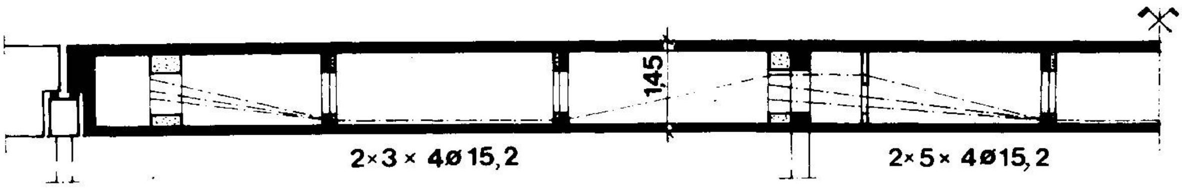

Fig. 10 - Disposition of additional tendons

are curved, correspondingly curved steel tubes have been placed instead of polyethylene tubes, Fig. 11. Such turnoff of the tendons has been executed on the places of the existing internal diaphragms whereat those diaphragms have been reinforced. On the places on which the tendons should be turned off but on which there have been no diaphragms, the new ones are made. The tendons are anchored into the newly built anchor blocks positioned in the inside of the box at the ends of the structure, as the access from the front of the structure was not possible, Fig. 11.

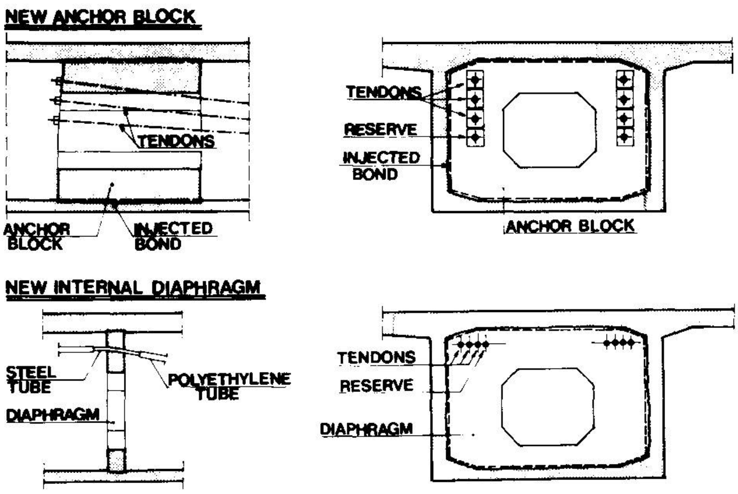

Fig.11-Details of the anchor block and diaphragm

Special attention has been paid to the bond between the old concrete of the existing structure and the new concrete of the erected anchor blocks. In order to improve those bonds and in order to eliminate negative effect of shrinkage and other factors, the existing contact concrete surfaces have been knocked off as deep as the protective layer and textured. Upon the concreting of anchor blocks and after the required strength has been acquired, the contact surfaces have been injected by low viscosity epoxy resin under pressure, with the ultrasonic control.

After tightening of tendons, injecting by cement grout has been performed in order to provide corrosion protection.

The magnitude of prestressing force has been determined for each structure separately, on the basis of the analysis and the assessment of the condition of the structure and of the existing tendons. Namely, in spite of the detailed examination, it was not possible to precisely establish the existing prestressing force in the tendons, along the whole route. In the conditions in which water penetrates to the cavities and cracks of the concrete section, when the corrosion of the tendons is very high, when on certain places the tendons are completely cut by the corrosion and only a small distance apart they are completely solid, the existing prestressing force, frequently lower then the designed one, significantly changes from section to section. That is why structural analysis has been made for ultimate cases of the lowest and the greatest prestressing force which, for the majority of structures, ranged between 60% and 90% of the designed force. Taking into consideration the most inconvenient combinations of those influences with maximum and minimum influences of loading in the sections, the possible ultimate states of stress have been obtained. According to the bearing capacity of the section and the minimum necessity to provide pressure in the cross sections, the prestressing force which can be safely subsequently introduced has been determined. For the majority of structures, by so determined subsequent prestressing force, the complete live load has been "covered" and about 10% of the dead load. For the structures in which the corrosion of tendons was much higher, the number of additional tendons was larger.

The possibility of subsequent intervention during the rehabilitation, or later on, during exploitation, has been also anticipated. The places for about 25% of additional tendons have been left prepared and can be used in case of the requirement.

In order to improve the traffic conditions on the bridges and in order to acquire protection and to increase durability of the structures, the following has been done beside the already mentioned works: waterproofing, replacement of the pavement and of gullies and expansion joints, repair or replacement of the bearings and of other damaged elements.

All the works have been executed without traffic brake only with its limitation on one side of the bridge. For only two of the structures, the condition of which required additional measures, temporary elastic support has been made during the execution of the works.

The structures have been examined by test loading whereat neither new cracks appeared nor the old injected cracks had opened.

During the stage of decision taking on the execution of the designed bridge rehabilitation works, technical and economic analysis has been made which has proved justifiability of investments to rehabilitation.

In order to achieve a more modern and a better approach to further regular maintenance of these bridges, which are located on a very important traffic route, all the information regarding the rehabilitation measures as well as the condition of all the elements and of the structure as a whole have been introduced to the computer "data bank" which enables regular follow-up of the condition and of the imperi for certain elements and the bridge as a whole, thus enabling due interventions.

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Verstärken von Stahlbetonbauten mit aufgeklebten CFK-Lamellen

Strengthening of Reinforced Concrete Structures Using Bonded Carbon Fibre Sheets

Renforcement d'ouvrages en béton armé avec des lamelles en fibre de carbone

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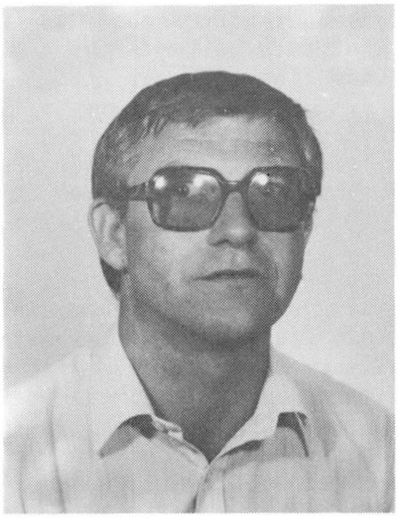

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ZUSAMMENFASSUNG

Es werden die Ergebnisse eines statischen Bruchversuches an einem Stahlbetonbalken mit wirklichkeitsgetreuen Abmessungen beschrieben, der mit einer aufgeklebten Kohlestoff-Faserlamelle (CFK) verstärkt worden ist. Die bisher vorliegenden Ergebnisse ermutigen, den eingeschlagenen Weg weiter zu verfolgen.

SUMMARY

In this paper the results of a static loading test on a reinforced concrete beam are reported. The beam was strengthened by means of externally bonded carbon-fibres. The results obtained in this test encourage carrying out further investigations in the field of this new technique of strengthening.

RÉSUMÉ

Les résultats d'un essai de charge jusqu'à la rupture d'une poutre en béton armé sont présentés. La poutre a été renforcée par une lamelle composée de fibre de carbone collée à l'extérieur de la poutre. Les résultats obtenus encouragent de continuer à expérimenter avec cette nouvelle technique de renforcement.

1. DIE ENTWICKLUNG DER KLEBETECHNIK

Die Möglichkeit, bestehende Stahlbetonbauwerke von aussen her durch Ankleben von Stahllamellen als zusätzliche Bewehrung zu verstärken, hat schon manchem dieser Bauten dazu verholfen, noch für viele weitere Jahre seinen Dienst zur Zufriedenheit seiner Benützer bei gleichzeitiger Einhaltung der geforderten Sicherheit zu versehen. Dabei wirkt sich günstig aus, dass das Verfahren unter Berücksichtigung aller Umstände wirtschaftlich interessant ist. Es kann daher nicht überraschen, dass die seit 1967 [1] angewandte Methode im Laufe der Zeit immer mehr Verbreitung gefunden hat und dabei auch weiter entwickelt worden ist [2]. Hierbei wird heute vor allem auch dem Einsatz neuer Materialien besondere Beachtung geschenkt, da auf diese Weise dem Verfahren weitere Einsatzmöglichkeiten erschlossen werden können. Allerdings wird man dabei sehr sorgfältig vorgehen müssen, um nicht Gefahr zu laufen, einerseits bestehende Mängel zwar zu eliminieren, andererseits aber neue, unbekannte und daher mit grösseren Risiken behaftete Schwächen einzuführen.

Gerade um dieser zuletzt genannten Gefahr zu begegnen, sind schon seit 1985 an der Eidgenössischen Materialprüfungs- und Forschungsanstalt EMPA (Schweiz) Untersuchungen im Gange, welche die Abklärung der Einsatzmöglichkeit von Kohlestoff-Faserlamellen (CFK-Lamellen) anstelle der bisher üblichen Stahllamellen als Zusatzbewehrung zum Gegenstand haben. Aufgrund der bisherigen Erfahrungen darf erwartet werden, dass bei Einhaltung gewisser Regeln bei Bemessung und Ausführung dieser Erweiterung der Methode der Klebebewehrung durchaus reale Chancen zukommen [3]. Bevor aber schon an einen Einsatz in grösserem Stil gedacht werden kann, müssen noch etliche Fragen sowohl bezüglich des Ermüdungs- und des Langzeitverhaltens als auch bezüglich der Ausführung sorgfältig abgeklärt werden.

Im Rahmen dieser Abklärungen wurde kürzlich an der EMPA ein erster statischer Belastungsversuch an einem Balken mit bauwerksähnlichen Abmessungen getestet, um daran das Trag- und Verformungsverhalten unter wirklichkeitsnahen Bedingungen zu untersuchen; über die dabei erzielten Ergebnisse wird nachfolgend berichtet.

2. DER CFK-BELASTUNGSVERSUCH

2.1. Versuchsbedingungen und -vorbereitungen

Für die Versuche stand ein Stahlbeton-Plattenbalken mit einer Gesamtlänge von 8.40 m, einer Höhe von 0.60 m, einer Plattenbreite von 1.25 m und einer Stegbreite von 0.35 m zur Verfügung. Für die statischen Belastungsversuche wurde dieser Versuchsbalken auf zwei Stützen im Abstand von 7.00 m frei drehbar aufgelagert und symmetrisch zur Mitte im Abstand von 2.00 m mit zwei Einzelkräften belastet. Als Hauptbewehrung enthielt der Träger auf der Zugseite vier Bewehrungsstäbe ϕ 28 mm; als Schubbewehrung waren Bügel ϕ 10 mm mit einem mittleren Abstand von 190 mm über die ganze Balkenlänge verteilt angeordnet. Die weiteren Details bezüglich der Bewehrung, Abmessungen, Anordnung der Messinstrumente und der Belastung sind in Fig. 1 angegeben.

Die unterseitige Verstärkung des Balkens durch schubfeste Klebeverbindung erfolgte mit einer CFK-Lamelle der Querschnittsabmessungen $300 \cdot 1$ mm. Vorgabe der statischen Berechnungen war, im Versagensfall nur den Bruch der CFK-Lamelle herbeizuführen und andere Versagensarten für den Balken auszuschliessen.

Vor dem Aufkleben der CFK-Lamelle wurde der Balken, zusätzlich zu seinem Eigengewicht, mit den zwei in Fig. 1 angegebenen Einzellasten stufenweise bis auf 100 kN je Zylinder belastet, um Risse zu erzeugen; dabei traten die ersten Anrisse bei einer Kraft von $F = 30$ kN je Zylinder auf; daraus lässt sich, unter Berücksichtigung des Eigengewichts des Balkens und der Versuchseinrichtung, eine Betonbiegezugfestigkeit von $f_{cb} = 4.5$ N/mm² berechnen.

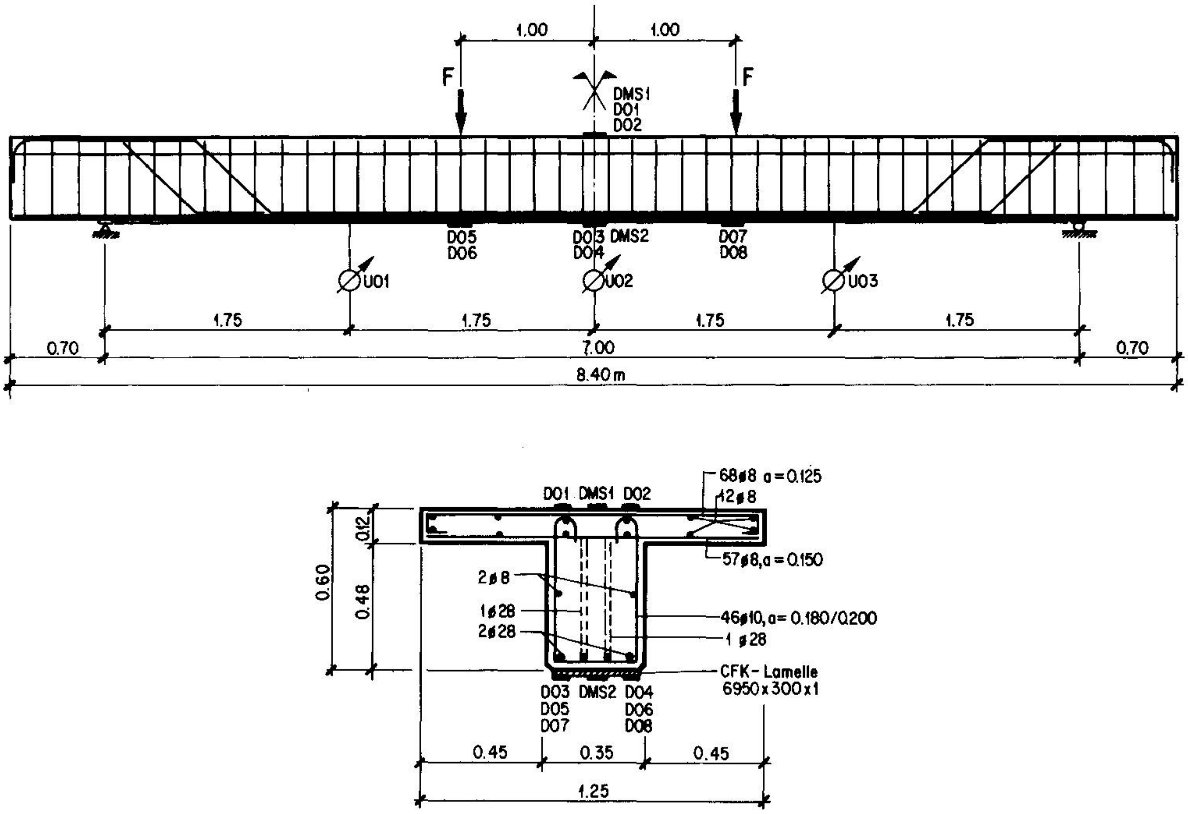

Fig. 1: Versuchsbalken: Abmessungen, Bewehrung, Lastanordnung sowie Art und Lage der verwendeten Messinstrumente

2.2. Materialeigenschaften und -verarbeitung

Der Beton entsprach der Güte B25 nach DIN 1045 mit einem Grösstkorn von 32 mm. Die Zementdosierung betrug 280 kg/m³, wobei ein Portlandzement zum Einsatz kam; der W/Z-Wert wurde zu 0.57 gewählt. Damit ergab sich eine Würfeldruckfestigkeit nach 28 Tagen Abbindezeit von f_{cw28} = 36.6 N/mm². Zum Zeitpunkt der Versuche hatte der Balken ein Alter von 1733 Tagen; die unmittelbar nach dem Versuch an Bohrkernen bestimmte Betonfestigkeit ergab Werte zwischen f_{cw} = 43.4 und 61.8 N/mm², im Mittel von f_{cw} = 49.8 N/mm², die Zugfestigkeit solche zwischen f_{ct} = 2.45 und 4.68 N/mm².

Die Streckgrenze der Stahlbewehrung ø 28 mm (Stahl III) betrug f_{sy} = 464 N/mm², ihre Bruchdehnung (Gleichmassdehnung) A_g = 9.4 %.

Hersteller der CFK-Lamelle mit der Bezeichnung HM 35 war die Akzo, Unternehmensbereich Fasern und Polymere, in Wuppertal (D). Die Lamelle wurde unter Druck und Temperatur im Autoklaven aus insgesamt neun UD-Prepreg Lagen der Einzeldicke 0.12 mm gefertigt und erhielt eine Blieder-Release-Oberflächenstruktur. Das verwendete Epoxidharz härtet bei Temperaturen von 130° C.

Die statische Berechnung des verstärkten Balkens erfolgte mit den folgenden, vom Hersteller angegebenen Werten: E-Modul der Lamelle E_{Lz} = 184'000 N/mm², Zugfestigkeit f_{cfkr} = 1'220 N/mm², Bruchdehnung ε_r = 0.65 %.

Als Haftkleber zum Aufkleben der CFK-Lamelle auf den Beton wurde ein Epoxidharz verwendet, das von CIBA-GEIGY AG in Wehr (D) hergestellt worden war und die Bezeichnung XB 3074 A (Harzkomponente) bzw. XB 3074 B (Härterkomponente) trug. Die an Probeprismen (40x40x160 mm) gemessenen Druckfestigkeiten betrugen im Alter von 7 Tagen f_{ew} = 84.1 N/mm², die Biegezugfestigkeit f_{ebt} = 33.4 N/mm².

Vor dem Aufkleben der CFK-Lamelle wurde der Beton auf der Unterseite des Steges sorgfältig mit einer Nadelpistole aufgerauht und nachher entstaubt, so dass eine innige Verbindung des Klebers mit dem Korngerüst des Betons gewährleistet war. Auch die Lamellenoberfläche wurde mit Schmirgeltuch aufgerauht und unmittelbar vor dem Verkleben mit einem Lösungsmittel entfettet, um auch hier eine gute Haftung mit den Fasern zu erhalten.

Das Aufkleben erfolgte in der von Stahlverklebungen her gewohnten Art und Weise, indem der Kleber auf die Lamelle aufgebracht wurde; dabei lag die Lamelle auf einem Holzbrett. Zusammen mit diesem wurde sie anschliessend von unten her gegen die rauhe Betonfläche gepresst, so dass ein sich auf der frischen Kleberoberfläche im Luftkontakt gebildeter "Film" beim Anpressvorgang unweigerlich aufgerissen werden musste. Damit war ein guter Verbund mit dem Beton gewährleistet.

2.3. Sensor-Messtechnik

An die Überwachung und Kontrolle des Verformungsverhaltens von Tragwerken werden zunehmend höhere Anforderungen gestellt. Das in Zusammenarbeit der Firmen Strabag Bau-AG, Köln (D) sowie Felten und Guilleaume Energietechnik AG, Köln (D) entwickelte Überwachungssystem mit Hilfe von Lichtwellenleiter-(LWL)-Dehnungs-Sensoren [4] wurde während des Versuches für die Demonstration der Leistungsfähigkeit neuer und bewährter LWL unterschiedlicher Empfindlichkeit genutzt.

Die Sensoren wurden in Höhe der inneren Zugbewehrung des Balkens nach Einfräsen einer Nut und durch Kunstharzvermörtelung in den Stegflächen appliziert, nachdem sich dieser schon im Zustand II befand. So konnte die nachfolgende Rissbildung im Mörtel durch Messungen über den gesamten Versuchszeitraum verfolgt werden; es ist vorgesehen, die diesbezüglichen Ergebnisse noch zu veröffentlichen.

3. UNTERSUCHUNGSERGEBNISSE

3.1. Rechnerische Werte

Im unverstärkten Zustand beträgt die rechnerische Bruchlast des Versuchsbalkens unter Berücksichtigung seiner Eigenlast F_{r0} = 236 kN. Wird angenommen, dass die Lamelle bis zum Erreichen ihrer Zugfestigkeit am Beton haften bleibt, dann ergibt sich eine theoretische Bruchlast von F_{rth} = 303 kN, so dass, wenn gegenüber Bruch mit einer Sicherheit von γ = 1.8 gerechnet wird, die Nutz- oder Gebrauchslast F_{adm} ungefähr zu 160 kN eingesetzt werden kann.

3.2. Versuchsablauf

Der Versuchsablauf gliederte sich in drei Phasen: Eine erste Phase betraf die Belastung des unverstärkten Balkens, um ihn in den gerissenen Zustand (Zustand II) überzuführen. Dabei wurde die mittlere Durchbiegung des Balkens erfasst (Fig. 2). In diesem Zustand erfolgte dann nach seiner vollständigen Entlastung das Aufkleben der CFK-Lamelle auf der Balkenunterseite.

Die nächste Phase umfasste sodann die Belastung im Bereich der Nutzlast. Hierzu wurden die beiden Einzelkräfte in Stufen von je 40 kN von F = 0 bis F = 160 kN erhöht. Dabei wurden in Balkenmitte und im Bereich der Lasteinleitungspunkte sowohl die Durchbiegungen als auch die Lamellendehnungen und die Betonstauchungen gemessen.

Nachdem die Nutzlast von F_{adm} = 160 kN je Lasteintragungspunkt erreicht worden war, wurde fünfmal auf eine Grundlast von F = 5 kN entlastet und wieder auf die

Nutzlast belastet, um so das Verhalten des Balkens im Gebrauchszustand verfolgen zu können; bei der letzten Entlastung und Wiederbelastung wurden deshalb sämtliche Verformungsmessungen sowohl bei Oberlast als auch bei F = 80 kN und F = 5 kN ausgeführt.

Die letzte Belastungsphase lag schliesslich im Bereich zwischen Nutz- und Bruchlast. Um auch hier die Verformungsmessungen korrekt ausführen zu können, wurde das Kriechen des Betons kompensiert, indem die Belastung für die Zeit der Messungen abgemindert wurde, so dass sich die Durchbiegung in Balkenmitte während der gesamten Messzeit nicht veränderte.

3.3. Verformungen

Die Fig. 2 und 3 zeigen als Beispiele der Verformungsmessungen den Verlauf der Durchbiegungen sowie die Lamellendehnung bzw. die Betonstauchung in Balkenmitte.

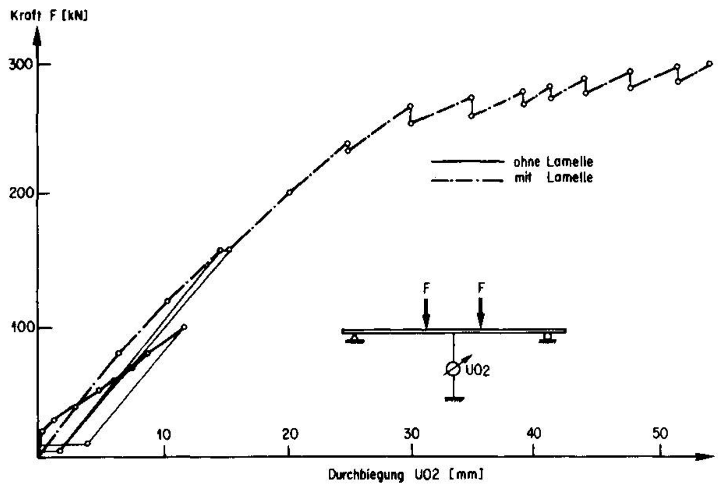

Fig. 2: Verlauf der Durchbiegung in Balkenmitte in Abhängigkeit der aufgebrachten Belastung F.

3.4. Bruchlast und Bruchart

Der Bruch trat bei einer Belastung von F_r = 302 kN je Lasteintragungspunkt auf und lag somit erstaunlich nahe beim im voraus berechneten Wert von F_{rth} = 303kN. Der Bruch selber erfolgte, indem sich die oberste Schicht der Lamelle vom übrigen Teil der Lamelle löste (Trennbruch innerhalb der Lamelle), nachdem vorgängig im Bereich der Lasteintragungspunkte einzelne CFK-Fasern gerissen waren (Zugbruch in den Fasern). Die gemessenen grössten Verformungen in Balkenmitte erreichten Werte von $\delta \approx 52$ mm für die Durchbiegung und $\varepsilon_c = -1.1$ °/oo für die Betonstauchung bzw. $\varepsilon_L = 5.7$ °/oo für die Lamellendehnung.

4. DISKUSSION DER ERGEBNISSE

Bis zur Nutzlast von F = 160 kN konnte ein beinahe linear-elastisches Verformungsverhalten des Balkens festgestellt werden. Dabei war nur eine unbedeutende Zunahme der Biegesteifigkeit des verstärkten Balkens gegenüber dem unverstärkten, aber gerissenen Balken zu beobachten (vgl. Fig. 2).

Bei einer äusseren Belastung von $F_{sy} \approx 240$ kN trat Fliessen der innen liegenden Stahlbewehrung auf, so dass diese bei der weiteren Laststeigerung keinen wesentlichen Beitrag zum Widerstand mehr leisten konnte; die festgestellte Belastungssteigerung bei zunehmenden Verformungen ist somit fast ausschliesslich auf das linear-elastische Verformungsverhalten der CFK-Lamelle bis zum Bruch zurückzuführen. Die Verformungen bildeten sich denn auch bei der Entlastung fast vollständig wieder zurück, wodurch sich dieser Balken gegenüber einem mit einer aufgeklebten Stahllamelle verstärkten wesentlich unterschied [5].

Damit konnte anhand eines ersten Versuchs an einem Bauteil mit wirklichkeitsnahen Abmessungen gezeigt werden, dass auch diese neuartige Technologie für Verstärkungen bestehender Bauwerke in Betracht gezogen werden kann, wobei allerdings dem gegenüber Stahl anderen Verformungsverhalten die notwendige Beachtung geschenkt werden muss. Bis zur Anwendungsreife in bestehenden Bauwerken sind die Bemessungs- und Ausführungsbedingungen in Theorie und Praxis jedoch noch sorgfältig abzusichern und weiter zu entwickeln. Hierbei ermöglichen die modernen Sensortechniken eine permanente Überwachung der Gebrauchstauglichkeit.

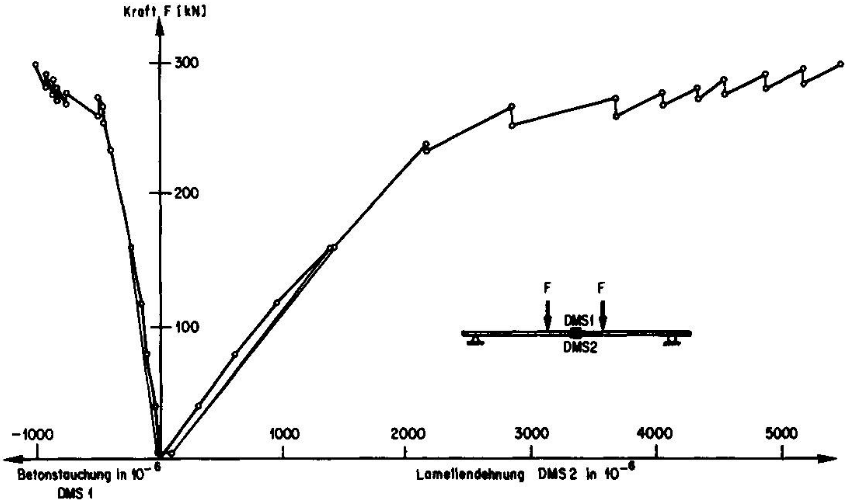

Fig. 3: Verlauf der Lamellendehnung bzw. der Betonstauchung in Balkenmitte in Abhängigkeit der aufgebrachten Belastung F.

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Repair of Fatigue-Cracked Plate Girders in Highway Bridges

Réparation de poutres métalliques de ponts ayant des fissures de fatigue

Reparatur von Blechträgern mit Ermüdungsrissen in Brücken

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SUMMARY

Recent investigations showed the existence of many small cracks at the intersections between main girder and cross beam or bracing of highway bridges in Japan. The bridges comprise reinforced concrete slab and I-shaped plate girders and floor beams. Failure analysis and Finite Element Analysis were performed, then repairing method was studied and executed. The result was incorporated into the new design specification. Vulnerability of the new design specification to fatigue defects was also checked and confirmed to be appropriate.

RÉSUMÉ

Au Japon, dans de nombreux ponts à poutres en acier en I comportant des dalles en béton armé, des fissures ont été remarquées au raccordement entre la poutre principale et la traverse ou l'entretoise. Les causes de ce défaut et les réparations à faire ont été analysés par éléments finis et par des essais de fatigue. Les résultats sont inclus dans une nouvelle norme, qui tient compte des effets de la fatigue.

ZUSAMMENFASSUNG

In vielen japanischen Doppel-T-Träger-Brücken mit Stahlbetonplatten sind Risse am Verbindungsteil zwischen Hauptträgern und Querträgern bzw. Verbänden gefunden worden. Analysen des Schadenzustandes, Berechnungen nach der Methode der finiten Elemente, Ermüdungsprüfungen mit grossformatigen Modellen und Schweissprüfungen wurden durchgeführt. Die Ursachen für das Auftreten der Schäden wurden analysiert, und Reparaturmassnahmen wurden untersucht. Weiter sind auch Untersuchungen nach den gleichen Methoden in bezug auf die Möglichkeit des Auftretens solcher Schäden bei neuen Brückenträgern durchgeführt worden.

1. PREFACE

Plate girder bridges have been of predominant use in the construction of urban highway system in Japan. These are comprised of main girders, cross beams and sway and lateral bracings. And reinforced concrete deck is placed on the top of girders. Recent investigation reveals that small fatigue cracks are apt to be found at welding around connection plate at the intersection between the main girder and the cross beam, or the bracing. Some propagate through web of the girder.

Detail investigation was made to analyze the causes of the fatigue cracks, hence to devise the repairing or strengthening techniques. It was also expected that the results could be applied for revision of the contemporary design specifications.

2. INSPECTION

Hanshin Expressway Public Corporation in Osaka, Japan now provides 138.5 km of urban highway system with elevated type structures, most of them comprises of steel plate girders. 80% of the plate girder bridges were thoroughly inspected, and consequently 20% of them revealed a defects of the above-mentioned type of crack. Failure analysis concluded as follows:

(1) Defects are limited to bridges with reinforced concrete deck served more than 14 years. There is a tendency that the cracks are found for bridges,

-1) designed according to the specification edited in 1964 [1],

-2) with RC slab of span length more than 2.5 m long, and

-3) with relatively shallow slab depth.

(2) Defects can be found for bridges with rigid cross beam, which was intended to secure good load distribution.

(3) Most cracks are classified into Type 1 shown in Fig. 1. Type 2 cracks follows.

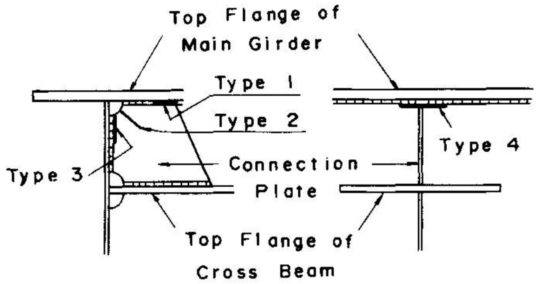
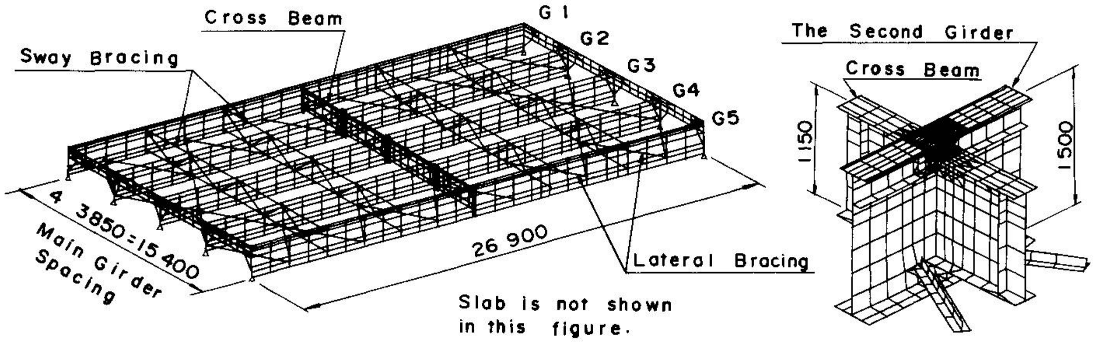


Fig. 1 Types of Fatigue Crack

3. FAILURE ANALYSIS

3.1 Analysis of Stress Distribution

Finite Element Method was used to analyze stress distribution and its value. Phase-1 analysis was made on coarse three dimensional model shown in Fig. 2(a) to asses the global stress flow. A 20-ton-truck load, designated as T-20 loading in design specification [1](Fig. 3), was placed in the longitudinal



(a) Global Model (b) Local Model

Fig. 2 Analysis Model

direction, two in the transverse direction so that those cause the most severe stress distribution at the place of concern. To make a detail investigation of stress distribution, Phase-2 analysis was made on a partial three dimensional model shown in Fig. 2(b) by applying the sectional force given by the Phase-1 analysis.

From Phase-1 analysis, it was concluded there is a close correlation between the equivalent stress at the connection plate and the rotational difference between the slab and the cross beam as shown in Fig. 4.

From Phase-2 analysis it was concluded,

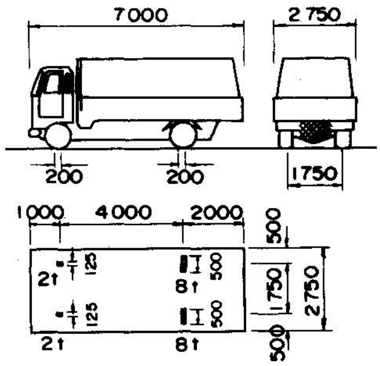


Fig. 3 T-20 Truck Load

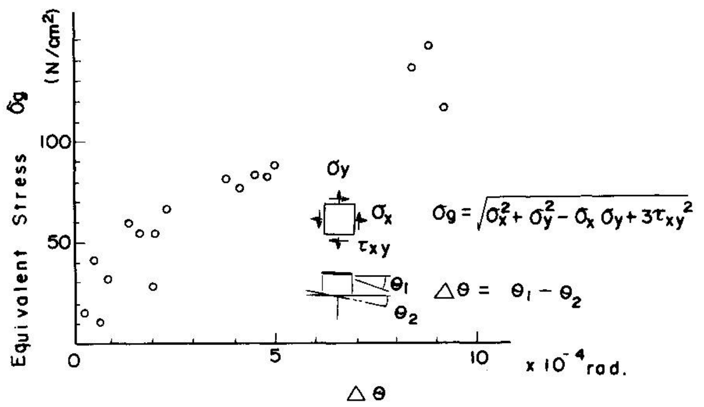


Fig. 4 Relation between σg and $\Delta\theta$

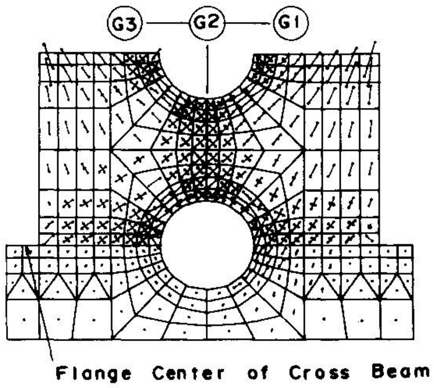


Fig. 5 Principal Stress Diagram

-1) The Principal stress over 200 N/cm^2 is detected on the connection plate adjacent to the free edge of the top flange of the main girder. Due to shearing stress, the principal stress direction vary significantly as it comes closer to the web as shown in Fig. 5.

-2) There is a stress turbulence due to scallop.

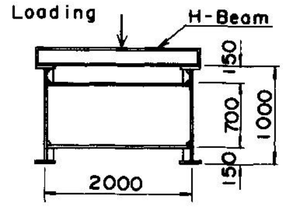
3.2 Stress Measurement at Bridge Site

Stress measurements were executed at the bridge in service to confirm the resutls obtained by the stress analysis. Stress concentration was of a primary concern on the connection plate adjacent to the free edge of the main girder. It was revealed that the tensile stress is 80 N/cm^2 at 20 mm away from the point of interest. The stress is small compared with the analyzed value, about a fourth, but is not small enough as a tensile stress to be ignored for fillet welds.

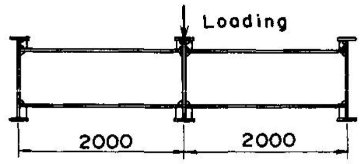
3.3 Fatigue Experiment [2]

It was assumed that the principal causes of the defects are 1) the rotational difference between the slab and cross beam, and 2) the difference of vertical displacement between the adjacent main girders, fatigue experiment was executed in this respect. Two types of the specimen shown in Fig. 6(a) and (b) were tested. The first was a two-girder type. An equivalent vehicle load was applied at the center of H-beam, each end was fixed on the top flange of the girder, with equivalent bending rigidity with RC slab. The second was a three-girder specimen. The test load was applied directly on the interior girder, and was adjusted so that the curvature of the test girder is equivalent with the actual girder when the rear axle load 16 tons of the T-20 loading is applied

on the girder.
It was revealed that the three-girder specimen, which causes only the vertical mutual difference of deflection, caused no cracks after 2 million cycles of load repetition, but on the two-girder specimen, which causes the rotational difference, cracks were induced at the cycles of about 100,000. The three-girder system caused no fatigue cracks even the test was further continued by the load amplitude 1.45 times greater.
The similar experimental results were reported by KATO and three [3].



(a) Two-Girder Specimen



(b) Three-Girder Specimen

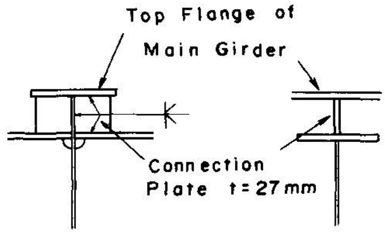
Fig. 6 Fatigue Test Specimen

3.4 Causes of the Crack

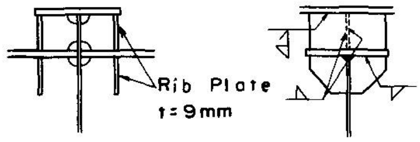
From the experiment, it was concluded that the rotational difference between the slab and the cross beam or the bracing is the main cause of the defects. Other inducements are to be welding defects caused by an excessive root-gap, or a notch of weld bead around a scallop.

4. STUDY OF REPAIRING METHOD

It is assumed that followings are effective for retrofitting the above-mentioned defects,
-1) to lessen the local stress at the welding, and
-2) to lessen the stress at the structural part as a whole.
It is available to exemplify the former one by,
-a) attaching a piece of structural member to increase the cross-section,
-b) increasing the welding size, or
-c) grinding the weld bead to alleviate the stress concentration.
It is not so practical to introduce the latter method 2) because it requires the modification of the structural system.
From the economical stand point, the simpler one 1) is preferred. Fatigue experiment on the method c) showed crack generation at about 30,000 cycles, so the method was concluded not effective at all.
Also the method b) was rejected because it causes large stress to the existing connection plate.



(a) Model A

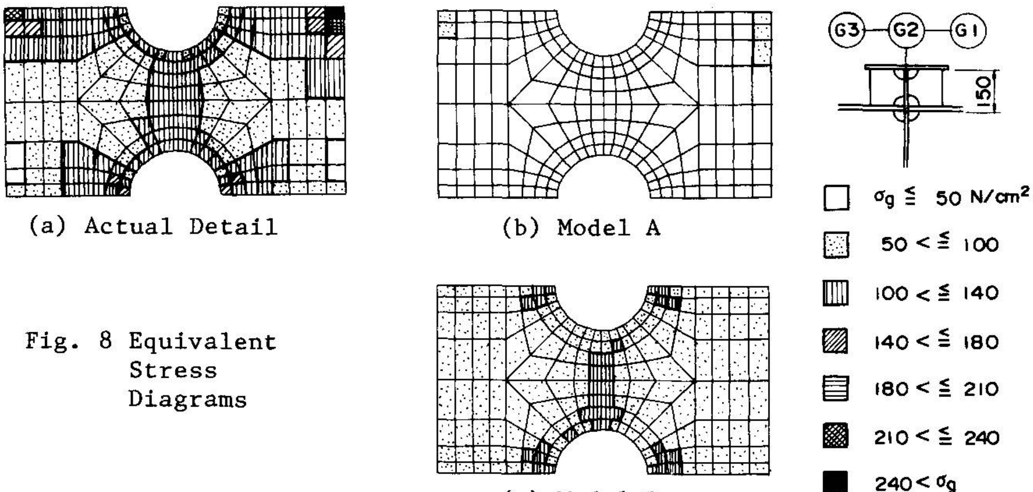


(b) Model B

Fig. 7 Prototypes

4.1 Analysis of The Repairing Prototypes

Two prototypes shown in Fig. 7 are proposed as repairing prototypes. For Model A, Fig. 7(a), the thickness of the connection plate was tripled. For Model B, Fig. 7(b), a rib plate was attached along the edge of the connection plate. Analysis was performed in accordance with the method shown in 3.1. The results are shown in Fig. 8. Model A lessens stress to one third. Model B improves the stress distribution at the free edge of the connection plate, but stress distribution around the scallop remains almost the same.
From these results, Model A was expected as an effective repairing detail and its availability was confirmed by fatigue testing on the tested two-girder type specimen by replacing cracked connection plate with tripled thickness one.



(a) Actual Detail

(b) Model A

(c) Model B

Fig. 8 Equivalent Stress Diagrams

4.2 Trial by Specimen

In the practice of repairing, the influence of heat stress caused by welding have to be taken into consideration besides the stress caused by vehicles. An experiment with one half scale model was performed to confirm the structural behavior and the working condition. An equivalent dead load is applied through the test. No harmful deformation to cause structural instability was detected. But the top flange of the main girder between the studs attached on the flange deformed so as to cause a gap between a slab and the flange. This seems due to the heat effect of full penetration welds, initially intended to secure strength at the intersection.

4.3 Decision of Repairing Method

As a result of the investigation shown above, the repairing method was decided as follows: replace the present 9 mm thick plate with 19 mm one by partial penetration welds. It is expected to secure strength and minimize the harmful influence at field welds.

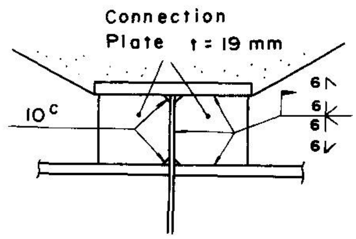


Fig. 9 Repairing Detail

5. FEEDBACK TO DESIGN CODE

Current Specification for Highway Bridges in Japan[4] revised in 1980 specifies narrower girder spacing and thicker slab thickness after the experience of several damage of RC slab. Now, it was to be tested whether the bridges designed according to the revised specification can sustain the vehicle load without showing any fatigue cracks. The same procedure as shown in chapters 3 and 4 was applied.

5.1 Stress Analysis

Structural analysis was made by Finite Element Method by assuming a girder as a beam element, and a cross beam as a plane stress rectangular element. It was confirmed that the stress level is about one third and a favorable behavior can be expected under the new specification.

5.2 Field Measurement of Strain

Field measurement of strains caused by vehicle loads in service was executed on a newly constructed plate girder bridge. The maximum measured stress was 25 N/

cm^2, so the stress in practice is assessed to have been improved to one third.

5.3 Fatigue Test [2]

The similar fatigue test as 3.3 was also performed for the structure designed by applying new specification. No cracks were detected.
A series of the fatigue test was summarized as S-N relation in Fig. 10. The doted line shows the S-N line given by the least square method. The slope is,

$k = 1 / m = 0.185,$

$m = 5.4,$

therefore the stress amplitude for 2,000,000 cycles is about 200 N/cm^2.
Assuming that the bridges showed the defects are estimated to have experienced stress amplitude of 80 N/cm^2 for the past 14 years, we may conclude that bridges designed by new specification can sustain more than $(80/25)^{5.4} \times 14$ years.

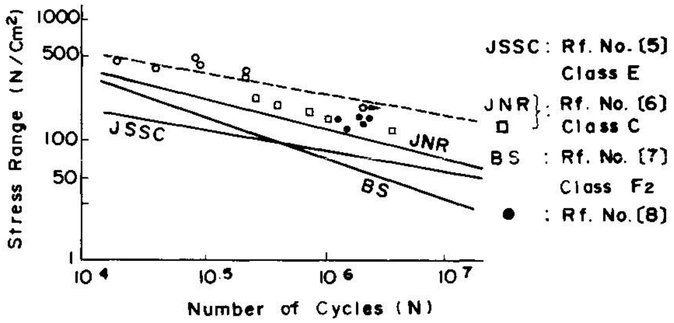


Fig. 10 Fatigue Test Result

5.4 Recommendation for Specification

Slab span and the slab thickness have been improved from the stand point of fatigue. But in view of the increase of excessive truck loads, 12 mm thick connection plate is recommended to be applied in place of 9 mm one. And, no scallops are allowed in this portion, as it can be seen in 3.1.

6. CONCLUSION

From the present investigation, it seems necessary to give more safety margin to some structural details, i.e. connection plate on cross beam, from the point of view of fatigue. The repairing method on the fatigue defected members and the revision of the current specification were recommended. It is essential that the fatigue design is to be introduced for the design of highway bridges.

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Aluminum Curtain Wall Panel Failure, Assessment and Repair

Rupture, évaluation et réparation d'éléments de façade en aluminium

Versagen, Beurteilung und Instandstellung von Aluminium-Fassadenelementen

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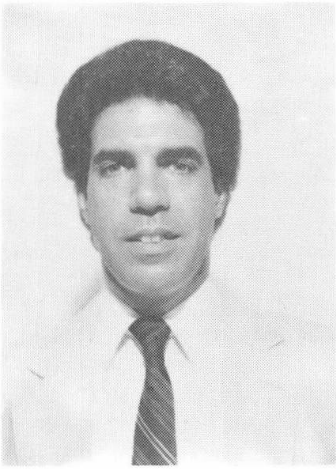

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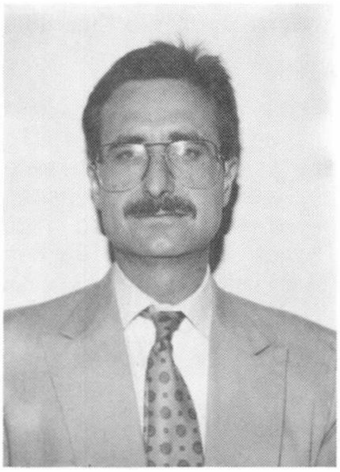

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SUMMARY

This paper summarizes an investigation of the failure of an aluminum curtain wall panel of a righ-rise building in New City. The investigation included a review of the original panel and fastening system design, metallurgical evaluation, field inspections, in-place testing, development of interim repairs, structural analysis, derivation of a cyclic wind loading spectrum, laboratory static and fatigue testing, and fatigue life analysis. A repair and monitoring program was developed which represents a prudent, yet not overly conservative, means of ensuring a very low probability of panel failure in the future.

RÉSUMÉ

Cet article décrit l'examen de la rupture d'une façade en aluminium d'un immeuble à New York. Il comprend un contrôle des éléments originaux et de leur système d'attache, un examen métallurgique, des inspections et essais in situ, le développement de réparations provisoires, des calculs statiques et la détermination d'un spectre de la charge de vent cyclique et de la durée de vie du point de vue statique et dynamique. Un programme de réparation et de surveillance a été mis en place. Il réprésente un moyen sûr, sans être trop conservatif, afin d'éviter la rupture d'autres éléments.

ZUSAMMENFASSUNG

Dieser Beitrag beschreibt die Untersuchung des Versagens einer Aluminiumfassade eines Hochhauses in New York City. Sie umfasste eine Ueberprüfung der ursprünglichen Fassadenelemente, des Befestigungssystems, metallurgische Abklärungen, Felduntersuchungen und- versuche, Entwicklung von provisorischen Reparaturlösungen, Tragwerksnachrechnung, Ermittlung eines zyklischen Windlastspektrums, statische und ermüdungsspezifische Lebensdauer. Ein Reparatur- und Ueberwachungsprogramm wurde entwickelt, welches auf nicht allzu konservative Weise eine geringe Wahrscheinlichkeit für das Versagen weiterer Fassadenelemente gewährleistet.

1. BACKGROUND

During a windstorm in December 1983, an aluminum curtain wall panel separated and fell from one of the upper floors of a high-rise building in New York City. The building was constructed in 1977 and its exterior contains more than 9,000 of these panels.

The authors' firm was retained by the building owner to conduct an investigation in order to determine the cause of the panel separation, determine the magnitude and extent of the possible problem, and report on the general structural condition of the balance of the aluminum curtain wall panels on the building.

2. INVESTIGATION

2.1 Information Reviewed

Since the building had been recently constructed, information pertaining to the design and construction of the aluminum curtain wall panels was readily available. This included the architectural design drawings and specifications, the panel manufacturer's shop fabrication drawings, and the original design calculations relating to the wall panels and fastening system which had been prepared by the panel manufacturer.

A review of the above information indicated that the wall panel and fastening system design was not unusual and in fact was commonly used on numerous buildings at the time.

The typical wall panel assembly consists of a 1.42m x 2.13m x 4.76mm aluminum flat plate with 19 aluminum clips welded to the inside surface of the plate (Fig. 1). These clips attach the plate to the perimeter mullion system and an intermediate horizontal stiffener at the approximate mid-height of the panel. In addition, the top edge of the plate fits into a continuous groove formed into the upper horizontal mullion. Since the building has approximately 9,000 wall panels, there are almost 200,000 clips utilized on the building.

The original wind tunnel test report for the building was also available. This indicated that the design wind pressure in the area of the separated panel was approximately 170 kg/m^2 suction, which was relatively high (but not the highest) as compared to other areas of the building.

2.2 Metallurgical Evaluation

A metallurgical evaluation of the separated panel and its remaining clips was conducted. It was concluded that the fatigue-sensitive nature of the clip-to-panel weld detail, combined with poor welding, resulted in fatigue cracking of panel clip weldments which caused the panel to separate from the building.

In order to provide an initial indication of clip capacity, a panel was removed from the building and selected clips were subjected to static load tests, fatigue load tests, and metallurgical examination. The static load tests indicated clip breaking strengths ranging between 110 kg and 155 kg. The fatigue test results were very erratic due to the poor and variable weld quality and the limited number of samples, and were considered to be inconclusive. Metallurgical examination of several clip welds revealed substantial lack of fusion between the clip weld and the panel surface, as well as fatigue cracks which had initiated from lack of fusion at the weld root.

Therefore, the results of clip testing and metallurgical evaluation of clip-to-panel welds from the removed panel confirmed fatigue of panel clip weldments as the cause of the panel separation.

2.3 Field Inspection and Survey

Concurrently with the metallurgical evaluation, a detailed close-up inspection of accessible aluminum panels was performed from the window washing rig on the outside of the building. The purpose of this inspection was to detect and identify any loose and/or misaligned panels which might be indicative of broken clip welds.

In addition, a field survey of several thousand existing clip welds was performed from the interior of the building. Many of the welds were found to have defects in the form of shrinkage cracks, porosity, lack of fusion, and insufficient weld size. However, only one clip weld was found to have fractured due to fatigue loading under service conditions.

2.4 In-Place Pressure Tests

In order to determine the in-place behavior of the aluminum panels, such as their load-deflection characteristics and their ability to resist design loads, five typical wall panels in the vicinity of the failed panel were selected for in-place pressure testing on the building. Most of the clip welds on the test panels contained shrinkage cracks and were considered representative of the clip welds examined in the field survey.

The accessible lower portion of each of the test panels, i.e., below the intermediate horizontal stiffener, was subjected to an internal pressure (simulating wind suction) of 266 kg/m^2, based upon New York City Building Code requirements, by using a air bag. No failures of clip welds occurred, nor was there any visual evidence of propagation of the pre-existing shrinkage cracks. No permanent deformation of the clips or panels was sustained, and the structural integrity of the tested panels was considered to be the same as prior to the test.

The results of the in-place pressure tests supported the conclusion that the clip failures in the separated panel were the result of fatigue rather than a single overload.

2.5 Interim Repair

Clips that were found to be missing or broken during the field inspection and survey were repaired by fastening either the existing clip or a new replacement clip to the panel using stainless steel screws.

In addition, stainless steel screws were installed along the bottom edge of all panels by drilling and tapping into the panels from the inside of the building. This, along with the existing continuous groove support at the top of the panel, provided basic stability for the panel independent of the clips, and was performed as an advance safety measure while the investigation was proceeding.

2.6 Structural Analysis

An analytical approach was devised using computer-assisted, finite element methods to model a typical aluminum panel and its supporting clips and curtain wall framing, in order to study the behavior of a typical panel under various loading conditions and to determine the corresponding reactions on each aluminum clip.

2.6.1 Mathematical Model

A three-dimensional mathematical model of the panel assembly was developed utilizing one-dimensional beam and two-dimensional shell finite elements. The aluminum plate was discretized into 648 triangular constant stress, linear displacement field, shell elements. The aluminum clips were modeled using one-dimensional rigid links. These rigid links connected the plate elements to the mullions and intermediate horizontal stiffener. The mullions and stiffener were modeled using one-dimensional beam elements with assigned material and cross-sectional properties equal to the actual properties of the members.

2.6.2 Loading Conditions

Three loading conditions were applied to the mathematical model. First, a uniform suction load of 197 kg/m^2 was applied to all plate elements. This represented the original panel design load.

Second, a temperature differential of 22.2°C between the plate and the mullions was applied, based upon in-place temperature measurements on the building.

Third, in order to calibrate the finite element model with the actual panel load-deflection characteristics, a uniform test load of 266 kg/m^2 was applied to all plate elements below the horizontal stiffener. This loading case simulated the in-place pressure tests performed on the accessible lower portions of five panels on the building.

2.6.3 Results of Analysis

A stiffness analysis was carried out for each of the imposed loadings using the TPS-10 program on a VAX 11/750 computer. Nodal displacements, shell stresses, beam forces and moments, and reactions were calculated. Clip reactions were obtained as axial forces on the rigid links.

For the original design load case of 197 kg/m^2, a maximum clip reaction of 56.7 kg was obtained for the center clip at the bottom edge of the panel. A maximum plate stress of 2.9 kg/mm^2 and a maximum out-of-plane displacement of 21.3mm were obtained at approximately the mid-point between the bottom edge of the panel and the intermediate horizontal stiffener.

All clip reactions and plate stresses for the temperature differential load case were found to be negligible.

2.7 Derivation of Wind Loading Spectrum

Based upon data contained in the original wind tunnel test report for the building, a cyclic wind loading spectrum was derived. This loading spectrum was utilized in the analysis of panel clip fatigue life.

2.8 Laboratory Testing

In order to determine the expected fatigue life of existing panel clips, constant-amplitude cyclic testing of numerous panel clips was performed. These panel clips consisted of existing clips removed from the building, and also new clips fabricated with "imperfect" weldments simulating the existing weldments in the building. The test results were plotted to obtain a "load vs. cycles-to-failure" curve for the clips (Fig. 2).

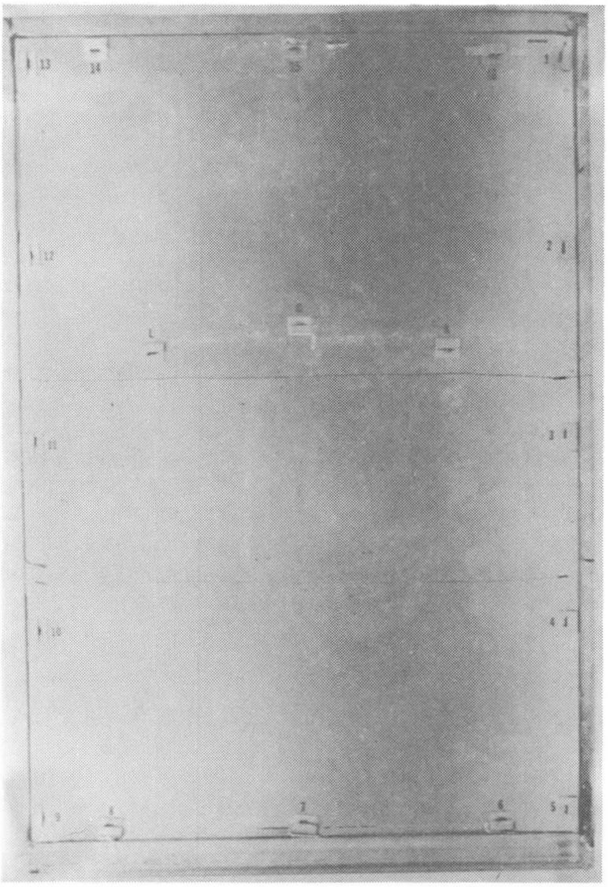

FIG. 1 Typical wall panel assembly

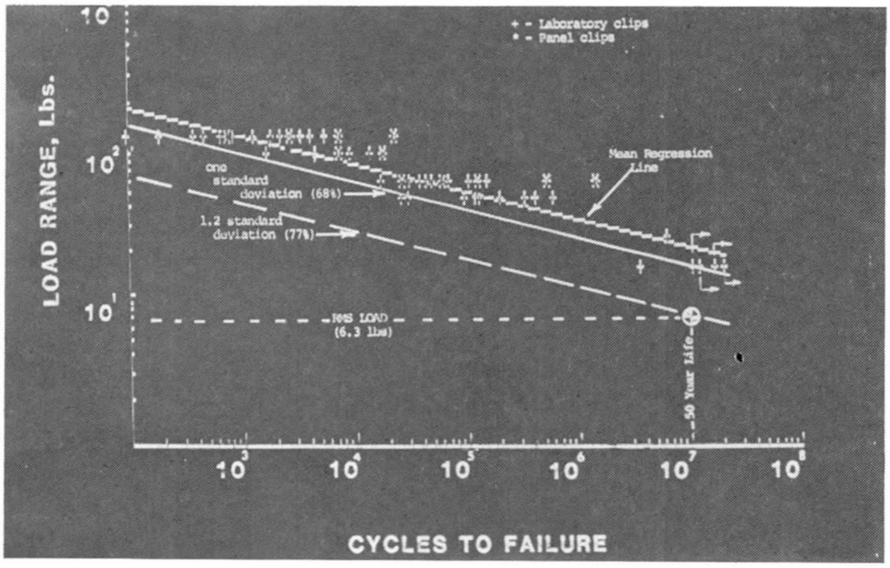

FIG. 2 Load vs. cycles to failure

In addition, laboratory static and fatigue testing of aluminum and stainless steel rivets and screws was performed in order to evaluate potential remedial fastening details.

2.9 Fatigue Life Analysis

Based upon the cyclic wind loading spectrum that was derived, a root-mean-square (rms) wind loading of 10.4 kg/m^2 at 10 million cycles, corresponding to a 50-year life, was computed. This translates to an rms load of 2.9 kg at 10 million cycles on the most highly loaded clip. Fig. 2 indicates that there is a 77% confidence level that a maximum-loaded clip weld will survive 50 years.

3. REPAIR

In addition to the repair of missing and broken clips, and the installation of stainless steel screws along the bottom edge of each panel, a periodic monitoring program was implemented in order to permit timely detection and repair of failed clips. The monitoring procedure consists of in-place testing of selected clips from the window washing rig on the exterior of the building, using a special load-deflection device.

3.1 Results of Periodic Monitoring Program

The first monitoring procedure was conducted on all panels of the building from November 1986 to October 1987. Only 46 clips out of more than 68,000 tested, i.e., less than 0.07%, were found to be missing or broken and were repaired.

A second monitoring procedure for a limited number of selected panels, to be conducted within a 2-week period, is in progress at the time of this writing. The time intervals and extent of clip testing for subsequent procedures will be determined based upon an evaluation of the results of prior procedures.

4. CONCLUSIONS

The results of this investigation indicated that the aluminum curtain wall facade structure is fundamentally sound with a confidence level of 77%. However, due to the variability of the cyclic wind loading spectrum derived from the wind tunnel data analysis, and the inherent scatter in laboratory fatigue testing results, there can be no absolute assurance (i.e., 99% confidence level or greater) that some panel clip welds may not fail in the future. Therefore, in addition to interim repairs, a periodic monitoring program was implemented in order to permit timely detection and repair of failed clips.

It is considered that the repair and monitoring program outlined in this paper represents a prudent, yet not overly conservative, means of ensuring a very low probability of panel failure in the future.

Repair of the Bascule Pier of the Oddesund Bridge

Réparation de la pile du pont mobile de Oddesund

Instandstellung des Klappfeilers der Oddesundbrücke

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SUMMARY

Throughout its fifty-year life the bascule pier of the Oddesund Bridge has suffered both from alkali-silica reaction in the concrete and from a slanting pile foundation. These primary defects have in the course of the years resulted in movements and settlements. Attempts have been made to repair these primary defects and the resulting defects in various ways. However, these makeshift solutions have not fully been successful. After a final analysis in 1986 it was decided in 1987 that a total renovation of the bascule pier pile foundation and pier itself should be carried out. The field work was started in March, 1989.

RÉSUMÉ

La pile du pont de Oddesund souffre, depuis 50 ans qu'il existe, de réactions alcalino-siliceuses du béton et d'une médiocre fondation sur pieux. Ces défauts primaires se sont manifestés sous forme de déplacements et de tassements. A intervalle réguliers il a été tenté d'y remédier, mais sans succès. A la suite d'une étude approfondie il a été décidé de rénover entièrement la pile, y compris la fondation sur pieux. Les travaux ont commencé en mars 1989.

ZUSAMMENFASSUNG

Der Klappfeiler der Oddesundbrücke hat Zeit seines Bestehens, d. h. seit 50 Jahren teils unter Alkali-Kiesel-Reaktionen im Beton, und teils unter einer schiefen Pfeilerfundation gelitten. Diese Primärschäden haben im Laufe der Jahre Bewegungen und Senkungen zur Folge gehabt. In regelmässigen Abständen hat man versucht, die Primärschäden sowie die Folgeschäden auf verschiedene Weise zu beheben. Diese Ausbesserungen haben jedoch das Problem der Klappfeiler nicht vollauf gelöst. Nach einer gründlichen Analyse des Schadenumfanges und der Ursachenkette wurde beschlossen, den Klappfeiler total zu renovieren, d. h. sowohl die Pfeilerfundation als auch den Pfeiler selbst. Die Bauarbeiten begannen im März 1989.

1. THE ODDESUND BRIDGE

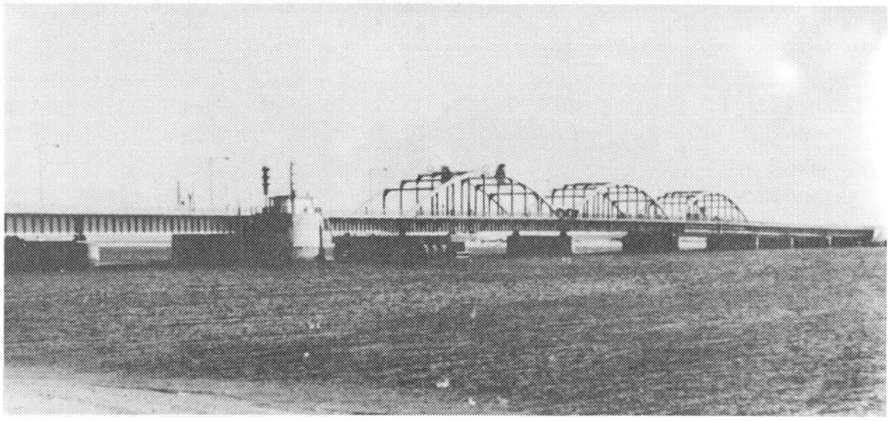


Fig. 1

The combined road and railway bridge over Oddesund was built in the period 1934-37. Fig. 1.

Its capacity for traffic is still considered sufficient, but with regard to road traffic the bridge is exposed to maximum utilization in the peak load period in the summer holiday season.

The navigation conditions seen in relation to the ships that pass the bridge are also acceptable.

Already shortly after the Oddesund bridge had been finished in 1937, settlements and cracks as well as deformations were found in the bascule pier (Fig. 2). Since then the bascule pier has been subjected to continued examinations, repairs and reinforcements. Likewise, it was the main object in connection with the work of the Danish Alkali-Silica Committee in the 1950's and the 1960's.

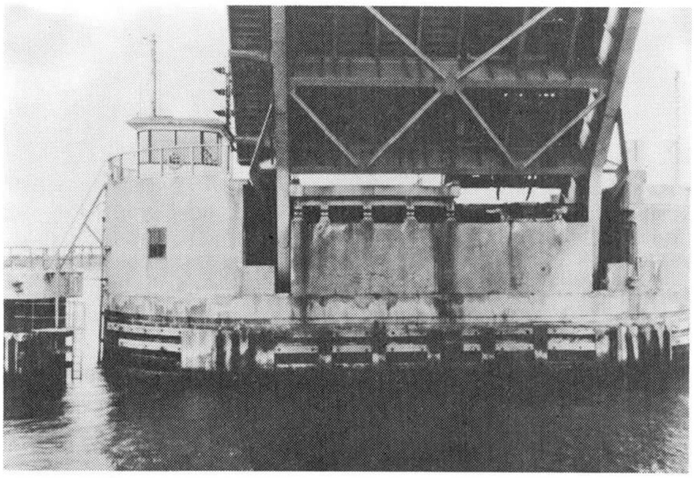
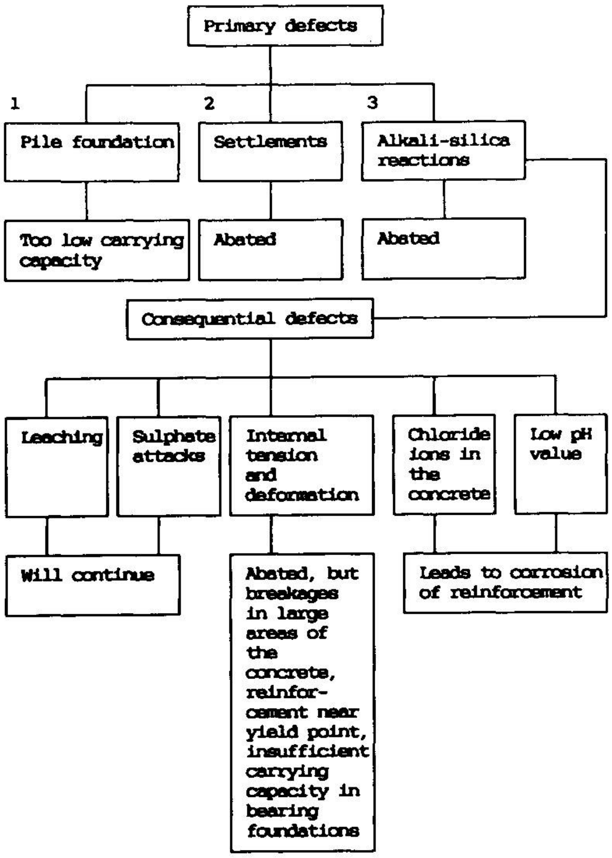


Fig. 2

2. SPECIAL EXAMINATION

In 1986 the Bridge Maintenance Department of DSB (Danish State Railways) carried out a special examination of the bascule pier in co-operation with Rambøll & Hannemann, Consulting Engineers A/S.

Registration of the condition of the pier was carried out as well as evaluation of its carrying capacity and structure, and the history of the defects was clarified. The history of the defects appears from the outline below.



The defects can be divided into three "primary defects":

1. The carrying capacity of the pile foundation is too low, especially with regard to the ability to absorb impacts from ships and the load of ice.

2. The pier has settled towards the channel and towards the west owing to different foundation conditions and different pile point levels, but the settlements have now abated, and the consequential damage has been repaired.

3. The concrete of the bascule pier structure is heavily affected by alkali-silica reactions resulting in expansion and differential expansions. As a consequence of this the concrete has become severely cracked, and changes in the original geometrical form of the bascule pier have occurred.

   Both the total expansion and the differential expansions have now abated.

As a consequence of the alkali-silica reactions three types of "consequential defects" have occurred:

3.1.a and 3.1.b

Through the concrete which is cracked all through, an entrance has been opened for sea water, which has leached the concrete and brought in sulphates.

These attacks will continue to disintegrate the concrete.

3.2.a and 3.2.b

Through the cracked concrete the sea water has brought in chloride ions in dangerous quantities, and the leaching has reduced the pH value of the concrete, which has led to/will lead to corrosion attacks on the reinforcement.

3.3

The expansion due to alkali-silica reactions has had the effects that the reinforcement is at or near the yield point everywhere, and that in large areas the concrete is in danger of breaking under compressive as well as tensile stresses.

The concrete in the bearing plinths is so seriously disintegrated that the carrying capacity of this structure is considered insufficient.

The carrying capacity of the bascule pier structure is sufficient with the exception of the abovementioned bearing plinths, but estimated on the basis of an elastical consideration the carrying capacity is too small. The elastic tension gives rise to large areas of breaks and cracks in the structure, and this in combination with the continued disintegration of the concrete and corrosion of the reinforcement implies that its durability cannot be secured.

The pile foundation of the bascule pier has had too low carrying capacity right from the beginning. For net weight alone the carrying capacity is approximately 85 % of the required value, and for ice load and impact from ships it is as low as approximately 35 % of the required value.

From the point of view: How can the bascule pier be renovated, or can it be renovated at all? - was it specially important to classify the observed movements of the bascule pier according to their nature as either settlements or total and differential expansions.

This was important in order to see whether these movements would continue or not, and in order to evaluate the strain in the very structure.

The differential expansions explain the cause of the many cracks, local breakages and deformations of the bascule pier.

The final report on the special examination dealt with four different possibilities of solving the problem:

1. Interim renovation (will not secure the carrying capacity of the pile foundation).
2. Total renovation.
3. New bascule pier.
4. Establishing of a passage span at the other end of the bridge and change of the existing passage span into a fixed span.

On the background of the report on the special examination the DSB decided in the autumn of 1987 to go in for a total renovation.

3. PROJECT PROPOSAL

3.1 Preliminary Conditions

- The bascule pier will be strengthened so much that its carrying capacity is sufficient according to the current load regulations and standards.

. It is the aim of the project that the total renovation shall have a life of 60 years, corresponding to the estimated remaining life of the rest of the bridge. This is achieved by:

  - making sure that the carrying capacity of the newly built structures and the entire structure will last 60 years.

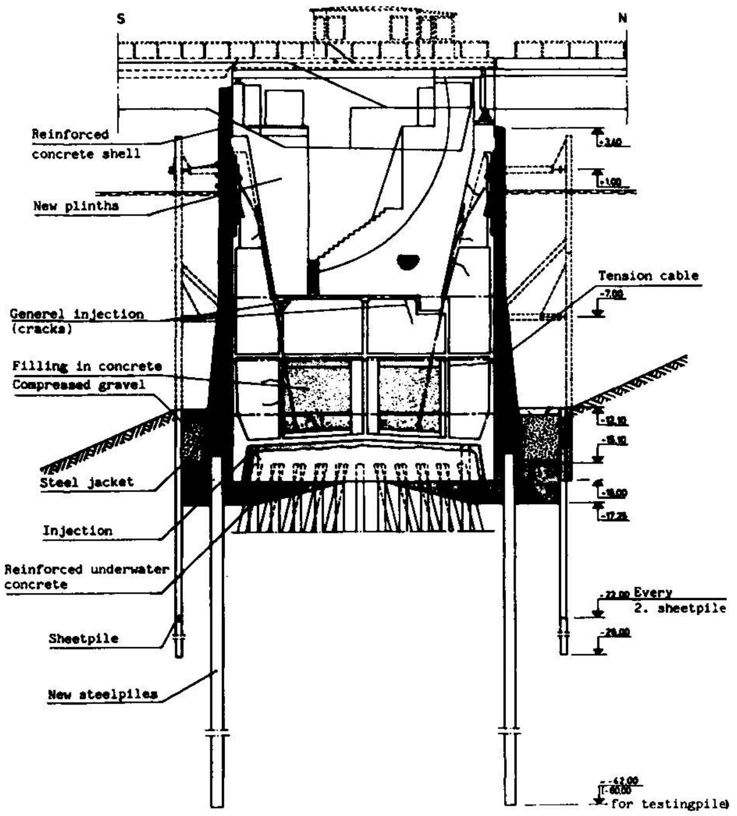
  - repairing the existing structures to such an extent as circumstances require, while at the same time precautions are taken to ensure that it will be possible later on to carry out replacements/repair in such a way that the desired life of 60 years is achieved.

. Impediments to operation (impediments to railway traffic, road traffic and navigation) in connection with the renovation will be minimized to the greatest possible extent.

. The risk in connection with work on the badly damaged pier will be minimized.

. The construction period will be spread over 3 years from 1989.

3.2 Description of the project



CROSS SECTION

The renovation project comprises an external strengthening of the pier in the form of a reinforced concrete shell around the pier, founded on new piles and connected with the old structure by means of transverse anchors. External the concrete shell provides with a steel jacket. Internal repair and tightening are made by injection. New bascule bearing plinths are concreted.

The work is carried out in a dry building pit behind a provisional steel sheet piling, which is driven into the ground around the pier. Traditional working methods can then be applied for the carrying out of uncovering, hewing, renovation of coarse cracks, establishing of transverse anchors, displacement locks and welding of steel jacket etc.

4. POSTSCRIPT

When the special examination of the bascule pier of the Oddesund Bridge is seen in relation to the main damage, which is alkali-silica reactions, it shows

that we have a fairly good knowledge of:

- How the concrete should be composed in order to prevent the occurrence of alkali-silica reactions.
- The chemical reactions that occur in connection with alkali-silica reactions.
- The types of external environment that promote/start alkali-silica reactions.
- Testing methods for the evaluation of the alkali-silica reactivity of aggregates.

that we have too little knowledge of:

- The temporal dependence of alkali-silica reactions (total expansion and differential expansion) on:
  - The external environment.
  - The composition of the concrete.
  - The particle size of the reactive particles in the aggregates.
  - Reinforcement percentage.
  - The geometry of the structure.
- The correlation between alkali-silica expansion in concrete in laboratory experiments (tests) and in concrete in the structures (in "nature").
- The strength of concrete that has been damaged by alkali-silica reactions:
  - Reinforced concrete.
  - Non-reinforced (plain) concrete.
- Repair methods.
- The optimal time for repair seen on the background of the fact that the temporal dependence of alkali-silica expansion is principally an S-shaped curve, where the expansion begins and ends slowly, but is violent in an intermediate period.

On this basis the DSB count on implementing an extended examination of several years' duration in order to clarify the lack of knowledge in this field.

The renovation has started on site March 1, 1989.

Repair of Chloride-Contaminated Concrete Walls in a Tunnel

Réparation de murs en béton de tunnels endommagés par des chlorides

Sanierung von chlorid-geschädigten Betonwänden in einem Strassentunnel

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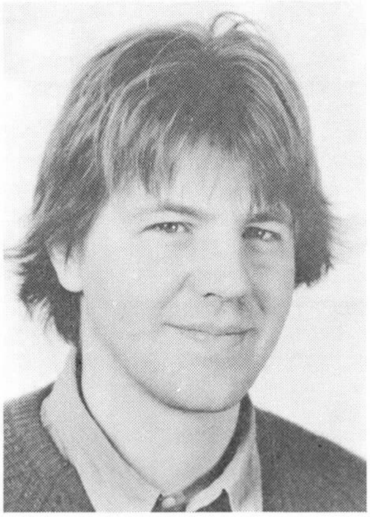

Ole Viggo Andersen, born 1958, obtained his civil engineering degree at the Danish Engineering Academy. For three years he was involved in the design of structures for a Danish contractor. For five years he has been responsible for condition surveys on concrete structures and development of new techniques to maintain and repair concrete structures.

Jens Vejlby Thomsen, born 1940, graduated from the Danish Engineering Academy in 1968 as a civil engineer. Experience as site engineer and project leader for construction companies. Since 1980, senior engineer in the Bridge Department of the Danish Road Directorate, responsible for operation, maintenance and repair of tunnels and major bridges.

Egon A. Sørensen, born 1946, graduated from the Technical University of Denmark in 1970 as a civil engineer. From 1971 to 1987 he worked for Christiani & Nielsen A/S and since 1987 he has been working for COMAR Engineers A/S. His main tasks have been design of bridges and tunnels and the maintenance of tunnels.

SUMMARY

Many concrete structures, especially in countries where de-icing salts are used, are suffering from chloride contamination. The consequences are durability problems and increasing maintenance and repair costs. In order to solve some of these problems the Danish Road Directorate is testing remedial measures on concrete walls in a road tunnel. The measures include replacement of deteriorated concrete by new concrete, cathodic protection by a conducting coating and chloride extraction. The object of the tests is to help in choosing the most suitable time and the most appropriate method for carrying out repair work, as well as to determine the required extent of such repairs.

RÉSUMÉ

Beaucoup de structures en béton subissent des dommages dûs à des chlorides, plus particulièrement dans les pays où sont utilisés les sels de déverglaçage. Les conséquences en sont des problèmes de durabilité et des coûts croissants d'entretien et de réparation. Afin de résoudre ces problèmes, différentes méthodes d'assainissement des murs en béton de tunnels routiers sont examinées. Ceci comprend le remplacement du béton endommagé, une protection cathodique par une surface conductrice et l'extraction des chlorides. Ces recherches permettent de fixer le meilleur moment, la méthode adéquate et l'envergure des travaux d'entretien.

ZUSAMMENFASSUNG

In Ländern mit Tausalzverwendung erleiden viele Betonbauwerke Chlorid-Schäden. Die Folge davon sind Probleme mit der Gebrauchstauglichkeit sowie Unterhaltungs- und Sanierungskosten. Zur Lösung dieser Probleme wurden Methoden zur Sanierung von Betonwänden in Strassentunnels untersucht. Dies beinhaltet den Ersatz des geschädigten Betons, kathodischen Schutz mittels einer leitenden Oberfläche und Chloridextraktion. Die Versuche sollen den zuständigen Behörden ermöglichen, den geeigneten Zeitpunkt, die beste Methode und den Umfang von Sanierungsarbeiten festzulegen.

1. INTRODUCTION

The tunnel is situated in the northern suburbs of Copenhagen, and leads the southbound road traffic on Bernstorffsvej under and into Lyngbyvej - the motorway which is the main road link between Copenhagen and Elsinore.

The traffic is heavy during the morning rush-hours, and in winter de-icing salts are often used. The winter climate in Denmark is characterized by repeated frost and thaw periods.

The tunnel was built in 1969 and is about 180 m long and 11 m wide, including two 1.75 m wide emergency and service footways and a 7.5 m wide dual traffic lane for one-way traffic. The reinforced concrete cast-in-situ walls are 0.5 m thick, shown in Fig. 1.

The tunnel is administered by the Road Directorate and during periodic inspections it was found that the lower parts of the walls showed signs of reinforcement corrosion.

Therefore COWAR Engineers A/S as general technical consultants on maintenance of structures and the Technological Institute as special consultants for investigation and testing of concrete were asked to make an investigation of the state of the concrete and the reinforcement in the spring of 1988.

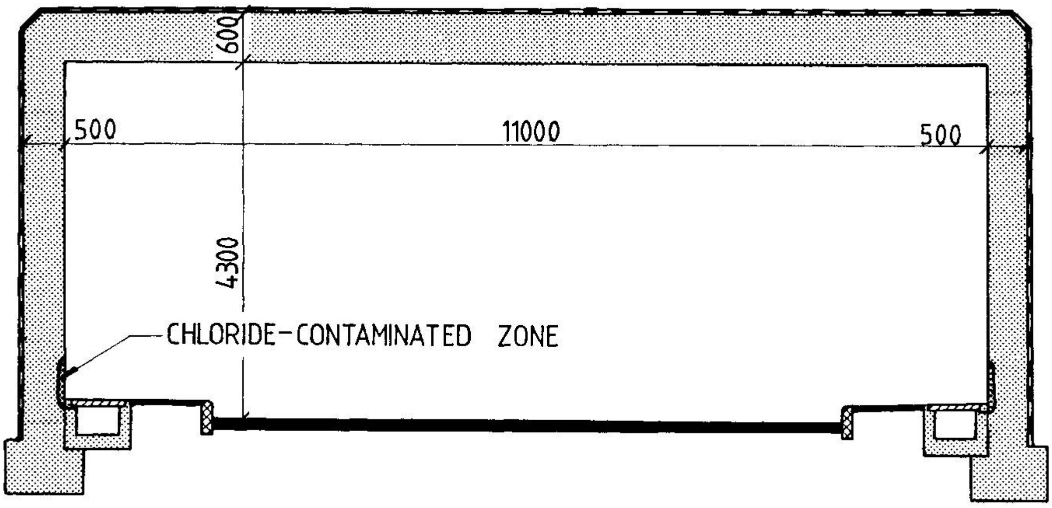

Fig. 1 Cross Section of Tunnel

2. RESULTS OF CONDITION SURVEY

The condition survey is an attempt to combine a visual assessment with appropriate on-site measurements and laboratory tests.

The visual assessment of the walls indicated cracking of medium width in approximately 25% of the tunnel in the lower parts of the walls. Distinct rust stains were visible on the concrete surface in approximately 40% of the lower parts of the walls.

On-site measurements were limited to a half-cell potential survey of the lower parts of the walls. The concrete cover was measured and parts of the reinforcement were exposed to visual examination. The half-cell potentials measured with a copper-coppersulphate electrode indicate a high risk of corrosion of the reinforcement, limited to the lower parts of the wall. Visual examinations of exposed reinforcement at these locations revealed severe pittings, while there were found no signs of corrosion on reinforcement located higher than 0.5 m above ground level in the tunnel. The concrete cover is 25-30 mm. The variation of the chloride content was measured on powder samples from more than 100 locations.

Ten concrete cores were sampled for laboratory tests. The locations of the cores were determined by the analysis of the half-cell potential survey. Tests of the cores included visual examination, petrographic analysis on thin sections, determination of the chloride and moisture contents and distribution: The concrete is mixed with a water-cement ratio between 0.45 and 0.50. Carbonation depth is approximately 15 mm (half the depth of the concrete cover). The sand fraction contains alkali-silica reactive aggregates. Severe alkali-silica reactions were observed locally in the lower parts of the wall, where the chloride content is very high throughout the length of the tunnel. Typical chloride profiles are shown in Figs. 2 and 3. The distribution of chloride content indicates that the chloride is permeating from outside sources - probably from de-icing salts. The moisture content is medium - but high enough to generate corrosion and alkali-silica reactions.

On the basis of these observations it was concluded that severe corrosion of the reinforcement located in the walls 0 to 0.5 m above ground level is caused by an excess of chloride in the concrete.

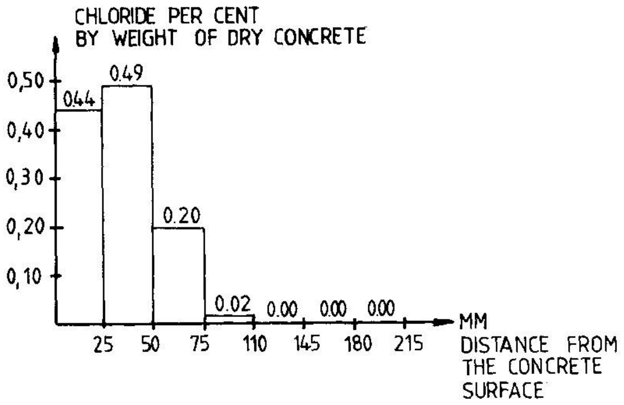

Fig. 2 Typical Chloride Profile at Ground Level

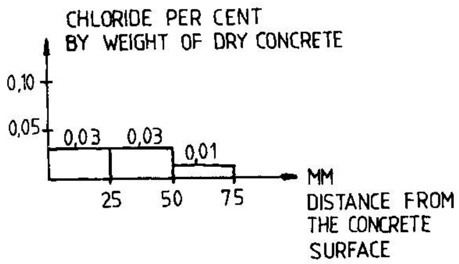

Fig. 3 Typical Chloride Profile approx. 0.7 m above Ground Level

3. SELECTION OF REPAIR METHOD

Before repair work commenced, possible alternative methods were considered. These were:

- Replacing of deteriorated concrete by new concrete
- Cathodic protection
- Chloride extraction

The traditional method involves removal of chloride-contaminated concrete, cleaning of reinforcement bars and, finally, placing of fresh chloride-free concrete. The disadvantage of this method is that the original homogeneity of the structure will not be fully re-established. Further, if all chloride-contaminated concrete has not been removed there is a risk that new corrosion of the reinforcement will start at the casting joint between new and old concrete.

Cathodic protection systems for reinforced concrete have been developed during the past 30 years. Today several systems are available, including:

- Mesh anodes placed in protective concrete layer
- Conductive coating anode systems

The mesh anodes system was left out of consideration for architectural reasons as the mesh should be protected by a 30-50 mm thick layer of concrete.

For the actual job, the conductive coating anode system seemed to be the most attractive. It is easy to apply and there are no dimensional limitations.

For the successful use of a cathodic protection system several conditions have to be fulfilled:

- Corrosion of the reinforcement should not be so advanced that the strength of the structure is reduced to an unacceptable level
- Concrete deterioration, such as spalling, coarse cracks and lamination should not be more advanced than could be repaired prior to installation of the cathodic protection system
- All reinforcement should be in mutual electrical connection

These conditions were fulfilled in the actual case.

The disadvantages of cathodic protection systems are their maintenance requirements and the limited lifetime.

The method of chloride extraction is fairly new and only little experience is available. The system consists of a steel mesh embedded in a fibre mass with an electrolyte. The system is connected to a rectifier, positive to the mesh and negative to the reinforcement. By applying a sufficient DC current the chloride ions should migrate from the concrete into the fibre electrolyte.

Chloride extraction systems should only be used if the above-mentioned conditions for the use of cathodic protection are fulfilled.

On the basis of these considerations, it was decided to make a trial installation of both cathodic protection and chloride extraction systems, while traditional replacing of chloride-contaminated concrete was used for the major part of the tunnel walls. A short section was left untreated as a control.

After the repair work had been carried out the walls were coated with paint to avoid new chloride contamination of the concrete.

4. EXECUTION OF THE REPAIR WORK

4.1 Replacement of Chloride-Contaminated Concrete

The criterion chosen for the repair work was that all concrete with a chloride content of more than 0.05% by weight of dry concrete should be replaced by new concrete. Based upon half-cell potential survey and chloride test results the extent of repair was determined. Typically, the concrete was removed up to 0.4-0.7 m above the road surface. In depth a minimum of approx. 100 mm concrete was removed in order to expose the reinforcement. At that depth the chloride content was well below the 0.05% level.

Most of the concrete was removed by a pneumatic machine hammer and only the last part was removed by hand tools.

Cleaning of the reinforcement was mainly done by sand blasting but, as expected, it was found to be very difficult to remove all chlorides from grooves in the reinforcement. Normally the sand blasting was done twice and the last traces of chlorides were removed by grinding. Supplementary reinforcement was placed near bars which had been unacceptably reduced by corrosion or by the grinding. In order not to overstress the wall it was also necessary to do the repair work in two steps.

After cleaning of the reinforcement, formwork was installed and new concrete cast. Alternatively, the Contractor could have placed new concrete by guniting.

4.2 Cathodic Protection

A cathodic protection system was installed on the lowest 1.0 m of the wall on a 50 m long section. The system consists of a platinum metal anode wire placed along the wall in the lower zone, where the highest chloride content was measured. The wire was embedded in a conductive paste and a conductive coating was then applied to the wall. The conductive coating (black) was overcoated with a white decorative topcoat. The cathodic protection system is powered by a computer-controlled rectifier. The purpose of the conductive paste is to provide a more uniform current distribution from the anode into the conductive coating. The conductive paste and coating are carbon filled waterbased polymer products.

Prior to the installation of the cathodic protection system the tunnel wall was cleaned by sand blasting to remove existing paint, dirt, etc. Then all metallic projections were treated so that they would be electrically isolated from the conductive coating and all major cracks in the concrete were sealed to prevent a short circuit between the reinforcement and the conductive coating through the crack.

In order to ensure the best possible results the work was performed under close quality control. This involved checking of preparation of the wall and temperature and humidity monitoring. Adhesion and conductivity of the coating was measured and finally a thermographic survey was performed to locate possible shorts which had not been discovered during work execution.

After starting operation of the system a new half-cell potential survey was carried out in order to adjust the current density so that on the one hand sufficient protection is obtained and on the other hand overprotection is avoided.

In order to permit continuous monitoring of the cathodic protection system and ambient conditions, reference electrodes and humidity and temperature sensors were installed in the tunnel wall.

4.3 Chloride Extraction

A system for chloride extraction was installed on the lowest 1.0 m of the wall on a 50 m long section.

The system consists of a 100 mm mesh of 5 mm dia. wire which is fixed at a distance of 20 mm from the wall by means of insulated inserts.

The mesh is then embedded in an approx. 50 mm thick layer of fibre mass with a $Ca(OH)_2$ electrolyte. The wire mesh is connected to the positive pole of a rectifier and the reinforcement in the wall to the negative pole.

Before installation of the system, the wall was cleaned of existing paint, dirt etc. by water jetting. In order to avoid short circuits major cracks were sealed and projecting steel parts were insulated.

The current is adjusted to a density of max. 1 A/m^2 concrete surface. The fibre mass is kept wet by daily spraying with water. When a decrease of current density is observed the spraying is done by $Ca(OH)_2$ to maintain the electrolyte. After a period of 8 weeks, the mesh and the electrolyte were removed and the wall was cleaned by water jetting.
It was specified that the chloride content should be reduced to a max. of 0.03% by weight of dry concrete. This was achieved for the major part of the wall, but at some locations no reduction at all was measured, probably because of lamination of the concrete, not discovered during the preliminary investigations.

4.4 Untreated Control Area

In order to be able to evaluate the three different repair methods described above, a 10 m long section was left untreated apart from the fact that it received the same final coating as the remaining parts of the walls.

5. EVALUATION

5.1 Problems for future Investigations

Replacement of chloride-contaminated concrete involves two major problems:Is the corrosion of the reinforcement stopped and what is the quality of the casting joint? The quality of the casting joints was analysed on cores and no entrapped air, cracks or lack of adhesion were found. The corrosion activity in the reinforcement will be monitored by half-cell potential measurements and visual evaluation of corrosion on the reinforcement.

The cathodic protection system is continuously supervised and maintained. The maintenance programme includes:

- Observations with permanent probes measuring temperature, humidity and half-cell potentials
- Observations with external probes: temperature and half-cell potentials
- Visual observations on the installations and anode

A test programme is planned to supplement the maintenance program with this information:

- Chemical reactions
- Visual evaluation of corrosion on the reinforcement
- Moisture movements
- Adhesion strength and durability of the paint and
- Variations of the concrete properties

to be measured after three years.

Chloride has been extracted with a high current density for a period of 2 months. Migration speed of the chloride ions towards the anode will vary with concrete properties, cracks, porosities, etc. The chloride content and distribution will be measured in concrete cores. The high current density may accelerate alkali-silica reactions. Petrographical analyses on thin sections from concrete cores will reveal such reactions. The long-time effect on the corrosion activity on the reinforcement is monitored by half-cell potential measurements.

A detailed condition survey of the untreated control area after three years will enable the effect of the three methods described above and the speed of the deterioration of the concrete and reinforcement to be evaluated.

5.2 Economic Aspects

As the actual costs for the repair work previously described include expenses for both development and investigations, they do not represent the real costs for the execution of similar works in the future.

To obtain a realistic cost comparison we have estimated the costs of repairing 350 m^2 of tunnel wall by each of the three methods.

- Traditional replacing of chloride-contaminated concrete incl. subsequent coating:	3400 DKK/m^2
- Cathodic protection:	700 DKK/m^2
- Chloride extraction incl. subsequent coating:	900 DKK/m^2

In addition to the above-mentioned costs, approx. 20,000 DKK/year must be added for operation and maintenance of the cathodic protection system.

Study and Design of Spherical Gasometers

Etude et projet de gazomètres sphériques

Untersuchung und Bemessung von Kugelgasbehältern

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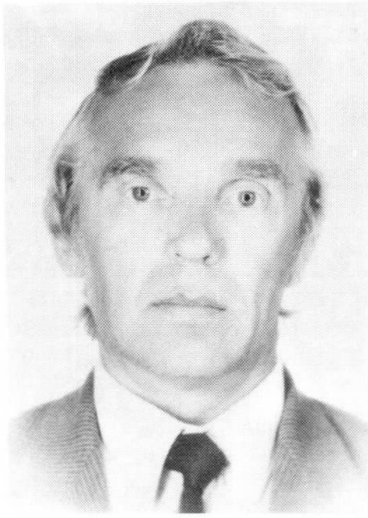
V. V. Larionov, born 1938. Graduated from the Moscow Institute of Aviation Technology. Chief Engineer leading scientist in the field of steel structures cyclic durability.



E. M. Basko, born 1940. Graduated from the Moscow Building Institute. Active in investigations on the field of brittle fracture- and crack-resistance of metal structures.

V. V. Evdokimov, born 1949. Graduated from the Moscow Institute of Aviation Technology. His main field of activity is cyclic durability of structural members.

SUMMARY

The results of experimental investigations of two pilot spherical gasometers carried out with the aim of determining the actual life of similar structures are considered. The mechanism of fatigue crack initiation and development in the zones of stress concentration was found on the basis of which the criteria which apply for the limit states of the gasometers were determined.

RÉSUMÉ

La communication présente les résultats d'études expérimentales sur deux gazomètres sphériques, réalisées afin de déterminer la durée de vie réelle de structures analogues. On a défini la loi d'amorçage et de propagation des fissures de fatigue dans des zones de concentration des contraintes, dont on a tenu compte lors de la détermination de l'état-limite des gazomètres.

ZUSAMMENFASSUNG

Es werden Ergebnisse komplexer experimenteller Untersuchungen an zwei kugelförmigen Gasbehältern vorgestellt, die zur Bestimmung der tatsächlichen Lebensdauer dieser Art von konstruktionen durchgeführt wurden. Es werden Gesetzmässigkeiten der Dauerrissbildung und -entwicklung in Bereichen der konstruktiven und technologischen Spannungskonzentration dargestellt, die bei der Bestimmung der Grenzzustandskriterien der Gasbehälter berücksichtigt wurden.

INTRODUCTION

Ball gas-holders of a volume up to 2000 m^3, made of low-alloy steel of 09Г2С mark, with a shell 17÷36 mm thick and an internal pressure about 0.5÷1.2 MPa are widely used in various industrial fields. A repeated-statical character of loading as well as low temperature climatic conditions influence alongside with structural stress concentrators and such inner welds imperfections as lack of penetration, pore chains, slag inclusions, etc. may contribute, under certain conditions, to premature failure of these structures. Their useful life may be determined on the basis of complex experimental-and-theoretical investigations, including:

- an experimental investigation of stress-strain conditions in the zones of stress concentrators (such as zones of supporting straps and branch pipes reinforcement connections with the gas-holder shell) with regard for its kinetics in the process of cyclic loading;
- full-scale hydrostatic tests of pilot ball gas-holders under static and repeatedly-static loading conditions with ultrasonic control at inner defects development in field butt joints with the successive revealing of all zones of possible failure of the structure shell;
- analysis of the physical and mechanical characteristics at static and cyclic loading, including the crack resistance properties of steel and welded joints of the gas-holder;
- analysis of the work loading spectrum of the conventional gas-holders;
- fatigue life and crack resistance analysis.

Investigations according to the above-mentioned programme were carried out on ball gas-holders of 2000 m^3 volume, with shell thickness 36 mm, fabricated of steel of 09Г2С mark.

1. GAS-HOLDERS FULL-SCALE TESTING PROCEDURE

Experimental investigations were carried out on two gas-holders loaded by internal excessive pressure created by water pouring; loading conditions are as follows: repeated-static loading with frequency 15÷20 cycles per hour and an asymmetry $\rho \approx 0.1$; maximum pressure at the loading cycle was P_{max} = 1.6 MPa and that 30% exceeded the admissible level of operating loading.

At the first stage of testing, using a method of strain measurement, a character of stresses distribution in the zones of supporting straps and branch pipes connections with the shell in the initial and the subsequent loading cycles (up to the 100th) was investigated. For this purpose chains of transverse-and-longitudinal metal-film 2 mm-base resistance strain gauges were used. In order to measure the level of residual welding stresses in the above-mentioned zones, by a method of a local surface off-loading, 0.5 mm-base resistance strain gauges were used.

It was found out as a result of strain measurement investigations that the maximum level of stress concentration didn't exceed the value of $K_\sigma^* = 3.4$, which is characteristic for the cases of an abrupt transition of a fillet weld to base metal and shallow undercuts (up to 0.5 mm) along the bead of the weld. In the course of the structure testing, repeated elastic-plastic deformations took place in the local zones of the structure, which range of values practically didn't change (a condition of the material rigid loading).

Maximum level of residual welding stresses in the above-mentioned zones of the structure before its loading σ_{w_0} didn't exceed $0.65\sigma_{0,2}$, where $\sigma_{0,2}$ is the shell material yield point. However, during the first loading cycles a considerable reduction of σ_{w_0} took place with the subsequent stabilization at the level $\sigma_w = 0.25 \div 0.5\sigma_{0,2}$, depending on the level of stress concentration, K_σ^*, as well as a nominal stress in the shell.

Before the excessive pressure loading of the gas-holders a random ultrasonic non-destructive inspection of field butt joints was carried out. As a result, some continuous technological defects such as pours chains, slag inclusions, lack of weld root penetration were detected, the sizes of these defects exceeded the admissible level specified in the technical requirements for the structures fabrication.

Having all above-mentioned in mind, fatigue cracks were expected to develop at full-scale cyclic tests in the zones of structural stress concentration and in the zones of welded butt joints where continuous technological effects were observed, that's why a periodic ultrasonic inspection of these zones was performed. When a through crack developed in the shell, which could be discovered from the appearing a slight water spurt, the gas-holder testing was stopped, the water drained and a sealing cover plate 3 mm thick was placed from the inner side of the damaged place of the shell and welded with a fillet weld along the perimeter. Then the gas-holder was refilled with water and cyclic testing continued up to the moment of through cracks development in other zones of the shell. Simultaneously, the growth of through cracks in the zone of sealing cover plates (their length allowed the development of fatigue cracks up to the critical level) was measured. Special displacement gauges were used for determining stress intensity factors in the tip of the developing cracks.

In such a way, the reliable results were obtained for the zones of the most probable failure of the gas-holder shell and the characteristics of the cyclic crack resistance of the structural material were estimated.

2. DISCUSSION OF THE RESULTS

The full-scale test results showed that the zones of the most probable failure of the ball gas-holder are, first of all, butt welds with long inner defects such as pour chains, slag inclusions, lack of penetration, etc. As a rule, these welds were performed by a manual arc method in a vertical or overhead position. Initially, at the cyclic loading of the structure the process of merging of separate closely adjoining defects into one of a considerable length took place, further this defect developed up to the moment of the shell depressurization. The four observed cases of the gas-holder shell failure in the zone of butt welds allowed to find out that at a through crack formation its dimensions on the inner surface of the gas-holder shell are about 90÷130 mm, and at the outside surface - about 15÷20 mm, and this is likely to be characteristic of the given type of the stressed state, the shell thickness and welding defects which are near to the inner surface of the shell.

The other zones of the possible gas-holders failure are the zones of junction of the bottom reinforcement (the inner ring strap) in the zone of the gas inlet pipe and the supporting straps on the gas-holder shell. All cracks initiated near the bead of the fillet weld. This type of the failure is of a multi-nucleative character. For example, in one case a through crack initiated in the shell in the zone of its junction to the inner ring strap. Its length at the inner surface of the gas-holder along the fillet weld bead was 150 mm. Besides, 5 surface cracks in the immediate vicinity of each other were found along the perimeter of this junction, one of the cracks, as it was discovered after examination of the fatigue fracture, was nearly half of the thickness of the gas-holder shell. It should be noted that a multi-nucleative initiation of cracks in the zones of structural stress concentration of ball gas-holders with their subsequent merging in the process of the structure cyclic loading, may lead, under certain conditions, to the shell fracture and depressurization and, as a result, to even more serious consequences.

Having in mind all above-mentioned and with regard for the found out mechanism of fatigue cracks in a gas-holder shell initiation and development, the following marginal state criteria were accepted:

- development of a through fatigue crack in the zone of butt welded joints;
- development of a long surface crack in the zones of junction of branch pipes reinforcement and supporting straps with the gas-holder shell.

The full-scale testing results showed that the shell maintained its load-carrying capacity at the presence of through cracks in butt welds and long surface cracks in the zones of structural stress concentration until the temperature of testing was not lower than +10C and at nominal stresses 30% exceeding the maximum working stresses. To find out the critical crack dimensions margins and to make account of the lowered up to -40C operating temperature, the experimental investigation of characteristics of a crack-resistant steel and welded joints on metal, cut of the gas-holder shell after completion of endurance tests were performed. Static cracking tests were performed at axial tension of large-scale specimens of a real thickness with through cracks and surface long cracks in the temperature range from +20C to -100C.

Experimental results for the specimens with surface cracks are given in the fracture graphs, Fig. 1, as the relationships used for the evaluation of the surface crack, in the gas-holder shell, critical depth with regard for the maximum operational stresses and stress concentration influence. The results of the strength analysis, taking into account the experimentally found characteristics of the static crack resistance for the near-weld zone of the welded joint at minimum operating temperature $T = -40C$ showed that at development of long surface fatigue cracks ~ 4 mm deep in the gas-holder shell the load-carrying capacity margin at max. working pressure 1.2 MPa was $n_\sigma = 1.75$ and the critical crack depth margin was $n_a = 2.0$.

The design evaluation of the fracture resistance of the gas-holder shell with a through crack in the butt weld was carried out using a critical stress intensity factor K_c as a strength criterion for the weld metal. The above-mentioned margins of the critical stress intensity factors and crack lengths are ensured at max. working internal pressure up to 1,2 MPa and climate temperatures up to -25C. At lowering the temperature to -40C the working pressure in the gas-holder should also be lowered to 0.8 MPa in order to ensure the corresponding crack resistance margins.

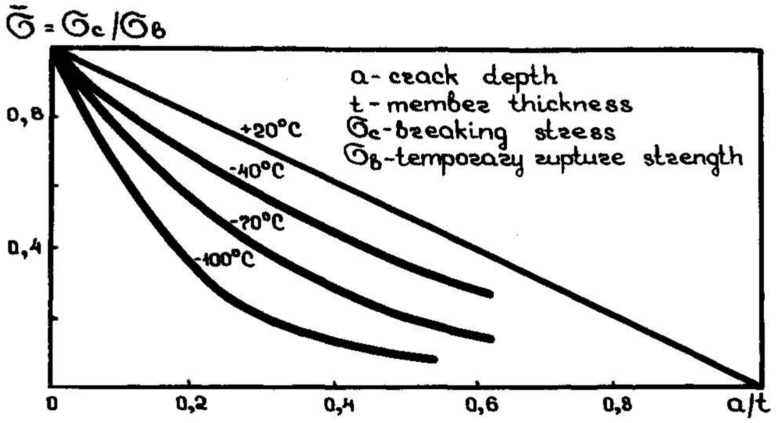


Fig. 1 Fracture curves for the specimens with surface cracks in the near-weld zone of the welded joint

The results obtained were used at the evaluation of the gas-holder useful life, considering cracks dimensions in the shell.

3. THE USEFUL LIFE EVALUATION

The useful life evaluation was performed with regard for 2 criteria of the structure limiting state:

- development of long surface cracks approximately 4 mm deep in the zones of structural stress concentration, which guarantee the absence of the brittle fracture of the shell at minimum operating temperature T = -40C;
- depressurization of the shell as a result of fatigue cracks development initiated from large inner imperfections in welded joints.

At the evaluation of the useful life based on the criterion of surface cracks development the design fatigue curve found on the basis of the experimental data was used, which connected the amplitude of local stresses σ_a^* in the zones of structural and the admissible number of stresses [N]; the safety factors for stresses n_σ and durability n_N being equal to 1.25 and 3.0, respectively (Fig. 2a).

At the determination of local stresses the highest level of structural stress concentration $K_\sigma^* = 3.4$ was considered. The range of operational loading corresponded to the real one for the group of gas-holders with capacity 2000 m^3.

As a result of calculations made with a regard for a rule of linear summation of fatigue damages, it was found out that the useful life of gas-holders according to the criteria of cracks of a critical size development in the zones of a structural stress concentration was 44 years.

At calculation of the useful life according to the criteria of a shell depressurization the fatigue curve was used (Fig. 2b), which characterized the relationship of the admissible number of the loading cycles to the range of nominal stresses in the structure $\Delta\sigma$ and corresponding to the lower envelope of the experimental data according to the moment of through cracks development in ball gas-holders, as well as models of welded joints with the similar weld imperfections at the above-mentioned safety factors according to stresses and durability.

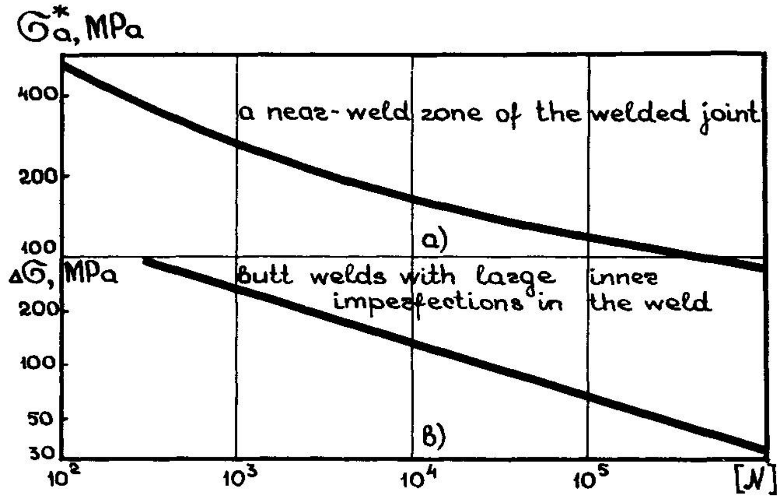


Fig. 2 Fatigue curves for the evaluation of the useful life: a) according to the criterion of surface cracks development, b) according to the criterion of the shell depressurization

The calculation of the useful life was also carried out on the basis of the rule of linear summation of damages without taking into account those amplitudes of nominal stresses at which fatigue cracks had no growth; the largest imperfection found at a random ultrasonic non-destructive testing in one of the gas-holders (lack of penetration of 450x8 mm size) being taken as an initial imperfection. As a result, it was found out that the useful life of a battery of ball gas-holders according to the criterion of the shell depressurization caused by fatigue cracks development from long inner defects in butt field welded joints was 34 years.

In such a way, the factor limiting the structure useful life was the presence of long inner defects in butt welded joints, the elimination of these defects would increase the durability of ball gas-holders up to 44 years.