SESSION III

Construction, Performance, Renewal

Construction, performance, rénovation

Bau, Bewährung, Erneuerung

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Brücken – Bauverfahren (Beton und Stahl)

Bridges – Construction techniques and methods (concrete and steel)

Ponts – Procédés de construction (béton et acier)

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ZUSAMMENFASSUNG
Das Ziel eines guten Bauverfahrens sollte sein, eine Brücke mit möglichst niedrigen Baukosten bei grösstmöglicher Sicherheit im Bauzustand zu errichten. Da die Baukosten im allgemeinen über die Realisierung einer Konstruktion entscheiden, kann das Bauverfahren dominante Bedeutung gewinnen und im Zusammenhang mit einem verringerten Arbeitsaufwand die alte Priorität des „geringsten Materialverbrauchs" zweitrangig werden lassen.

SUMMARY
The object of a good construction method should be to build a bridge in the safest possible way for the least possible expenditure. As in general the construction costs determine the cost of a structure, the construction methods used can become of importance and, together with a reduction in labour costs, can make the old priority "lowest material expenditure" of secondary importance.

RESUME
Le but d'une bonne méthode de construction devrait être de construire un pont dans des conditions de sécurité maximum pour des frais de construction minimum. Etant donné qu'en général, les frais de construction sont décisifs pour la réalisation d'une construction, la méthode de construction peut prendre une importance capitale: l'ancienne condition "réduction au minimum des quantités de matériaux nécessaires" peut devenir d'importance secondaire.

1. BEDEUTUNG DER BAUVERFAHREN

Wurde in der Vergangenheit die Konstruktion einer Brücke vor allem für das Tragverhalten im endgültigen Zustand ausgelegt, gewinnt zunehmend der Einfluss der Bauzustände auf die Konstruktion an Bedeutung. Häufig treten die Grenzbelastungen bereits im Bauzustand auf und in Kombination mit den Hilfsmethoden zur Errichtung des Bauwerkes liegt deshalb die größte Einsturzgefahr in der Bauzeit. Zwangsläufig beeinflussen die Bauverfahren die Entwicklungen im Brückenbau darum nachhaltig. Das Ziel eines guten Bauverfahrens sollte sein, eine Brücke mit möglichst niedrigen Baukosten bei größtmöglicher Sicherheit im Bauzustand zu errichten. Da die Baukosten im allgemeinen über die Realisierung einer Konstruktion entscheiden, kann das Bauverfahren dominante Bedeutung gewinnen und im Zusammenhang mit einem verringerten Arbeitsaufwand die alte Priorität des "geringsten Materialverbrauchs" zweitrangig werden lassen.
Ein etwa notwendig gewordener Mehraufwand an Material kann für die Langzeit-Standfestigkeit eines Bauwerkes häufig durchaus sinnvoll eingebracht werden, und kann gegebenenfalls die Anfälligkeit für Reparaturen vermindern. Darüber hinaus sollten Reparaturmöglichkeiten konstruktiv so berücksichtigt werden, daß der Verkehr auf der Brücke im Reparaturfall gar nicht oder möglichst wenig eingeschränkt werden muß.

Rund ein halbes Jahrhundert ist der Start des Stahlbrückenbaus dem Stahlbetonbau voraus, und rd. 150 Jahre mußten vergehen bis die vorgespannten Stahleinlagen den Betonbrückenbau entscheidend neu befruchteten. Der "zugfest gemachte" Beton kam in seinen Eigenschaften dem Werkstoff Stahl näher und es ist deshalb nicht verwunderlich, daß der voranschreitende Stahlbau zunächst auf die Betonkonstruktionen und später auch auf ihre Herstellungsverfahren fördernd ausstrahlte und noch ausstrahlt.
Der Gewichtsvorteil und die einfachere Beherrschung des Materials in Statik und Konstruktion gaben jedoch dem Stahlbrückenbau bei der Eroberung der zunehmend größeren Spannweiten weiterhin den Vorzug. Die starke Konkurrenz des Spannbetons hat aber rückwirkend einen großen Zwang zur Rationalisierung des Stahlbrückenbaus ausgelöst, der zunächst vor allem die Konstruktion, dann aber zunehmend die Bauverfahren beeinflußte, erkennend, daß Konstruktion und Bauverfahren eng miteinander verknüpft sind. Eine optimale Abstimmung dieser beiden Komponenten aufeinander ist Voraussetzung für den wirtschaftlichen Ablauf eines Brückenbaus ganz allgemein und besonders eines Groß-Brückenbaus.

2. WICHTIGE GRUNDSÄTZE ZUM STAND DER TECHNIK

2.1 ... im Stahlbrückenbau

Die rationelle Fertigung geht davon aus, möglichst "fertigungseinfache" Brückenquerschnitte für durchgehende, gleichbleibende Balkenträger herzustellen unabhängig davon, wie und in welchem Abstand der Balken gestützt oder aufgehängt wird. Eine geringe Zahl solcher "Brückenlängssysteme" ist in der Lage, den für den Stahlbrückenbau interessantesten Bereich von etwa 100 m Spannweite aufwärts bis zur heute etwa erkennbaren praktischen Grenze von rd. 3000 m abzudecken. [1] Dies bedingt eine konstruktive Entkopplung der Trag- und Fertigungssysteme. Hierfür zeigen sich die Balken-, (Vielseil)-Schrägseil- und die Hängebrücken aufgeschlossen, während sich Tragsysteme mit örtlich konzentrierten Krafteinleitungen, wie Rahmen-, Zügel- oder unechte Hängebrücken schlecht oder gar nicht entkoppeln lassen. Ebenso schließen sich Bogen und in gewissem Umfang sehr weit gespannte Balken von diesem Rationalisierungstrend aus. So profitiert in starkem Maße das System der Schrägseilbrücke (Fig. 1) von dieser Entwicklung, die in der Spannweite zwischen Balken und Hängebrücken einzuordnen ist und sich dort "breit" macht, indem sie die wirtschaftliche Spannweitengrenze der Balken herunterdrückt und die der Hängebrücken heraufschiebt. Für die Trägerlängssysteme hat sich der Vollwandhauptträger fast ausnahmslos gegenüber dem Fachwerk, selbst bei den weitgespannten Hängebrücken, durchgesetzt (Fig. 2).

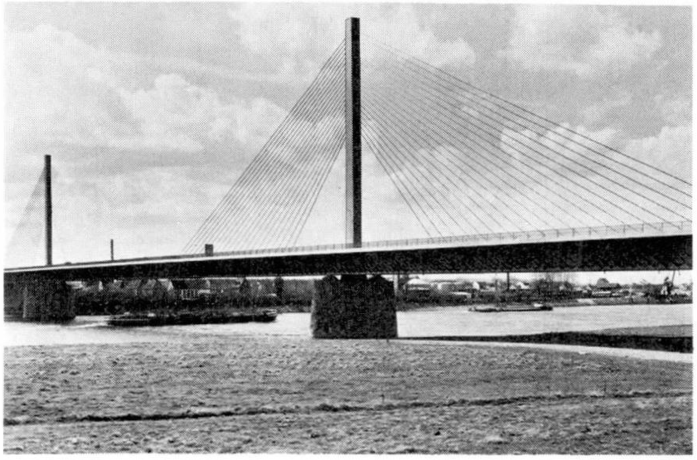

Fig. 1 Rheinbrücke Bonn-Nord

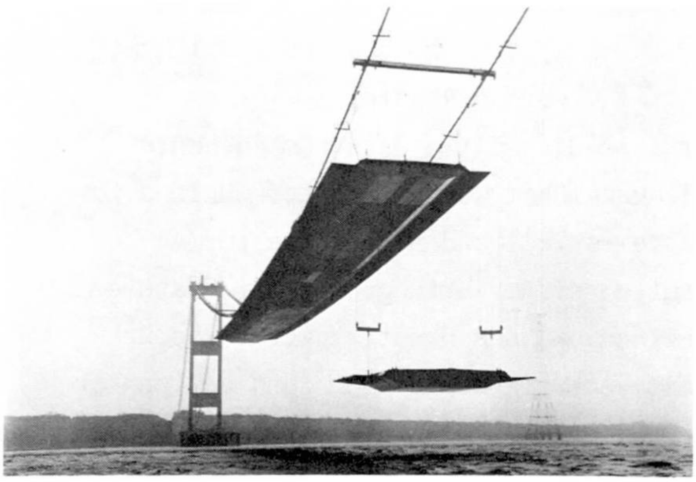

Fig. 2 Severnbrücke

Wesentliche, zum Teil voneinander unabhängige Gründe dafür sind:

- Technologische Fortschritte der rohstofferzeugenden und weiterverarbeitenden Industrie bei der Herstellung großer Bleche,
- theoretische Erkenntnisse zur Erfassung kontinuumspezifischer Stabilitätsprobleme,
- konstruktive Entwicklungen mit der Einbeziehung der Fahrbahn in das Haupttragsystem,
- Fortschritte auf dem Gebiet der Verbindungstechnik durch den Übergang vom Nieten zum Schweißen,
- Fortschritte im Transportwesen zur Beförderung großer Konstruktionseinheiten,
- Entwicklung leistungsfähiger Montagegeräte.

Eine Analyse der bevorzugten Brückenquerschnitte in den letzten 30 Jahren führt bei einer groben Vereinfachung zu der Erkenntnis, daß bei Großbrücken die Grundquerschnitte der Fig. 3 in der genannten Verteilung zur Anwendung kamen (m^2 ausgeführte Brückenfläche). [1]

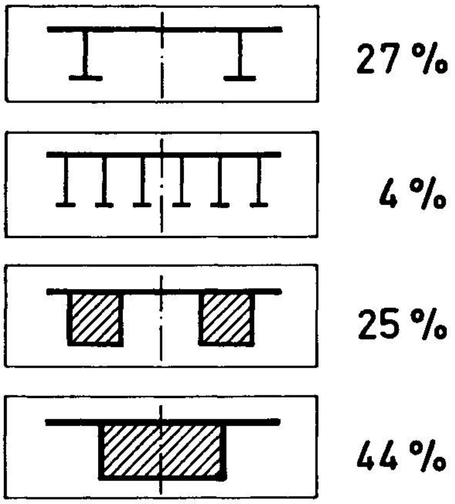

Fig. 3 Stahlbau: Grundquerschnitte der Quersysteme. Anwendungshäufigkeit

Aus heutiger Sicht ist festzustellen, daß eine Konzentration auf wenige Querschnittsformen möglich ist und daß auf die Mischsysteme weitgehend verzichtet werden könnte.

Danach ist die Aussage erlaubt, daß etwa je die Hälfte der Brücken heute wirtschaftlich mit offenem Querschnitt ausgeführt werden könnte, während für die andere Hälfte vorteilhaft torsionssteife Vollwand-Hohlkästen zur Anwendung kommen. Diese Aussage beruht auf einer Auswertung, die nur Systeme mit orthotroper Stahlleichtfahrbahn als Bestandteil des Haupttragwerkes berücksichtigt. Weitgespannte Hängebrücken mit Fachwerkhauptträgern und getrennter Fahrbahn blieben bei dieser Betrachtung unberücksichtigt. Sie würden nach heutiger Auffassung sowieso als windschlüpfrige Vollwandkastenträger ausgeführt werden, wenn nicht eine zweistöckige Verkehrsnutzung für die Beibehaltung von Fachwerkhauptträgern spricht.

Eine besondere Bedeutung wurde und wird der statischen und konstruktiven Durch-

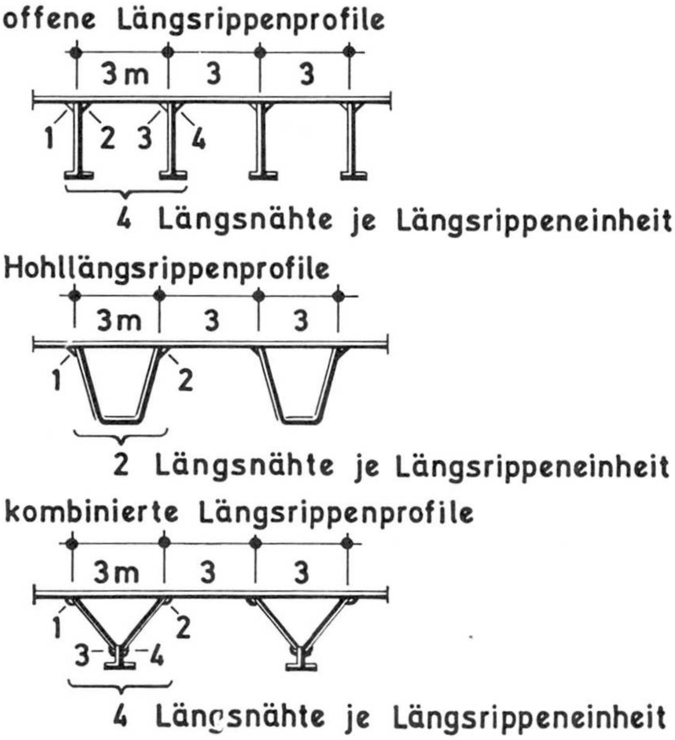

Fig. 4 Stahlleichtfahrbahnen

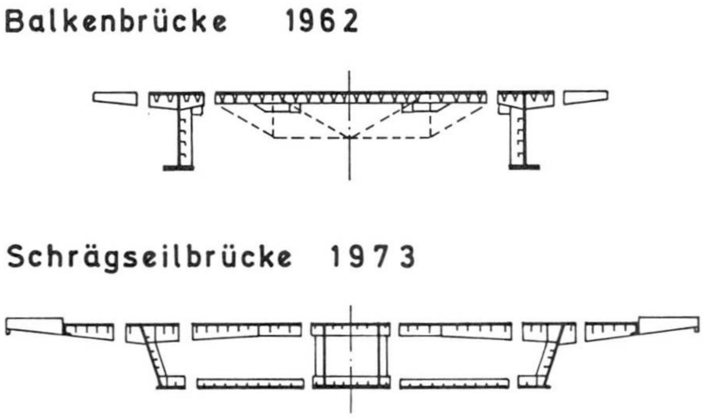

Fig. 5 Längsorientierte Systeme - Querschnittseinheiten

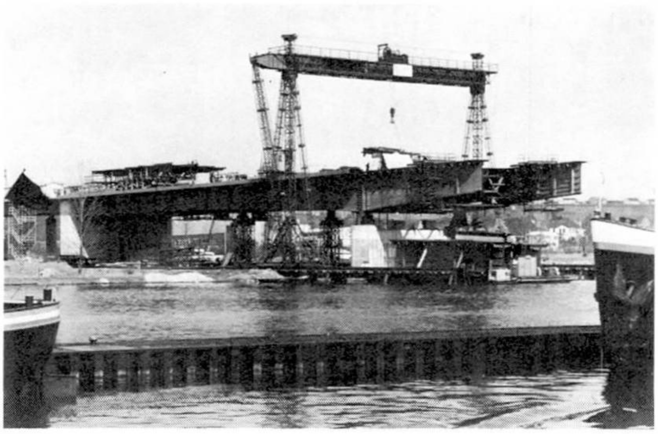

Fig. 6 Mainbrücke Hochheim Werkfoto MAN

arbeitung der orthotropen Stahlleichtfahrbahn beigemessen. Die Berechnungsverfahren liefern aber nur noch maximal 1 % Differenz bezogen auf das Gesamtgewicht, die sich nur im Materialanteil auswirkt. Der heute noch getriebene Aufwand für die Plattenberechnung steht also in keinem Verhältnis mehr zum wirtschaftlichen Erfolg. Drei bewährte Konstruktionen für Stahlleichtfahrbahnen zeigt Fig. 4. Den geringsten Aufwand an Querträgern und Kreuzungspunkten benötigen die Hohllängsrippen, den größten die einfachen Längsrippen als Stege.

Von größerer Bedeutung ist die Entwicklung zum sogenannten "Konstruktiven Kontinuum"; d.h. ein geschickter Zusammenbau des Brückenquerschnittes aus ebenen Querschnittseinheiten. Das besagt: Vorfertigung möglichst großer ebener Einheiten in der Werkstatt mit möglichst geringem Montageaufwand an der Baustelle zum Zusammenfügen der Querschnittsteile zum Brückenquerschnitt. Zur Verdeutlichung zwei richtungsweisende Beispiele für einen offenen und geschlossenen Querschnitt (Fig. 5). [2]

Die praktische Anwendung der Montage eines offenen Querschnitts im Freivorbau zeigt Fig. 6.

Die Entwicklung der letzten Jahre bringt zunehmend durch geschickte Kombination rationeller Grundsysteme den Aufbau von Querschnitten, die verschiedene Längssysteme befriedigen, so daß Vorteile in der Fertigung durch Beschränkung der Systemzahl gewonnen werden. Außerdem ist der Vorteil für Entwurf und Bauausführung offenkundig, wenn es gelingt, ein Längssystem zu finden, das sich aus bewährten Teil- oder Grundsystemen aufbauen läßt. Dieser Rationalisierungs-Trend entwickelte sich aus der Erkenntnis, daß bis zu 60 % der Kosten einer Brücke im Lohnbereich Planung, Werkstatt und Montage anfallen können. Diese Erkenntnis nimmt an Gewicht noch zu, wenn man beachtet, daß z.Bsp. in einem Untersuchungszeitraum von 10 Jahren die Materialpreise um rd. 20 %, die Löhne aber um fast 140 % gestiegen sind. Das Kostendreieck "Material, Lohn, Maschinen" kann also im wesentlichen im "Sektor Lohn" verbessert werden; allerdings durch Investitionen im "Sektor Maschinen". Dies gilt im besonderen für die Werkstatt, aber auch für die Montage. Die Leistungsfähigkeit der Montagegeräte beeinflußt den Lohnaufwand auf der Baustelle entscheidend; sie entscheidet häufig auch neben der Werkstatt und dem Transport über den möglichen Grad der Vormontage als Indikator für die Montagegeschwindigkeit. Das Ziel sollte sein, die Zahl der Montagestöße so niedrig wie möglich zu halten. Dies ist auch im Sinne der Konstruktion selbst, weil damit die Anzahl der Störstellen vermindert wird.

Eine Mittelwertbildung aus zahlreichen Großbrückenbauten verschiedenen Typs der letzten Jahre mag einen Anhalt dafür geben, wie sich der Stundenaufwand für die Werkstatt und Montage darstellt:

- Werkstatt	-- Vorzeichnen	3,3 %	55 %
	-- Maschinenbearbeitung	7,7 %	
	-- Zusammenbau	24,8 %	
	-- Schweißen (Nieten)	14,8 %	
	-- Vormontage	4,4 %	
- Montage	-- Einrichtung, Hilfskonstruktion	8,0 %	45 %
	-- Montage Brücke	31,0 %	
	-- Bauleitung, Sonstiges	6,0 %	

Für eine vollautomatisierte Fertigung zeigen sich konstruktiv entkoppelte Systeme am meisten aufgeschlossen. Hierbei lassen sich gleichmäßige Montageeinheiten herstellen und zeitlich fortlaufend dem Montagetakt angepaßt ausliefern, so daß Arbeitsunterbrechungen infolge Diskontinuitäten vermieden werden können. Die Montage setzt also den in der Werkstatt begonnen Fabrikationsprozess an der Baustelle bis zur Fertigstellung des Bauwerkes fort, und zwar mit den der Montage eigenen Mitteln. Dabei hat man längst das Prinzip

verlassen, das Tragwerk spannungsfrei zusammenzusetzen und erst das fertige Brückensystem seinen Lasten auszusetzen. Die dafür notwendige kontinuierliche Hilfsunterstützung der Montageteile ist finanziell nicht mehr tragbar und häufig auch technisch gar nicht vernünftig darstellbar. Bei den heutigen Montageverfahren wird das Tragvermögen der Montagezwischensysteme ausgenutzt und die Montageschüsse werden in freiem Vorbau mit oder ohne Hilfsstützungen oder -abfangungen nacheinander fortlaufend an den jeweils bestehenden Brückenträger angeschlossen (Fig. 7). Dabei ergeben sich häufig komplizierte Zwischentragsysteme, die mit dem endgültigen Tragsystem nicht mehr übereinstimmen. Hierbei entstehen im allgemeinen große Beanspruchungen in der Konstruktion und aufgespeicherte Montagespannungen sind den Spannungen aus dem endgültigen Gebrauchszustand zu überlagern. Hierdurch bedingter Mehraufwand an Material steht aber im allgemeinen in keinem Verhältnis zu den gewonnenen Vorteilen einer freien Montage. In Fällen, in denen die Montageeinheiten "von außen", also nicht über die Brücke selbst zugeführt und montiert werden können (Fig. 8), lassen sich die Zusatzlasten der schweren Hebegeräte an der Montagespitze einsparen.
Naturgemäß sind für die Montage "weitgespannter Balken" die Schrägseilbrücken besonders gut geeignet, vor allem die Vielseilsysteme, weil sie durch die Zwischenseilabfangungen günstige Montagezwischensysteme anbieten, die sich auch mit dem endgültigen Brückensystem gut in Einklang bringen lassen, wenn es sein muß durch nachträgliche Korrektur der Seilkräfte. Da sie außerdem dem Wunsch nach der Werkstattfertigung des Brückenbalkens mit gleichmäßigem Querschnitt entgegenkommen, ist ihre schnelle Verbreitung im Stahlbrückenbau verständlich.

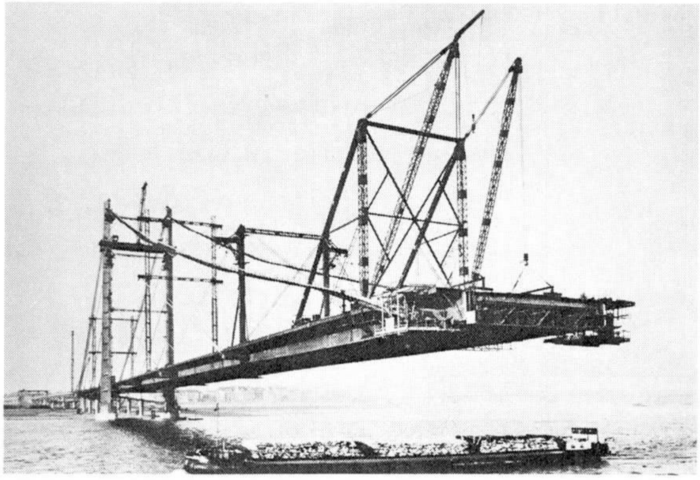

Fig. 7 Kniebrücke Düsseldorf

Fig. 8 Rheinbrücke Weisenau Werkfoto MAN

2.2 ... im Betonbrückenbau

Wesentliche Kostenfaktoren für die Herstellung von Betonbrücken sind "Schalung" und "Rüstung". Hier galt es vor allem, mit einer Rationalisierung einzusetzen, wollte man sich auch bei langen und größeren Brücken gegen den Stahlbau durchsetzen. Den Gepflogenheiten der Stahlbaumontage folgend, entwickelte sich zunächst vor allem die Fertigträgerbauweise und für größere Spannweiten der freie Vorbau in Ortbeton, bei dem der Balken in kurzen Abschnitten mit einer umsetzbaren Schalung schrittweise als Kragträger wächst (Fig. 9).

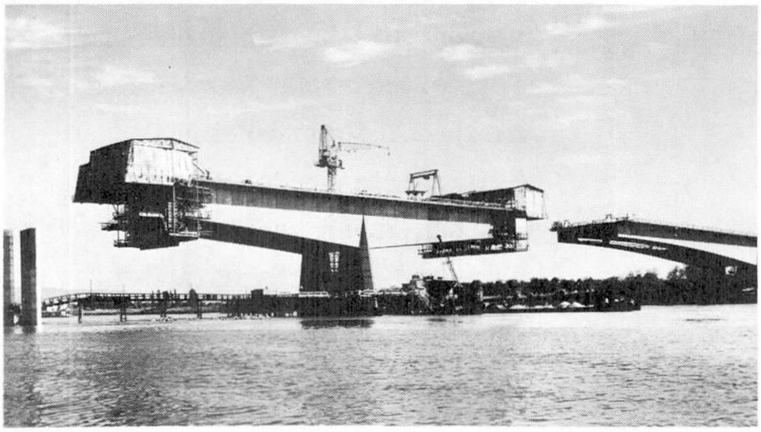

Fig. 9 Rheinbrücke Bendorf - Freivorbau in Spannbeton

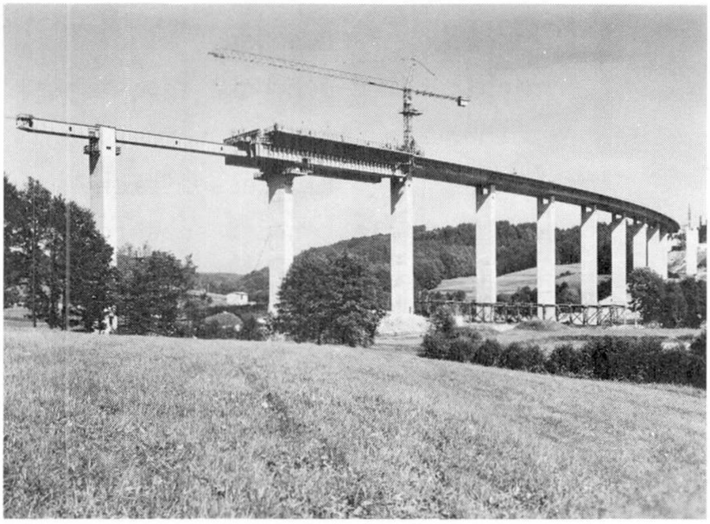

Fig. 10 Feldweiser Vorbau mit Vorschubrüstung

Vor etwa 20 Jahren begann dann der Einsatz der Vorschubrüstungen, die zwischen den Pfeilern freitragend den "Grund" überwinden und feldweise fortschreitend die Herstellung von Ortbetonbrücken im Taktverfahren ermöglichen (Fig. 10). Das Wandern der Produktionsstätte auf der Brücke mit dem fortschreitend wachsenden Überbau machte die Baustelle frei vom darunterliegenden Gelände und erlaubte die Herstellung einer beliebigen Brückenlänge, auch mit veränderlicher Trasse, praktisch problemlos.[3] Bei größeren Spannweiten empfahl sich dabei die Kombination des abschnittsweisen Baues mit dem feldweisen Vorbau (Fig. 11).
Es zeigte sich, daß ein möglichst gleichbleibender Brückenquerschnitt für diese Bauverfahren vorteilhaft ist, um laufende Änderungen an den Schalungen möglichst zu vermeiden. Gleichmäßige Spannweiten erweisen sich als zweckmäßig, um echte Taktwiederholungen ohne Störungen durch örtliche Zusatzmaßnahmen zu gewährleisten. Dennoch lassen sich

Fig. 11 Siegtalbrücke Eiserfeld - Bauzustand mit Vorschubrüstung

aber auch örtliche Abweichungen meistern, wie z.Bsp. die Überwindung einer größeren Zwischenspannweite (Fig. 12).

Die einfache Überlegung, die Baumethode umzukehren, d.h. die Fertigungseinrichtung am Brückenende stationär anzuordnen und dafür die Brücke wandern zu lassen, indem man sie wie aus einer Strangpresse herausdrückt, hatte aber andere Vorläufer. [3]

Fig. 12 Döllbachtalbrücke - Vorschubrüstung mit Zwischenstütze

Die Brücke wurde zunächst in Blöcken nacheinander betoniert, dann zusammengespannt und als Ganzes der Länge nach eingeschoben. Da ein Planum hinter einem Widerlager häufig aber nur beschränkt frei ist, war es besser, die Brücke im Takt, korrespondierend mit den Betonier-

Fig. 13 Talbrücke Gerlingen - Taktschiebeverfahren

längen abschnittsweise vorzuschieben (Fig. 13).
Stählerne Ausleger haben die Kragmomente im Balken aus dem Vorschub zu begrenzen. Eine Spitzenleistung dieser Bauweise wurde kürzlich mit dem Bau der rd. 760 m langen Brücke über den Shatt al Arab in Basrah vollbracht. Eine über zwei Öffnungen anzuordnende Spannbeton-Drehbrücke wurde in den Balken und in den Taktablauf so eingeordnet, daß sie erst aus der Verbindung zu lösen war, als der Balken, von einer Seite eingeschoben, seine endgültige Lage erreicht hatte. [4]

Bei den Versuchen, die Spannweiten zu steigern, erinnert man sich an Vorbilder der Stahlbauweise und wandelt sie "betongerecht" ab. Hier wie da galten die gleichen Ziele: Senkung der Baukosten, Beschleunigung des Baufortschrittes und Qualitätsverbesserung. Alle genannten Bauverfahren arbeiten ohne stationäres Lehrgerüst und mindern damit das schwer einschätzbare Baurisiko. Die Ausführung der sich stets im Takt wiederholenden Arbeitsabschnitte sollen den Arbeitsaufwand senken, nicht nur wegen der ständig steigenden Löhne, sondern zunehmend auch wegen des Mangels an qualifizierten Facharbeitern. So kommt der Vorfabrikation ganzer Querschnittsblöcke im Zusammenhang mit dem feldweise arbeitenden

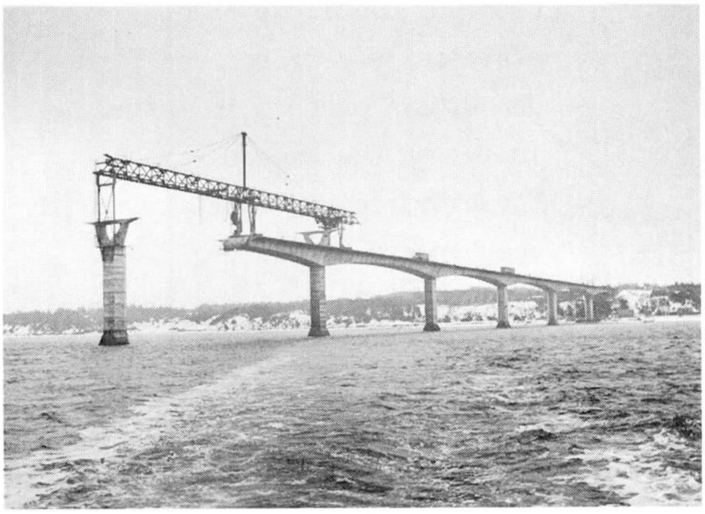

Fig. 14 Sallingsundbrücke - Montage

Vorschubgerüst vor allem (aber nicht nur) für "große Spannweiten" immer mehr Bedeutung zu. Hier laufen die Absichten den Überlegungen im Stahlbau, der Fertigung und Montage in Einklang bringen muß, entgegen. Man will die beim Ortbeton gegebene unmittelbare Verbindung von Montage und Fertigung absichtlich trennen, um vom Wochentakt frei zu sein und auch bei schlechten Wetterbedingungen im Schutz weiter fertigen zu können, um dann vor Ort eine schnellere Montagegeschwindigkeit zu erreichen. Die Fertigung in der "Werkstatt" hinter dem Brückenwiderlager kann außerdem eine bessere Qualitätskontrolle bedeuten. Ob es ausreicht, die Brücke nur mit den Spannkabeln zusammenzufügen und die Fugen ohne durchgehende schlaffe Bewehrung mit Klebern zu füllen, ist immer noch ein Streitgespräch. Die kürzlich sehr genau untersuchte Sallingsundbrücke in Dänemark (Fig. 14) hat jedenfalls keine diesbezüglichen Nachteile erkennen lassen. [5]

Individuelle Lösungen haben auch im Betonbau nur noch eine Chance, wo die Standardlösungen nur zum Teil anwendbar sind oder sich auf Grund besonderer Randbedingungen verbieten. So sind zum Beispiel den ersten Zügelgurt- und Schrägseilbrücken die Vielseilsysteme gefolgt, [3,6] die die kontinuierliche Fertigung des Balkens im abschnittsweisen Freivorbau in Ortbeton oder vorgefertigten Querschnittsblöcken ermöglichen (Fig. 15).

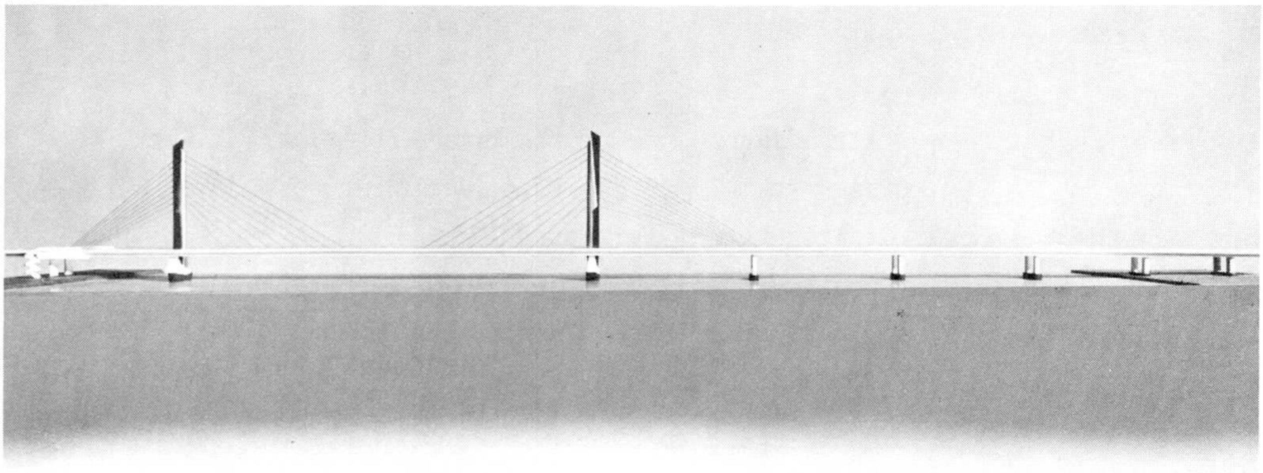

Fig. 15 Schrägseilbrücke - Vielseilsystem mit gleichbleibendem Balken

So wäre sogar denkbar, mit Hilfsstützen oder -abspannungen eine solche Brücke von beiden Seiten im Takt einzuschieben und die endgültige Aufhängung nachträglich vorzunehmen. Für alle Schrägseilbrücken gilt, daß die Seile (heute bevorzugt auf der Baustelle gefertigte Paralleldrahtkabel) als wichtigste und zugleich anfälligste Tragglieder auswechselbar sein sollten. Das spricht für die Vielseilbrücke, bei der es am leichtesten möglich wird, die Seile einzeln

ohne wesentliche Störung des Verkehrs zu erneuern.
Das den Betonbrücken häufig vorgeworfene hohe Eigengewicht kann bei den Seilbrücken überraschend auch positiv eingebracht werden. Der Schwellbeanspruchung aus der Verkehrslast steht ein größerer Seilquerschnitt entgegen, der zwangsläufig die Schwingbreite herunterdrückt; ein Vorteil, der bei hohen Verkehrslasten spürbar werden kann. Dennoch ist eine Gewichtsreduktion für große Spannweiten wünschenswert. Man spricht davon, daß im Spannbeton immerhin 600 bis 1000 m für die symmetrische Schrägseilbrücke möglich sind, für die Brotonnebrücke wurde schon l = 320 m erreicht. [6] Hier könnte ein hochwertiger Leichtbeton in der Zukunft Einfluß gewinnen.
Trotz einiger Ausführungen und interessanter Ideen ist der eigentliche Hängebrückenbau dem Spannbeton bisher verschlossen geblieben. Der Fortschritt im Schrägseilbrückenbau scheint erfolgversprechender, vielleicht schon deshalb, weil selbst im Stahlbau den Hängebrücken nur noch die "ganz großen" Spannweiten vorbehalten sind.

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Service Behaviour of Suspension Bridges

Comportement en service de ponts suspendus

Bewährung von Hängebrücken

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SUMMARY
This paper reports on portions of the experience record in the maintenance of four American suspension bridges and is restricted to discussion of only the superstructure, cables and suspenders. The contents evolved from personal experience, examination of engineering reports and interviews with some of those responsible for the operation and maintenance of the facility.

RESUME
Le rapport présente une partie des expériences acquises dans la maintenance de quatre ponts suspendus aux Etats-Unis et se limite à la superstructure, aux câbles et aux montants de suspension. Ce rapport est le résultat d'expériences personnelles, de l'examen des rapports d'ingénieurs et d'entretiens avec les responsables de l'exploitation et de l'entretien des ouvrages.

ZUSAMMENFASSUNG
Dieser Bericht zeigt Erfahrungen im Betrieb und Unterhalt von vier Hängebrücken in den USA, wobei er sich auf Brückendecken, Kabel und Hängesäulen beschränkt. Der Inhalt des Berichtes beruht auf persönlicher Erfahrung, Untersuchung von Ingenieurberichten und Gesprächen mit den für den Betrieb und Unterhalt des Bauwerkes Verantwortlichen.

Engineers and Architects are not the only ones to appreciate the concept of a bridge or its beauty; there is no greater expression of man's ability to build. Bridges are among the finest examples of structural art, powerful objects of pure utility and science.

One of the best known bridge types is the long-span suspension bridge which, as a result of its beautiful lines, has frequently captured the public's fancy and sometimes has even become a symbol of a city.

The United States is well known for its suspension bridges and one of New York City's main attractions is its eight suspension bridges, the most well known being the Brooklyn Bridge. Another example of a famous bridge is the Golden Gate Bridge which is immediately identified with San Francisco.

Engineers feel privileged if they are fortunate enough to be associated with the design or construction of such monumental structures. But as soon as the ribbon is cut on opening day of the bridge to traffic these same engineers rush on to another new exciting project. In general they are not interested in what they believe to be the tedious jobs of observing the performance, inspection or maintenance of the newly completed structure.

The public is not interested in day-to-day operations. It is even difficult to interest politicians in providing money for maintenance of an existing facility since they would rather have their names associated with new projects. Sometimes it takes a bridge failure to cause a public outcry which results in a spate of maintenance activity on an emergency basis.

While the task may be unglamorous, there is no more important job than ensuring that an existing facility is properly maintained and is capable of providing the services for which it was designed for the desired life of the project.

During this century and especially during the last 50 years, which coincides with the life span of IABSE to date, an experience record has emerged on the service behavior of long-span suspension bridges. This behavior, some of which was planned and some unexpected encompasses not only design but operations and maintenance as well. The designer, owner and users of a bridge have a view, each from a different perspective, of what satisfactory performance might be.

The unique features of suspension bridges, cables, suspenders, towers and anchorages are what distinguishes them from other type of bridges. In addition they are much more flexible and susceptible to damage from wind. In the early history of suspension bridges during the 19th century many suspension bridges were destroyed as a result of oscillations due to wind. Even to this day, the major performance problems of suspension bridges can be traced to their flexibility and response to wind forces. Another major problem concerns corrosion of suspenders, their anchorages and the deterioration of the protective system of the main cables.

This paper reports on portions of the experience record of four American suspension bridges and is restricted to discussion of only the superstructure, cables and suspenders. The contents evolved from personal experience, examination of engineering reports and interviews with some of those responsible for the operation and maintenance of the facility.

Delaware Memorial Bridge

The first Delaware Memorial Bridge was built at a cost of $43,900,000 by the State of Delaware to form a highway link between Delaware and New Jersey across the Delaware River just south of Wilmington. The bridge with a main span length of 2,150 feet and side spans of 750 feet was the sixth longest span in the world when opened to traffic on August 16, 1951. It was designed by Howard Needles Tammen & Bergendoff, and the American Bridge Company was responsible for the fabrication of the steel superstructure, main cables and suspenders. The 7" concrete roadway slab carries four traffic lanes and in addition there is a 3-ft. steel median and two 3-ft. sidewalks.

The Delaware Memorial Bridge was the first major suspension bridge to be designed following the Tacoma disaster other than the Tacoma Bridge itself. As such, the designers were conservative and called for a double lateral system as well as sponsoring a model at Princeton University of a portion of the suspended structure to study performance under torsional loads. The stiffening trusses are 20 feet deep with 26 feet panels and are 61 feet apart. Each cable consists of 19 strands of 436 No. 6-U.S. gage galvanized cold-drawn steel wires for a total of 8,284 wires. After application of substantially all of its load, the cable was coated with red lead paste and wrapped with No. 9-U.S. gage soft annealed galvanized steel wire and then painted. Four hanging suspenders at each panel point are 2-inch diameter galvanized wire ropes.

Traffic on the bridge grew so rapidly that the bridge became functionally inadequate. Consequently the Delaware River and Bay Authority, established in 1962, as the operator of the bridge, proceeded with the construction of a twin bridge, also four lanes wide. Each structure now operates as a one-way facility. The Consulting Engineers for the second bridge were Howard Needles Tammen & Bergendoff and E. Lionel Pavlo, and the bridge superstructure, as well as the cables and suspenders were constructed by the Bethlehem Steel Co. To the eye, there is not much difference between the twin spans.

For the new bridge, recognizing the trend to heavier wheel loads, a substantially stronger roadway was designed to provide for reduced maintenance costs and to keep future traffic disruptions resulting from required deck repairs to a minimum. In addition, the concrete slab was overlain with a 1-1/2-thick-inch bituminous concrete wearing course. The first bridge was designed for a 12,000-lb. wheel load and the second bridge for a 20,000-lb. wheel load which resulted in a concrete slab depth of 8 inches with an increase of 50 per cent in the required reinforcing steel. Furthermore, welded bar trusses were fabricated and used as the reinforcing steel in the new slab.

The concrete deck slab on the original bridge, which was only 7" thick, had developed hairline cracks, and spalling and potholes at many locations on the top surface of the slabs. In addition the 3-foot wide steel median and barrier had to be removed to provide for the free movement of four lanes of traffic in one direction. Also, each of the existing 24-foot wide concrete slabs were crowned at their respective center lines to provide drainage runoff in two directions and would have to be rebuilt to provide a normal four-lane roadway transverse profile with a single crown at the center line of the bridge, the area formerly occupied by the steel median. In view of the above three reasons, a decision was made to replace the concrete deck slabs in their entirety. The same design criteria for the design of concrete slabs as used for the new bridge was adopted and resulted in an 8" slab thickness. However it was not possible to add a separate bituminous concrete wearing surface on the new concrete deck slabs as on the second structure because calculations indicated that the additional weight imposed on the cables could not be tolerated.

In establishing the design criteria for the second structure, the Authority asked that a more extensive network of catwalks throughout the suspended span be provided in order to facilitate access to the various parts of the structure below the bridge deck for inspection and maintenance purposes. This was done and in addition a waterline system, extending across the entire bridge, with valves and outlets located at convenient intermediate points was also provided. This waterline enables maintenance personnel to wash and clean the structural steel members below the deck at regular time intervals. It was also decided to install these systems on the first structure to facilitate maintenance operations.

An important step in the continuing maintenance program of the original structure was the replacement of worn suspender ropes. The bridge deck is suspended from the two main cables by means of 276 suspender ropes, each 2 inches in diameter. At alternate panel points of the stiffening truss, two suspender ropes are looped over the cable bands which are placed around the main cables and at their lower ends are socketed by four forged steel sockets which are slid into slots of the bearing angles connected to the verticals of the stiffening truss. The suspenders pass through a steel collar casting which is riveted to the top chord of the stiffening truss. Reversed conical shaped holes were provided to permit the ropes to move slightly in horizontal directions under the movement of the suspended deck. In addition the suspenders were protected from abrasion at the collar by galvanized steel sizing wound around each wire rope. Nevertheless there was wear at the collar casting level when seizing wires became loose and broken and the suspender ropes came into direct contact with the surface of the steel collar castings. This situation was particularly serious at the panel points in the middle sections of the center span where the suspender ropes were the shortest and the deck movements greatest. The Authority decided to replace 44 of these damaged suspender ropes and to remove and replace the existing damaged sizing wires. In addition a research program was initiated with the purpose of developing a collar liner of suitable material that would minimize the damage to suspender ropes.

Results of the investigations indicated that adiprene (a urethane elastomer) or neoprene liners molded to conform to the shape of the suspender ropes would be the most effective in protecting the suspender ropes. Accordingly, all existing collar castings were replaced with new lined steel collar castings, half of which were lined with adiprene and the other half with neoprene. This procedure would allow a comparative study of the long term performance of the two types.

The two 24-foot concrete roadway slabs of the original bridge were separated by a three-foot steel median and supported on steel stringers that were continuous over four spans and 104 feet in length between expansion joints. The performance of these stringers was satisfactory. However shortly after the roadway deck modification as described earlier, cracks were discovered at the ends of the stringers in the web at or near the bottom flange. At these points a diaphragm between stringers, which supported the end of the concrete deck, was connected by an angle to the stringer web. This angle was not full depth and a gap was left between the bottom of the angle and the bottom flange of the stringer. It was concluded that lateral secondary bending of the web in this gap resulted in a fatigue failure. The secondary bending resulted from the resistance of the roadway deck system to the torsional motions of the suspended span. These same type cracks also appeared in the second structure and in at least one other suspension bridge.

The purpose of maintenance operations is to preserve the physical facilities constructed in as nearly perfect condition as possible. A continuous and thorough maintenance program assists in minimizing total maintenance costs for the life of the project.

The work of maintaining a system, as large and having so many different elements as the Delaware Memorial Bridge, includes many diversified types of activity. The useful life of a structure and its value is directly influenced by the care and thoroughness with which maintenance is conducted. Poor maintenance of a costly project will result in accelerated deterioration and eventually result in expensive emergency repairs or rehabilitation.

Maintenance operations are centered in a maintenance building constructed adjacent to the bridge. The equipment is housed and repaired at this location.

A highway facility for which a toll is charged must be kept in satisfactory operation as nearly 100 per cent of the time as possible for if the project is closed for any great length of time, then a distinct dollar loss accrues because of the inability to collect tolls. Locating the maintenance headquarters and equipment yards immediately adjacent to the project insures that emergency operations associated with snow and ice, the most frequent causes of temporary interruptions in service, will be initiated promptly.

Periodic inspections of the bridge are made by the Consulting Engineers who prepare and publish an annual report on the condition and operation of the bridge. The inspection includes all parts of the project including the substructure, anchorages, superstructure, concrete decks, towers, cables and suspenders, fenders, lighting, signing, signal and television surveillance systems, drainage, toll facilities and buildings. The report includes a recommended schedule of major repairs for the next year. Also included is a maintenance and operation budget.

To supplement this annual inspection a 5-year repetitive program was adopted by the Authority in 1971, which provided for a comprehensive detailed in-depth inspection covering every accessible portion of the bridge. The general limits of each year's in-depth inspection are as follows:

- 1st year : Underwater Inspection (both bridges)
- 2nd year : Approach Spans of Second Structure
- 3rd year : Main Spans of Second Structure
- 4th year : Main Spans of First Structure
- 5th year : Approach Spans of First Structure

It is expected that this program will be repeated after a one-or-two-year interval since it has proved to be very effective.

Golden Gate Bridge

The Golden Gate Bridge has a main span of 4,200 feet with side spans of 1,125 feet and was constructed at a cost of 27 million dollars. The bridge connects San Francisco with Sausalito and at the time of its opening to traffic on May 28, 1937 until 1964 it had the longest span in the world. The Chief Engineer for the project was Joseph B. Strauss, the superstructure steel was furnished and erected by the Bethlehem Steel Co. and the steel cables, suspenders and accessories were furnished by John A. Roebling's Sons Co.

The bridge has a 60-ft. roadway with two - 10-ft. shoulders. The roadway deck is a 7-inch thick concrete slab. The stiffening trusses are 90 feet apart and 25 feet deep. The panels of the stiffening truss are 25 feet on centers with suspender support points at alternate panel points. There is a top lateral system and at the time of opening there was no bottom lateral system.

Each 36-3/8-inch diameter cable consists of 61 strands with 452 wires for a total of 27572 wires. These wires are No. 6-U.S. gage galvanized cold-drawn steel wire which have a diameter of 0.196 inches. The wrapping wires, No. 9-U.S. gage soft annealed galvanized steel wire,were laid in a heavy red lead paste. A galvanized metal primer was applied to the wrapping and followed by a red lead paint and then the final coat of International Orange Paint.

Through the years the Golden Gate Bridge has been exposed to severe wind storms. In 1941, one of these storms lasted for three hours with wind gusts up to 60 mph. It was reported that lateral deflections of the bridge reached 5 feet and the vertical movements at the quarter point of the main span were 2 feet with a frequency of 0.13 hertz. The most severe storm occurred in December 1951 and lasted for over six hours. At its peak a wind gust velocity of 69 mph was recorded with an average wind velocity of 55 mph for 20 minutes. A double amplitude of 11 feet was measured at the quarter point with a frequency of .28 hertz. Traffic on the bridge was stopped at the peak of the storm. There was considerable damage to the lateral system connections of the center span to the tower which had to be repaired.

As a direct result of the storm a Board of Engineers was appointed to investitate the viability of installing a bottom lateral system and report on the benefits that would result. Section models to a scale of 1:75 were tested in a wind tunnel. In addition tests made at Princeton University in connection with the Delaware Memorial Bridge were cited in that they indicated that the bottom lateral system on that bridge increased the torsional rigidity by about 20 times. The Board recommended that a bottom lateral system be added to the Golden Gate Bridge and it was installed in 1954. It has evidently solved the wind stability problem of the bridge.

At the same time that the bottom lateral system was being installed, travelling maintenance platforms were added to the main and side spans to facilitate the inspection and maintenance of the bridge.

During the period 1967-1969, the firm of Ammann & Whitney conducted an in-depth inspection of the bridge. The wrapping on a section of the main cable 10 feet long near the center of the main span was removed and the condition of the wires observed. The surface of wires were all found to be in excellent condition. Wooden wedges were driven into the cable to examine some of the interior wires which were also found to be in excellent condition.

The cable band bolts on the bridge were retightened in 1954. Measurements in 1968 indicated average per cent relaxation from 31 per cent to 71 per cent with an average of 48 per cent for 14 panel points.

The suspenders consisting of 2-11/16-inch diameter galvanized wire ropes were found to require replacement due to severe corrosion especially in the bottom four feet of the ropes which were inaccesible. On the other remaining part of the ropes it was found that the galvanizing had worn quite thin. The severest corrosion occurred in the lower three inches of the rope and at the point where the suspender passes through the guide casting on the top chord. As on the Delaware Memorial Bridge, the suspender had been wrapped with wire at this point to prevent abrasion. The suspenders structural bearing connection to the stiffening truss vertical has undergone severe metal loss. These connections will be removed at the same time that the suspender ropes are replaced and replaced with a new bearing connection detail that provides ample space for inspection and maintenance.

Mr. Henry D. Reilich, Chief Engineer, Golden Gate Bridge, Highway and Transportation District, has reported that the bridge has been undergoing a complete new painting cycle which should soon be ended. He expects that the paint system will have a 20-year life. The following operations describe the process:

- First sandblast all metal.
- Apply 3 mils of a dry inorganic zinc primer.
- Apply 1/4 mil vinyl wash primer.
- Apply two coats of 1 mil each of a vinyl top coat keeping the the landmark color of the bridge namely International Orange.

Ambassador Bridge

The Ambassador Bridge spans the Detroit River between Detroit, Michigan, and Windsor, Ontario. The bridge was opened to traffic on November 15, 1929 and has a main span of 1,850 feet which at the time was a record span, which was exceeded in 1931 by the George Washington Bridge with a span of 3,500 feet. The bridge carries four lanes of traffic on a 47-foot wide concrete slab roadway deck and has an 8-foot sidewalk on one side only.

The bridge is 67 feet wide between cables but only 59.5 feet center-to-center of stiffening trusses which are 22 feet deep. The load from the deck is transferred via a cantilever bracket from the center-line of stiffening truss to the suspenders at the center-line of cables. An unusual arrangement but one that allows the suspenders to be accessible for maintenance and inspection at the bottom.

An in-depth inspection of the bridge was performed in 1974. Few alterations have been made to the bridge over the years. The bridge was found to be in remarkably good condition despite its age.

The condition of the suspender ropes is particularly noteworthy as little corrosion was observed. The bottom connections of suspender ropes are often a problem area. In this case, good initial design and continuing maintenance has paid off.

The main cables were found to be in good condition and the cable wrapping had just a few places where the paint had chipped. Cable anchorages, including strand shoes, eyebars and pins, are in good condition except for some corrosion on some strand shoes.

Three cable bands were reported as having moved slightly. In view of this observation it was decided to embark on a program of determining the remaining tension in the cable band bolts. The American Bridge Division of U. S. Steel performed the required testing for 11 cable bands. The average bolt tension at the cable bands tested was found to be 13,400 lbs. Assuming the initial tensioning to be 60,000 lbs., as was specified at the time the bridge was constructed, the average loss of tension is about 78 per cent in fourty-seven years.

Wheeling Bridge

The suspension bridge at Wheeling, West Virginia, spanning the Ohio River is one of the oldest bridges of this type in the world and was the first bridge with a span length greater than 1,000 feet. It was designed by Charles Ellet, constructed by the Wheeling & Belmont Bridge Company, and opened to traffic on August 1, 1850.

In 1969, the American Society of Civil Engineers dedicated the bridge as a National Historic Civil Engineering Landmark and in 1975, the National Park Service designated the bridge as a National Historic Landmark.

In 1854, a wind storm caused excessive vertical oscillations of the bridge which resulted in the hangers breaking loose and the deck dropping into the river. Mr. Ellet, making use of much of the original material, restored the structure to use and it was reopened to traffic in January 1856.

In 1872, a system of radiating stay cables from the tower tops to the deck was also installed in accordance with a scheme designed by the Roeblings.

During the late 1940's, more than a hundred badly corroded wires at the East Tower were discovered. The damaged lengths were removed and new lengths of wire spliced in.

In 1966, the 1-inch Ø hanger rods on all four cables for 49 panel points over the central 400 feet of the bridge were replaced.

Over the years, there have been numerous changes in the cross-section of the bridge. The original wood floor was replaced in 1930 by creosoted yellow pine laminated flooring. In 1956, the entire floor system was replaced with open steel grating, both for the roadway and sidewalks. In addition new floorbeams were installed at the same time.

The bridge has served well with several cycles of rehabilitation. However age has taken its toll in recent years and the bridge has deteriorated to such a degree that major repairs are required to allow the bridge to continue providing services. Because of the Bridge's great historical significance, the owner, the West Virginia Department of Highways, will make these repairs and also provide for on-going maintenance and inspection.

In the past 37 years, Howard Needles Tammen & Bergendoff (HNTB) has conducted six inspections of the bridge, the latest being in the summer of 1978.

The present-day bridge has a main span of 1008.5 feet, carries two lanes of automobile traffic on a 20-foot roadway and has two 4-foot sidewalks. The stiffening truss with approximately 8-foot panel lengths is 6.17 to 6.75 feet deep and is constructed of timber. This is a very shallow structure for so long a span.

There are four main cables supporting the bridge, each about 7.5 inches in diameter and consisting of 2200 - No.10-gage galvanized wires with a diameter of 0.135 inches. To protect the cable from the weather each is tightly wrapped with No.14-gage galvanized wire with a diameter of 0.080 inches. The cables are painted to produce a seal to exclude moisture. Of note, that while some of these wires have broken and rusted away, the original cables are for the most part still intact and carrying load after 130 years of service. Perhaps there is something to learn here for our modern day cable-stayed bridges, some of which are being constructed with ungalvanized wires. Some engineers are even talking about replacing cables on such bridges every 30 years which has to be expensive, inconvenient and not in the client's or public's interest.

During the latest inspection, the main cables were unwrapped at 13 locations, wedged apart to permit a close visual observation, and then rewrapped using the elasto-wrap system which utilizes an elastomeric material rather than wrapping wire. At 12 locations, the main cable wire strands appeared to be in good condition. At the 13th location, however, 15 broken strands were found. On the top half of the cable, the corrosion appeared to extend 4 layers deep and on the bottom half, only the outside layer showed indications of deterioration.

At many locations, the wire wrapping was found to be badly corroded, loose and broken away, all of which allows deterioration of the underlying main cable strands. At several of these locations serious corrosion of the main cable wires was found.

In view of the broken wires of the main cable found at location 13 and the indication of loose cable wrapping, a subsequent inspection was authorized to examine the entire length of main cables for additional signs of distress. The inspection was carried out and revealed that the paint coat was worn off and the wire wrapping loose or corroded over 50 per cent of the length of the main cables. More locations were pinpointed that had a number of broken main cable wires. At one point, 50 cable strands were found to be broken and about 85 additional strands had up to a 50 per cent loss of section. The cable was wedged open at four positions at this location and no corrosion was observed beyond the second outermost layer of wires.

To ensure that this historical bridge remains in service, the entire wire wrapping will be removed. New wire strands will be spliced in wherever a strand is found to be broken or seriously corroded and a protective coating will be applied consisting of liquid neoprene followed by a neoprene hypalon cable wrapping.

The timbers that comprise the stiffening trusses are weathered and deteriorated. The entire bottom chord will be replaced as well as the decayed sections of the diagonals and top chord.

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Some Remarks on the Service Behaviour of Steel Railway Bridges

Quelques remarques sur le comportement en service de ponts en acier

Einige Bemerkungen über die Bewährung von Eisenbahnbrücken aus Stahl

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SUMMARY
The paper touches on the assessment of the actual behaviour of several bridges with emphasis on load spectra, actual field measured behaviour, fatigue and fracture behaviour, bridge fires and the railway's control plans for the above. The reduction in safety against brittle fracture in changing from riveted to welded structures is pointed out.

RESUME
L'article traite de l'évaluation du comportement de divers ponts et spécialement du spectre des charges du trafic, de la performance réelle mesurée sur l'ouvrage, du comportement à la rupture fragile et à la fatigue, des incendies des ponts, et les mesures de contrôle adoptées par la Compagnie. L'article décrit également la diminution du coefficient de sécurité à la rupture fragile d'une structure soudée comparée à une structure rivetée.

ZUSAMMENFASSUNG
Der Artikel behandelt das Verhalten mehrerer Brücken: Lastspektren, Messungen am Bauwerk, Verhalten während der Ermüdungszeit und des Bruches, Feuer an Brücken und Schadenverhütungsmassnahmen an Eisenbahnbrücken. Es wird unter anderem beschrieben, wie der Wechsel von genieteten zu geschweissten Brücken zu einer Verminderung der Sicherheit gegen Bruchfestigkeit im spröden Bereich geführt hat.

1. INTRODUCTION

With over 3,500 steel bridges varying in size from the largest cantilever bridge in the world [1] to some very small structures, a similar number of timber structures, and a smaller amount of reinforced and prestressed concrete bridges, CN Rail is in a good position to observe the service behaviour of bridges in the northern part of North America.

The average age of CN's steel bridges is over 60 years with the oldest surviving rail carrying super-structure being 96 years old. Because of generous safety factors and good detailing, CN steel bridges have and continue to serve the Railway well. Our current design loading is Cooper's E-80 as per the American Railway Engineering Association (AREA) Manual [2].

Typical current loadings are in the Cooper's E-50 to E-60 range, typified by the 91 metric tonne (100 ton) capacity car weighing 119.3 tonnes (263,000 lbs.) gross on four axles with an equivalent load per foot of track of not more than 8.93 tonnes per meter (6,000 plf) from coupler to coupler, and six axle locomotives weighing 176.4 tonnes (389,000 lbs.) and meeting the same criteria. There are of course occasional heavy loads. The heaviest the author has cleared was equivalent to Cooper's E-109 or roughly double the standard heavy car. There are, in addition, several lines carrying captive ore cars with Cooper's equivalent ratings in the E-60 to E-80 class.

The distribution of current bridge service loadings (Load Spectra) is quite wide as shown in figure 1. The distribution shown in Figure 1(a) represents a line carrying primarily bulk commodities and some small quantities of mixed freight to the west coast port of Vancouver, B.C. [3]. The distribution shown in Figure 1(b) represents a good mix of freight with a predominance of grain traffic just west of Winnipeg, Manitoba, roughly in the middle of Canada. The distribution shown in Figure 1(c) represents an ore carrying line leading from Northern Ontario. Finally, the distribution shown in Figure 1(d), just west of Montreal, shows the influence of relatively dense passenger traffic on the load spectrum.

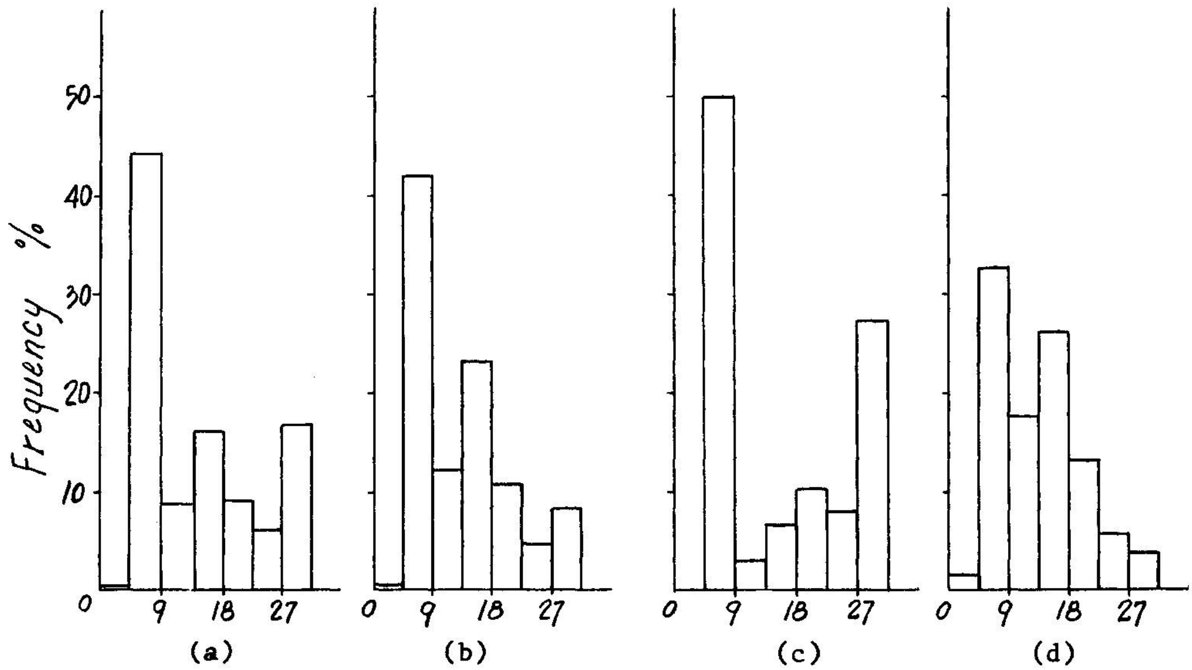

Fig. 1 Axle Load Histograms (Metric Tonnes)

As time goes on and our Railway becomes more efficient, the percentage of heavy traffic will increase [4]. On short spans, this will accentuate fatigue problems on those spans where these loadings were not adequately considered.

Six years ago, we examined our main line steel bridges and found 30 or so about which we were concerned. Several were replaced as it was less expensive to do so than to subject them to detailed analysis. Others were eliminated from consideration because redundancy would prevent collapse when cracking occurred. Of the remainder, six were chosen for detailed study. The emphasis has been placed on either major or typical structures where normal inspection would be unlikely to either find a crack or would not detect it in time. In short, those structures with little or no fail safe capability.

In examining the variables which control fatigue failure, the three most significant are geometry of the detail, stress range and load spectrum.

The influence of detail geometry, primarily stress raisers and defects have been studied by others [5]. Laboratory studies can be used to predict the useful life of various details with a degree of statistical certainty. On some details where insufficient information was available, we conducted our own tests as in the case of the fatigue life of wrought iron beams with holes in the bottom flange. One of the spans we were examining was destroyed by a barge just as a report was being prepared on its potential for fatigue failure. The pin plates were salvaged and examined to verify the bearing angle, size of initial defect and so on.

Nevertheless, critical details can usually be determined from the drawings supplemented by a field inspection. A preliminary check can be made to see if there is a potential problem. If there is, then a more detailed analysis may be necessary.

A rough idea of the loads to which a structure has been subjected can be gleaned from records of the loads carried. This has meant hours of laborious cross checking of data and of extracting from hand written records going back to the time before the formation of I.A.B.S.E. as computerized records were only begun in 1968. Fortunately, light cars tend to do little damage and ignoring them induces little if any error. It is indeed fortunate that the Miner-Palmgren Law (n/N) supports this. History also makes the task easier (or perhaps possible) in most cases. With the exception of locomotives, heavy vehicles in regular service are a recent phenomena. On our railroad, 91 tonne capacity, 4 axle cars (119.3 metric tonnes gross to rail) began to appear in regular service in the early 1960's. The introduction of major main line long hauls of bulk commodities started in April 1970.

To illustrate the point, we need only consider the gross ton miles carried by the Fraser River Bridge (F.R.B.) at New Westminster, B.C. One can see the tremendous increase in traffic that started in the late 1960's (Figure 2). It is evident that the traffic in this and the next decade will have a predominant bearing on the fatigue life of this bridge.

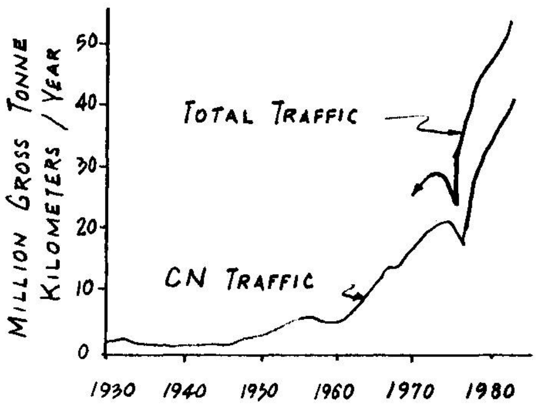

Fig. 2 Annual Tonnage (F.R.B.)

A sample of regular traffic was weighed at speed at each of the six locations under consideration using rail shear circuits developed by our own laboratory. This enabled us to check the effect of sequence which on the bridges examined so far has proven to be of no significance, and to verify the accuracy of the computerized data base being used to record the analytical load spectrum. Correlation has been quite good after correcting for known dynamic effects.

Recently, some concern [6] has been caused about the phenomena of high frequency vibrations in certain members. This is being examined.

Knowing the load, one then requires the stress that it causes at each critical detail. Load paths in some structures are too complicated to precisely determine the stress range by calculation alone. The actual behavior of certain structures can only be determined by field strain measurements. One must bear in mind that a 10% variation in stress range leads to a 33% change in the number of permissible cycles. This is for a slope of -3 on the Wöhler diagram. For flatter slopes, the effect is even larger. It should be evident that it is very important to have as accurate a stress relationship as possible.

On the six structures considered, a special work train was used to establish a load to stress relationship at each gauge both statically and at speed.

This relationship was used to confirm an analytical theoretical model. Of the trusses examined, one behaved as a simple truss, another as a 3-dimensional space frame under moderate loading, and the others somewhere in between. The same was true of stringers in terms of continuity. None of this is surprising.

A large sample of regular traffic was then recorded and a relationship established between the measured weights and the strains at each gauge. The work train test and theoretical analysis were used as a check on this relationship.

The measured weights were then compared to the computerized data base. This data base is prepared by sales and operating personnel. The relationship, after accounting for dynamic effects, was quite good. This justified the use of the computerized data base to derive a load spectrum for the structure under consideration. The longest test recorded every train passing for a seven day period.

After the field test data is analyzed and tabulated, the load spectrum from the computer data base is applied to the relationship, linking it directly to the strain at each gauge location from the sample of regular traffic. In this way, average impact and sequence are automatically included. The work train tests are used solely as back-up and justification for various effects in the relationship.

At this point, either Miner's rule or a Root-Mean-Square stress range can be used with known Wöhler curves to predict the onset of fatigue damage.

The first two applications of this technique were able to show that several previously condemned spans of a bridge could be retrofitted in lieu of complete replacement. The savings to the Company was well over $50 million.

In another case, the measurement of lateral impacts was sufficient to explain several failures that occurred on structures less than 5 years old. Needless to say, details on other structures would be designed to avoid recurrence of the problem. The data gathered by the writer has been instrumental in preparing the new A.R.E.A. fatigue clauses and has led, together with the work of others, to a new awareness of the importance of good detailing and enlightened assessment of new and older bridge structures. Because of the fact that most older metal spans are made of material having very low fracture toughness, fatigue cracking can lead to brittle fracture. CN's fracture control plan is designed to avoid any serious collapses.

For many years, cracking of riveted structures was controlled by the inherent crack stoppers between the component parts of built up members (Figure 3) [7]. Inspection intervals were frequent enough to report any cracks, and corrective measures were taken before an adjacent part could develop its own cracks. The historic exception has always been rolled sections which generally did not have serious stress raisers combined with frequent high tensile stress excursions.

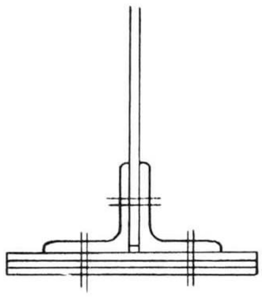

Fig. 3 Built up Member

It seems as though fatigue and fracture are a far more serious problem in welded structures [8] than in riveted structures. Part of this is due to the long period of experience with riveted structures during which most of the really bad details were eliminated, and part is due to the inherent component redundancy and somewhat lower rigidity of riveted structures. Unfortunately, welded structures tend to be less forgiving of small defects than riveted structures because they normally contain less excess material and because the welds themselves are points of rigidity and residual stress. Details which had been derived over many years of experience as adequate for riveted structures proved to be inadequate for welded structures.

With the advent of welded construction and its lack of component cracks stopping planes and its somewhat less tolerance for secondary out-of-plane displacements, the number of serious cracks requiring immediate attention has grown. Fortunately, with the advent of welding, CN Rail also went to very tough steels (Charpy values of 20 Joules at -17°C), with their large permissible critical crack sizes, and generally also went to more redundant (multi-beam) spans.

The first group of problems to turn up on our railway on welded structures were cracks at the bottom of stiffeners on skewed structures. Figure 4 is typical of this detail with a diaphragm or brace frame attached to the stiffener. Note that the stiffeners are not extended to the bottom flange. This was in blind obedience to the dictum of an early researcher in welded construction that one should not weld to the tension flange. Because of the stresses introduced by the differential deflections of the connected girders, and to small out-of-plane movements, the first cracks appeared in less than 5 years on heavily travelled lines.

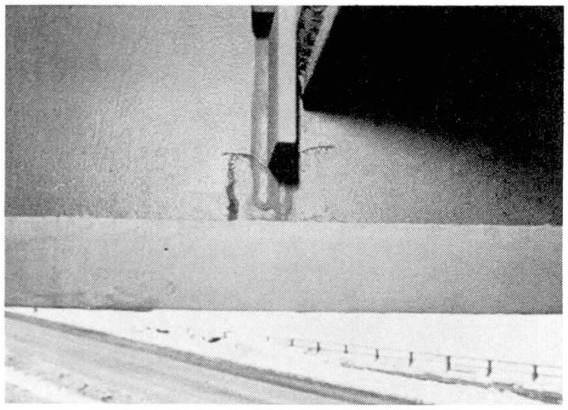

Fig. 4 "U" shape crack

The crack would start at the bottom of the stiffener and form a "U" shape crack around the bottom of the stiffener. If the original stiffener-web weld was of good quality, this crack would turn out into the web and slow down considerably.

After cracks have occured as described above, the next point of rigidity is between the web and flange. If there is any out-of-the plane of the girder motion, it is only a matter of time before cracks will occur in the web to flange weld below the stiffener (Figure 5). Stopping these cracks is very important.

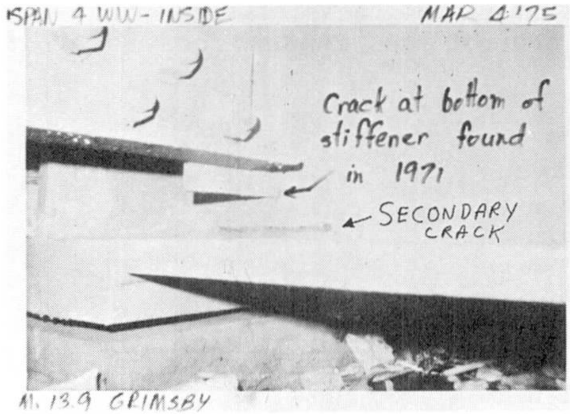

Fig. 5 Secondary Cracking

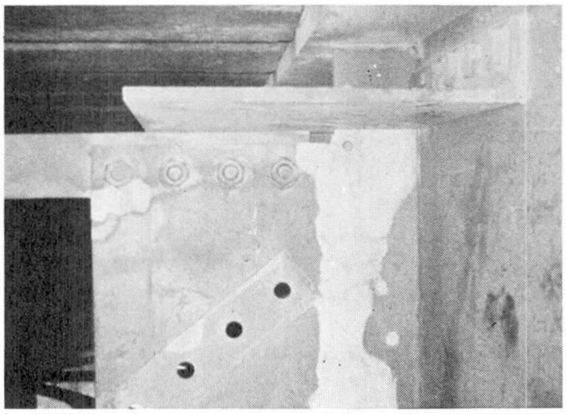

Fig. 6 Gusset to Stiffener

Similar cracks have occured on a few non-skew structures at the bottom of a stiffener simply because the stress range was too high. These take about 10 years to develope a sufficient number of load cycles.

Another detail that has given trouble is the connection of brace frame angles to stiffeners (Figure 6) with the gussets groove welded to the stiffener. Referring to Figure 7, the critical detail has a zero radius. The stress causing crack growth is the lateral force from trains. Since maximum lateral impacts often occur due to hunting (side to side snaking) of empty cars, it took less than five years for these cracks to develop.

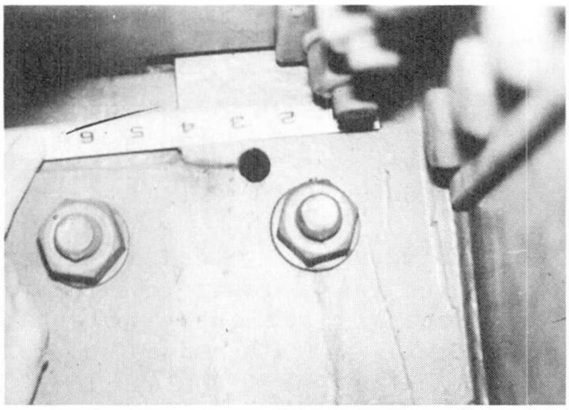

Fig. 7 Zero Radius

Longitudinal stiffeners are usually supplied in varying lengths and butt welded. Since they are usually in a compression area not much concern has been given to inspection in the past. At least one railway and several highway departments have suffered failures from defects in these welds. Although it is true that critical cracks exist only in tension areas; remember that most welds are tension areas because of residual stresses induced by the welding process. A crack can run along a weld until it reaches a tension area or until there is no material left [8].

2. REPAIRS

The subject of steel bridge repairs is just as important as initial design as often the cure is worse than the original problem [3]. The major problem with welded repairs to a riveted girder is that the weld destroys the initial component redundancy of the girder.

In the late 60's, it was decided to repair the corrosion that occurs in deck plate girders at the web-bottom flange junction on a number of our structures. Because of the production operation, patch plates were of various lengths, and were butt welded as required. After 10 years these welds started to crack under very low stress ranges. The maximum measured stress range was 36.2 MPa (5.25 ksi) with a mean peak per train of 31.92 MPa (4.63 ksi). There was little if any apparent dynamic augment.

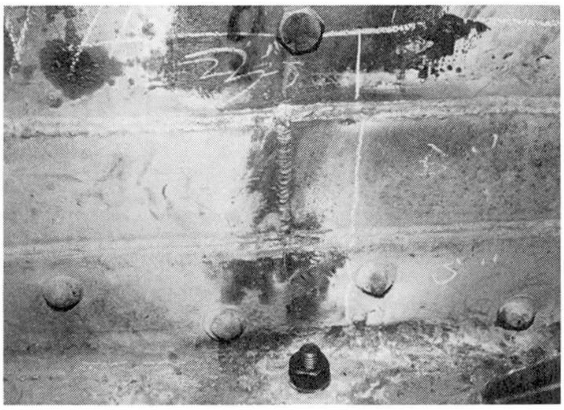

Fig. 8 Crack in Patch Plate

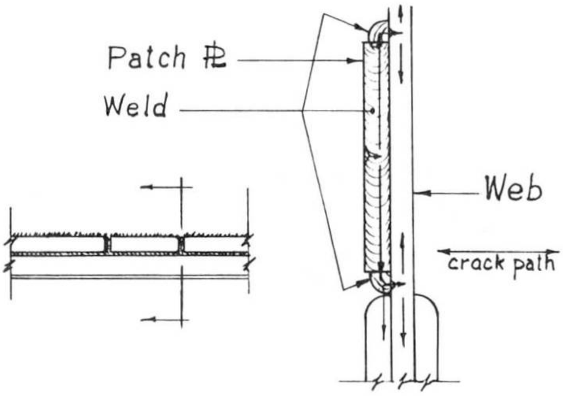

Fig. 9 Crack Growth

The cracks (Figure 8) in these welds were very hard to detect and were not detected until they had propagated from the patch plate welds into the web.

The crack path was as follows:

1. Crack initiation at a flaw.
2. Crack growth (Figure 9) in the weld upward and downward in those welds that did not have full penetration and horizontally into the web for those that did.
3. Growth from the level of the horizontal fillet welds into the web and bottom flange leaving multiple cracks.

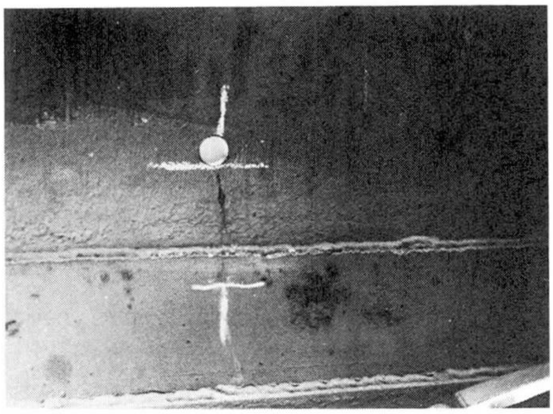

Fig. 10 Cracked Through

Figure 10 shows the same location looking from the inside of the girder. Note that there is no butt weld on the inside at the location of this crack. The crack has propagated from the opposite side.

Four crack fronts have been illustrated from the same initial crack. Depending on the degree of penetration, more crack fronts could theoretically develop. Hence, the repair procedure cannot consist solely in repairing the visible cracks as there may be other hidden cracks.

In the aircraft industry crack stoppers are used. This may consist of a line of closely spaced rivets, a stiffener or a band of much tougher material. In riveted or bolted bridge structures the interfaces between component parts act as crack stoppers. Join these with a weld and the crack stopper is by-passed. If this could lead to a catastrophic failure it must not be permitted.

3. STOPPING THE CRACK

The classic way of stopping a crack is to drill a round hole [9].

In some cases such as where a crack penetrates the stiffener to web weld a minimum of three holes must be drilled. The following procedure has been used successfully on at least two bridges:

1. Drill 7/16 diameter holes on both sides at the stiffener through the web at an approximate angle of 30° away from the stiffener (Figure 11) making sure

to get a truly circular hole with as few rough edges as possible.
2. The resulting hole in the web should be reamed from the outside using a reamer of the same diameter throughout.
3. Attention should be taken so that all rough edges are made smooth even though a truly circular hole is not obtained. Hand filing may be necessary.

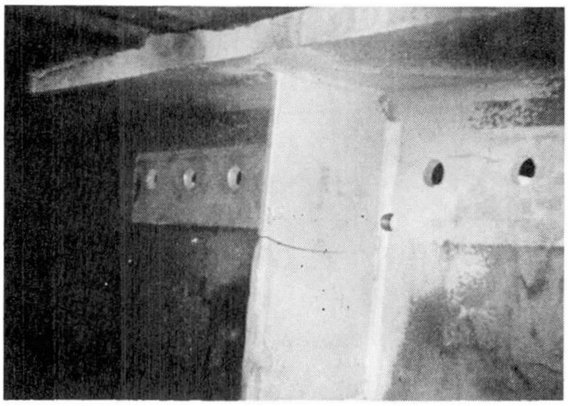

Fig. 11 Hole at Stiffener

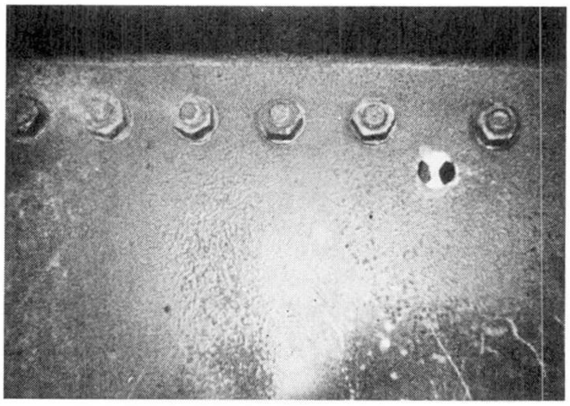

Fig. 12 Other Side

Figure 12 was taken after the two holes drilled from the inside had been reamed to one hole from the outside flush with the stiffener on the inside. The white metal along the vertical centerline of the hole is the stiffener.

In cases where the round hole remains an unacceptable fatigue detail, and where it is physically possible, a high strength bolt can be placed ensuring that the non-burr side of the washer is placed against the steel. The clamping force of the bolt will ensure a better detail and will prevent any further crack propagation should the crack tip have penetrated the far side of the hole. After stopping the crack, attention can then be focused on the repair.

4. RETROFITS

Unfortunately in many cases the only adequate cure is not to have the detail in the first place. If the crack isn't going to go anywhere, and if it has not damaged the structure too severly, it is best left alone as the retrofit may make things worse. Cracks at the bottom of stiffeners tend to be in this category.

Repairing the cracks that occur between the flange and web is extremely important. The general procedure for any crack is to:

1. Use a chisel to vee out the crack,
2. Fill the resulting groove using a proper electrode, and
3. Grind flush.

If the structure cracked because it was restrained from movement and if movement is essential to its function, as in the case of a skew-girder, then a repair of the existing detail will be of no value. In fact it may not crack in the same place next time but at a more serious location. In addition to the obvious differences in vertical deflection of skewed girders, one should consider conceptually that torsion in a girder will occur in the part most capable of rotating. The flange is generally much stiffer than the web, particularly in the space between the stiffener and the bottom flange, and any girder rotation (out-of-plane movement) will be forced to occur in this small space. Needless to say the stresses will be enormous. To solve the problem, the stiffeners must be run down to the bottom flange, or cut back far enough to relieve the stresses, or the source of rotation removed. If the stiffeners are connected to the bottom flange in a high positive moment area, this may entail a reduction in capacity.

At this stage an economic evaluation must be made as to the desirability of altering the detail that causes the crack, or of being prepared to accept that similar cracks will re-occur or worse still that eventually the structure may have to be replaced. For CN's rates of return, it is more ecomomical to leave the detail alone if the repair will last at least 10 years.

Future technology, in particular gas-tungsten arc Remelt [10] may make future repairs easier and more reliable.

In the case of the gussets groove welded to the stiffener, since these developed in less than 5 years, they could be expected to re-occur in the same time frame if repaired. It was decided to replace the butt welded detail with a bolted detail for the brace frames. Prior to doing this, all cracks were repaired by welding and the top and bottom three inches of the stiffener where the gusset place was removed were ground smooth to get rid of any incipient cracks.

In the case of longitudinal stiffeners, if the crack is arrested before it runs, it is usually left alone.

When patching girders, it is a simple matter to run patch plates far enough so that the girder stress will be small enough to permit such a detail.

Whenever a sharp notch is noticed in a member, if it is in a highly stressed area and if it is not on a nearly abandoned branch line, the notch is ground out at the first opportunity. The same should be done for tack welds which are serious stress raisers.

5. REDUNDANCY

Most riveted structures were internally member redundant in that they were fabricated of several components (Figure 3). Their advantage is that cracks do not jump from piece to piece. The chord member of the truss shown in Figure 13 cracked on one side, but not on the other side of the member, and carried rail traffic for some time before detection and repair. In a welded box like member the crack would have propagated all around.

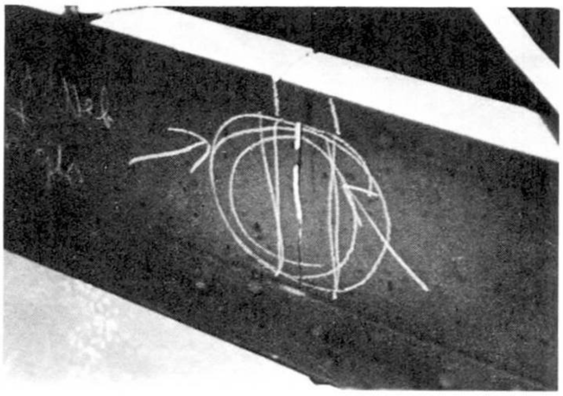

Fig. 13 Crack in chord

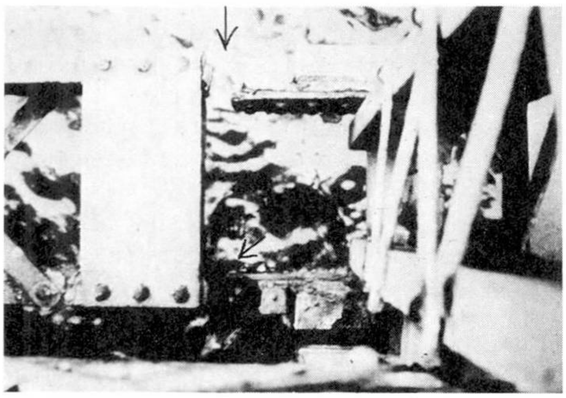

Fig. 14 Chord Severed

Both riveted and welded structures can be redundant by having multiple member load paths as in a multi-beam bridge. Bracing systems can add to redundancy.

Most trusses are multi-load path structures. The truss in Figure 14 had its bottom chord completely severed and yet remained standing because of the alternate load paths provided by the bracing, floor system, etc. The truss shown in Figure 15, which did not have adequate bracing, did not remain standing when one of its diagonals was severed by a shifting load. It is unfortunate that it didn't have sufficient redundancy.

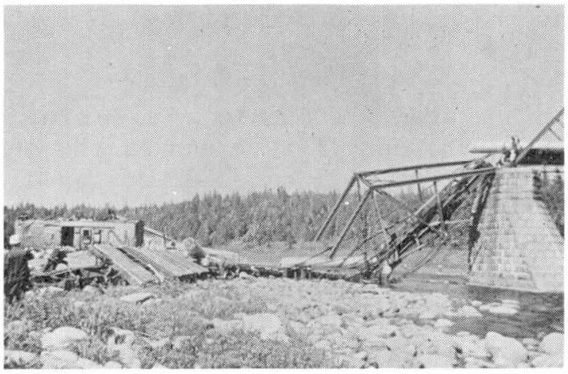

Fig. 15 Insufficient Redundancy

On the other hand, if one considers the deck plate girder bridge, which on railways is generally a two girder system, there is very little member redundancy and virtually no component redundancy in a welded structure. In this type of structure if a crack starts due to an accidental impact, or to a nick fabricated in or due to some less than ideal detail, it may run until the structure fractures [8]. In most cases the bracing will be hard-pressed to carry the dead load, let alone anything like full live load for any length of time. Inspection intervals must be frequent enough to spot these fractures otherwise catastrophic failure will result.

In the typical riveted structure, a crack, from whatever detail, will propagate only within the component that had cracked. For example, in a typical plate girder if a crack starts in a flange angle it will not transfer to the cover plates, web or the opposite flange angle. Another crack may develop, but this will take time and further application of load. Inspection should be able to detect the initial crack and initiate necessary repairs before a serious problem develops. This type of component redundancy permits much greater inspection intervals and permits more time to schedule repairs. It often permits the deferral of repairs until other items combine to make it worthwhile to send in a repair crew.

In order to emphasize the previous discussions, it is instructive to illustrate a few failures where redundancy has saved welded structures or where component redundancy has saved riveted structures.

A rather striking example (Figure 16) is this multi-beam welded structure with a composite deck which was hit by the top of a backhoe. Although several girders were badly damaged, the structure did not collapse under train traffic. It is known that several trains crossed on the adjacent track after the accident. Imagine what could have happened if that had been a two girder non-composite welded system.

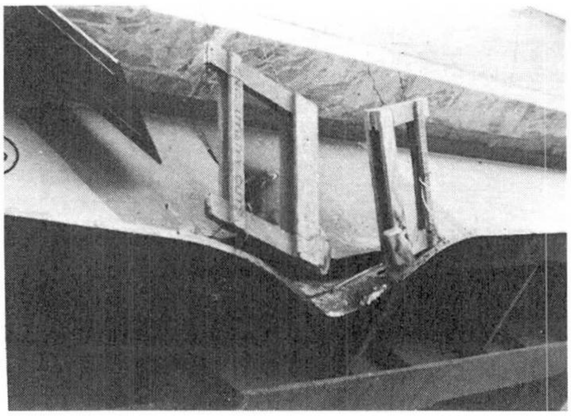

Fig. 16 Hit by Vehicle

The skewed multi-beam composite structure shown in Figure 5 cracked due to a fatigue related failure. Throughout the repairs, which were delayed for over a year, regular train traffic was permitted. This could not have been permitted on a non-redundant load path system.

This (Figure 17) structure cracked due to torsional loading caused by constant train breaking. Although the floor beams are not welded there is no component redundancy as the beams are rolled. Nevertheless, the redundancy of the deck system has permitted regular train operations for over a year with 4 adjacent floor beams failed.

The type of repair shown in Figure 18, the addition of welded plates to an eye bar member could be a disaster in a non-redundant load path structure. The crack is illustrated by the light line of magnaflux at the toe of the weld. In this bridge, because of its multiple load paths, collapse will not occur if one of these members cracks through.

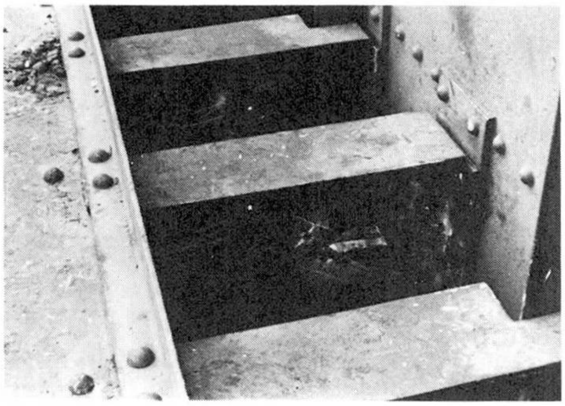

Fig. 17 Torsional loading

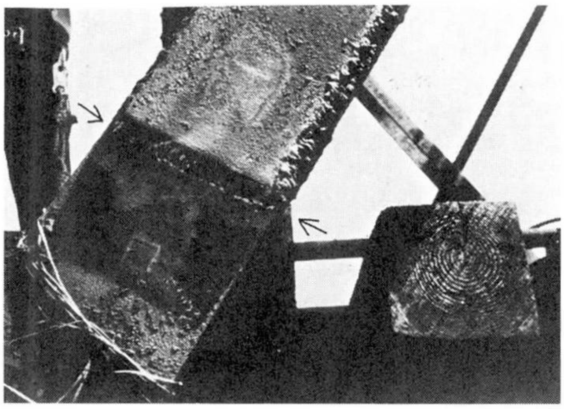

Fig. 18 Eye Bar Weld

With the aid of the new fatigue design rules [5], and a much better understanding of out-of-plane displacements [11], load spectrums, etc., many of our past problems can be covered by design. It must be emphasized that adequate design, particularly for non-redundant load path welded structures means good design, fabrication and inspection.

Our current bridge fracture control plan consists of periodic inspection for component redundant or member redundant spans as these spans may be considered fail safe due to their alternate load paths, and inspection plus live load restriction where necessary on non-redundant load path spans which is a safe life approach.

In the first case we let the structure crack and then take action as the spans are fail safe, in the second case, loads or sufficient cycles of loading that could cause initial cracks to form are not permitted, thus ensuring safe life.

6. FIRES

In a country as vast as ours bridge fires are a serious problem. The problems of getting a structure back into service after a fire are staggering [12]. Locked in residual stresses in apparently simple structures due to the limitations to expansion because of abutments, and other members such as bracing systems can and do approach the yield point. As a case in point the chord of one of CN's trusses cracked (without live load) on cooling after a fire and continued to crack due to daily thermal cycles for the next three weeks. Since many of the railway's structures are supported by open timber cross ties, the mechanical damage when these burn and a loaded car drops down can completely ruin a span.

On a simple three pinned arch [12], the measured residual stresses in the top chord after a fire reached the yield point. Again the internal redundancy preventing large movements was the cause.

As part of our fire prevention control plan, a policy was developed to eliminate major timber structures on main lines. Subsequently, a program of coating timber decks was developed using a propriety fire retardent material.

On those structures which can support the weight, prestressed concrete deck slabs are used to replace the timber decks.

Recently on long structures, the expedient of inserting fire breaks has been considered. On some of our structures in the Rockies, the cross-ties are 0.36 m x 0.56 m (14" x 22") supported by trusses 3.96 meters (13 feet) apart with no intervening floor system. The replacement in kind of this type of deck is costly. With this in mind and in view of the tremendous amount of potential structures, an inexpensive fire break which would assist in the majority of cases was required.

On one region, the expedient of placing an asbestos barrier between spans extending below the rail for a distance of 1.5 meters (5 feet) has been tried. Installation costs are roughly $100 and although the level of protection is not high, it is better than nothing.

Although these measures will not prevent fire damage, it is hoped that they will contain some of it.

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Le comportement en service des ponts dans les Alpes suisses

Brücken in der Bewährung in den Schweizer Alpen

In-Service Behaviour of Bridges in the Swiss Alps

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RESUME
Le comportement en service des ponts est peu influencé par l'action des surcharges extérieures. En revanche, les conditions d'exploitation dont on ne tient souvent pas assez compte — le service hivernal, les grands écarts de température ainsi que les caractéristiques géotechniques du site — sont déterminantes. L'examen des désordres dans une série de cas particuliers est complété par des recommandations à l'intention des constructeurs de ponts.

ZUSAMMENFASSUNG
Das Verhalten der Brücken im Betrieb wird durch äussere Belastung wenig beeinflusst. Massgebend sind hingegen die Betriebsbedingungen — Winterdienst, grosse Temperaturschwankungen und die örtlichen geotechnischen Verhältnisse —, denen oft zu wenig Beachtung geschenkt wird. Die Untersuchung der in einer Anzahl Einzelfällen festgestellten Schäden wird durch Hinweise auf die Gestaltung und Ausführung von Brücken ergänzt.

SUMMARY
The in-service behaviour of bridges is little influenced by the action of exterior load. On the other hand, service conditions which are often not taken into sufficient account, e.g. winter service, sharp temperature fluctuations and geotechnical conditions of the site, are determinant. The examination of damages in a series of cases is rounded off with some recommendations regarding the design and the construction of bridges.

1. INTRODUCTION

Le comportement réel des ouvrages étant révélateur de la bonne tenue ou de la faiblesse qu'accusent les éléments de structures en état de service, son examen permet d'en tirer de précieux renseignements indispensables au projeteur pour être en mesure de concevoir de bonnes constructions. Il apparaît par conséquent indispensable de diffuser des informations sur la pathologie des ouvrages avec bien entendu un certain nombre de réserves et de précautions et lorsque les problèmes dont il s'agit ont été examinés, éclaircis et traités.

Le but que nous nous proposons consiste à examiner les désordres constatés sur une série de ponts dans un certain nombre de cas particuliers significatifs et d'essayer d'en tirer des enseignements applicables pour l'étude et l'exécution de nouveaux ouvrages. L'examen en question comprend l'appréciation des dégâts constatés, l'examen de l'origine des désordres, l'exposé des conséquences pour le trafic ainsi que la descriptions des réparations exécutées, pour la remise en état des ouvrages.

L'analyse des désordres est une tâche difficile et, en même temps, une source précieuse de renseignements sur le comportement des ouvrages et dont il convient de tirer profit. De plus, le caractère des désordres constatés fait apparaître la nécessité de maintenir les ouvrages en service sous une surveillance permanente et efficace qui permet de déceler les désordres et leur origine, ceci dans le double but de limiter l'ampleur des travaux de remise en état d'une part et d'autre part, d'assurer la sécurité du trafic.

Relevons que dans la région des Alpes, en raison des conditions climatiques défavorables caractérisées par un fort enneigement et de nombreux cycles de gel et dégel, la chaussée et les ouvrages sont par conséquent particulièrement exposés aux actions destructives des chasses-neige et des sels de dégel, comme le prouvent les constatations que nous pouvons faire. Mentionnons encore que les désordres en question se rapportent à des ouvrages situés sur le réseau des routes du canton des Grisons. Le service des routes de ce canton nous a obligeamment autorisés à consulter sa documentation sur ce sujet.

Avant de passer à l'examen des désordres, il apparaît indiqué de définir la nature et les caractères des problèmes de pathologie, problèmes qui sont eux-mêmes fonction de l'origine des désordres, de leur importance et de leurs conséquences.

2. ORIGINE DES DESORDRES

Notons, pour situer le problème, deux tendances qui se dégagent de ces informations; la première est le nombre relativement important des désordres imputables aux phénomènes de corrosion, à la fissuration, à l'action de l'eau d'infiltration, à l'effet du gel au service hivernal et, en général, aux conditions climatiques; la seconde tendance à relever est le fait que de sérieux désordres se sont produits sur des ouvrages qui ne sont pas encore très anciens, entraînant même des restrictions de trafic importantes. Les cas des ponts de Crestawald et du Steilerbach sur la route du San Bernardino peuvent être classés dans cette catégorie.

Les causes des désordres peuvent schématiquement être rattachées à trois origines différentes: la conception, les matériaux et l'exécution. Cette classification s'impose pour apprécier le comportement d'un ouvrage. En pratique, cette classification n'est pas toujours facile à appliquer, certains désordres pouvant émaner à la foi de deux ou même de trois origines différentes.

2.1 Défauts de conception

En ce qui concerne les défauts de conception, on peut en distinguer deux types principaux, à savoir:

- le choix de mauvaises options - mentionnons à titre d'exemple les fondations mal adaptées aux déplacements admissibles de la superstructure - et l'exécution de revêtements de chaussées sans une protection étanche du tablier contre les infiltration d'eau et l'effet de la corrosion par le sel de dégel

- un second type de défaut comprend les mauvaises dispositions contructives. Mentionnons notamment les dimensions insuffisantes d'éléments en béton, impropres à permettre une mise en place correcte du béton à l'intérieur des coffrages; l'absence de cheminées de bétonnage; le défaut d'armatures de fissuration; un enrobage de béton insuffisant pour protéger les armatures contre la corrosion. Relevons encore le défaut de drainage des faces supérieures des éléments d'ouvrage - assises d'appuis sur piles ou culées - et d'une façon générale de tous les évidements et spécialement ceux qui ne sont pas visitables. Cette carence conduit à la longue à des dégâts de gel importants. Relevons enfin le choix d'appareils d'équipements mécaniques trop légers et non adaptés aux conditions de service, les joints de dilatation en particulier. Enfin notons encore le défaut de drainage des pots d'encastrement des montants de fixation des glissières de sécurité.

2.2 Défauts des matériaux

Un certain nombre de désordres ont pour cause l'utilisation de matériaux de mauvaise qualité ou simplement inadéquate. L'utilisation de ballast dont la granulométrie ne convient pas - comportant trop peu d'éléments fins par exemple - ou l'emploi d'agrégats trop tendres, poreux ou gélifs, de roches schisteuses contenant des micas - et, partant, impropres à la fabrication de béton non gélif, en constituent des exemples.

Notons que le choix de matériaux convenables pour la fabrication de béton pose des problèmes précisément dans les régions alpestres où l'on rencontre souvent trop de roches tendres dans les agrégats - des schistes et des micas - qui n'ont pas encore pu être éliminés par l'effet du charriage. Les résultats de contrôle de charriage dans le Rhin en amont du lac de Constance sont à cet égard très significatifs puisque la teneur en limon dans les alluvions charriées entre le confluent du Rhin antérieur à Reichenau et son embouchure dans le lac de Constance passe de quelques pour cent à nonante pour cent.

Mentionnons encore les chapes minces du support en mortier de ciment appliquées sur le béton des tabliers de pont. Ces chapes ne résistent pas à l'action de la circulation, elles sont rapidement désintégrées sous l'effet du martèlement des essieux lourds. Les morceaux de chapes sont ainsi dissociés du béton de structure, provoquant la perforation de l'étanchéité, conduisant à sa destruction et à celle du revêtement à plus ou moins brève échéance. Par la suite, le béton de structure et le béton du tablier sont exposés directement à l'effet destructif de la corrosion par le sel de dégel.

En ce qui concerne le matériel de conduites de l'évacuation de l'eau, nous citerons les dégâts de corrosion constatés sur du matériel ne résistant pas à l'action des fondants chimiques.

Pour les appareils d'équipement mécanique, relevons l'écrasement d'appuis élastomères par défaut d'adhérence du matériel de support à l'armature métallique.

2.3 Défauts d'exécution

D'une façon générale, relevons que les défauts d'exécution constatés sur des ponts dans la région des Alpes sont de même nature que ceux qui affectent tous les ouvrages de cette catégorie. Cependant, ces défauts et leur fréquence en particulier exercent une influence directe sur la qualité des ouvrages et, partant, sur le coût de leur entretien.

On peut distinguer entre les défauts d'ordre dimensionnel - écarts supérieurs aux tolérances -; les défauts de mise en oeuvre des matériaux - la présence de nids de gravier ou de fissuration au droit de mauvais joints de reprise avec pour conséquence l'apparition d'efflorescences et de suintements sur les parements -; les défauts de pose d'appareils d'équipement mécaniques, entraînant des blocages d'appuis mobiles ou le martèlement de joints de transition. On a également constaté des déboitements des conduites d'assèchement du tablier avec pour effet une accumulation d'eau à l'intérieur de caissons ou autres évidements inaccessibles et des dégâts de corrosion importants sur le fond de ces éléments.

En ce qui concerne les chapes d'étanchéité, relevons les dommages, les décollements, les soulèvements, les fissurations et perforations, avant tout lorsque leur adhérence au support fait défaut.

3. CAS PARTICULIERS

Le comportement des ponts en service dans les Alpes est moins influencé par les surcharges extérieures, c'est-à-dire par les charges utiles pour lesquelles ils sont dimensionnés que par les conditions climatiques - le déneigement par le sel qui, pour des raisons de sécurité du trafic et de commodité, a tendance à se développer et également à cause des nombreux cycles de gel et de dégel ainsi que des grands écarts de température - dont on ne tient pas compte assez souvent. De plus, les conditions géotechniques du site - le comportement de versants en reptation et l'ampleur des tassements des fondations - exercent dans ce domaine une influence déterminante sur la tenue des ouvrages. Les désordres que nous avons pu constater se rapportent à des ponts situés dans la région des Alpes orientales. Il sont illustrés ci-après par une série de vues de dégâts significatifs qui se sont produits à divers éléments d'ouvrages.

Dégradations des assises d'appuis, salissures et efflorescences sur les murs de culées.
Dégâts dus aux joints de dilatation non étanches (fig. 1).

A noter la disposition des appuis, incontrôlables et inaccessibles.

Pont sur le ruisseau de Suretta à Sufers, altitude 1420 m, N 13c, section Andeer - Splügen. Année de construction: 1959. Les désordres ont été réparés en 1972.

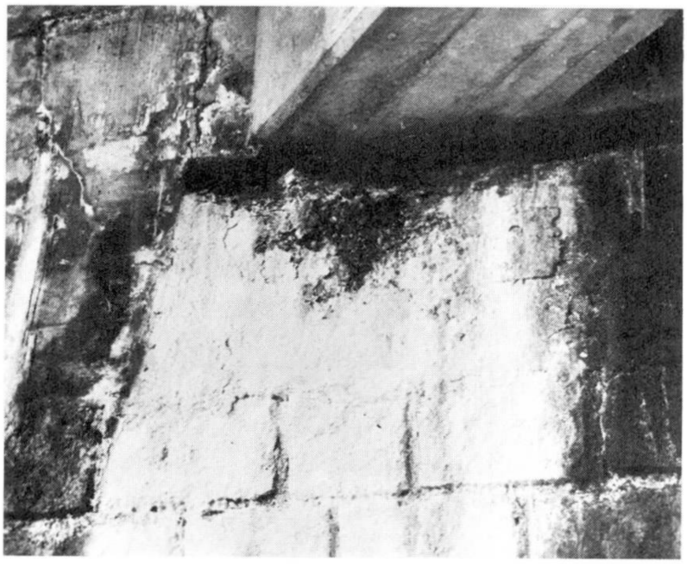


Fig. 1 Dégradations à l'infrastructure

Exemple de corrosion des murs de culée d'un ouvrage par suite de l'action du gel et des fondants chimiques (fig. 2).

A noter que l'eau s'infiltre par les joints de dilatation et que les appuis sont inaccessibles.

Passage inférieur de la jonction de Splügen, altitude 1450 m, N 13c, section Andeer - Splügen. Année de construction 1965. Les désordres ont été réparés en 1970.

Fig. 2 Exemple de dégât de gel

Destruction du revêtement et du béton de structure due a l'action du gel et des fondants chimiques par suite du défaut d'un dispositif d'étanchéité (fig. 3).

Pont de Giustia à Marmorera, altitude 1684 m, route du Julier, section Bivio - Marmorera. Année de construction: 1958

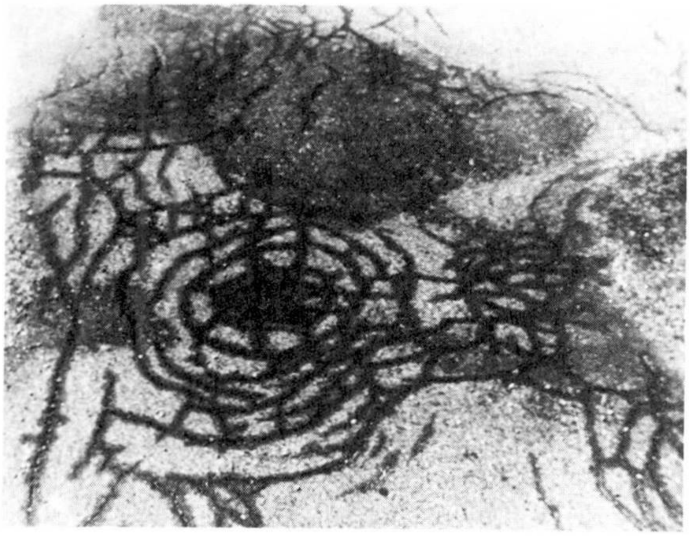


Fig. 3 Destruction du revêtement

La figure 4 montre l'état de la dalle du tablier en béton après l'enlèvement du revêtement en mortier bitumineux.

La remise en état a entraîné la démolition du revêtement en mortier bitumineux, le piquage du béton sur 10 cm d'épaisseur et son remplacement, le renforcement de l'armature de la dalle de roulement et la pose d'une couche d'étanchéité en lès de nylon enrobés d'une masse bitumineuse, adhérente sur le béton de la superstructure. Les désordres ont été réparés en 1974.

Fig. 4 Dégât de gel sous le revêtement

La route du val de Schyn entre Thusis et Davos traverse plusieurs versants instables, entraînant les culées d'ouvrages implantées dans des zones en mouvement.
Un tel déplacement est illustré par l'important décalage - env. 10 cm - entre les bordures de trottoirs sur le joint de dilatation (fig. 5).

Pont du torrent de Cugnal à Sils im Domleschg, altitude 800 m, section Thusis - Tiefencastel, année de construction: 1952.

Fig. 5 Déplacement de culée

La remise en état a nécessité le remplacement des appuis glissants linéaires décalés (fig. 6) par des appuis-pots mobiles en tous sens, les gros efforts dus aux effets d'ordre secondaire ne pouvant pas être repris. Les opérations de remplacement ont été difficiles en raison des mauvaises conditions d'accès aux appareils d'appuis. Les désordres ont été réparés en 1973.

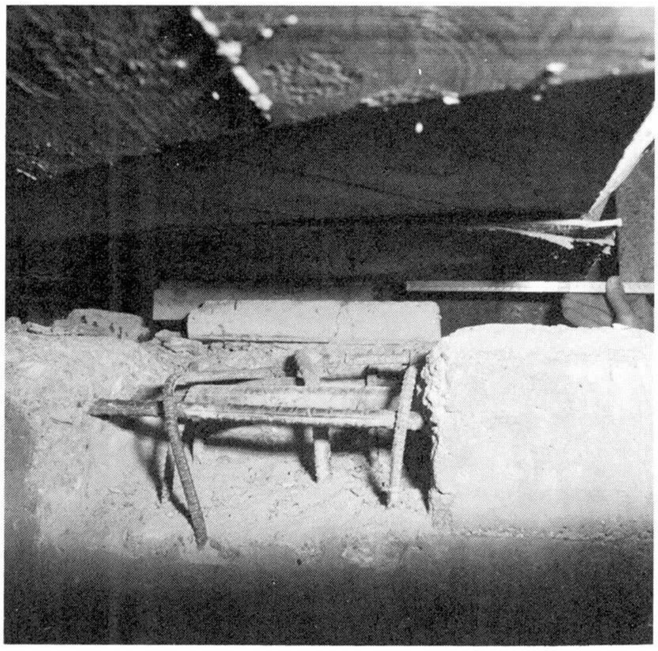


Fig. 6 Appuis déplacés

Destruction de bordure de pont par l'action du gel et des fondants chimiques (fig. 7).

Pont sur le Rhin postérieur à Splügen, altitude 1450 m, N 13c, section Andeer - Hinterrhein. Année de construction: 1962. La remise en état a nécessité le remplacement du cordon et l'application d'une étanchéité de protection. Les désordres ont été réparés en 1978.

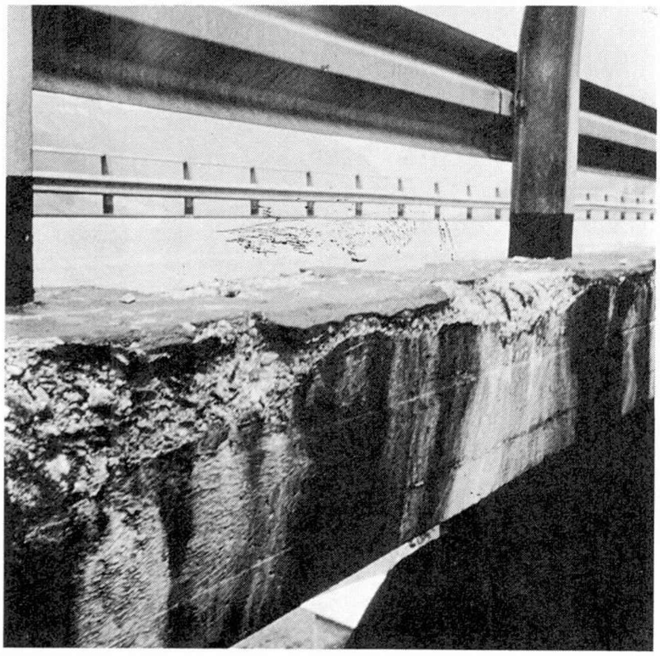


Fig. 7 Destruction de bordure de pont

Dégâts de gel au plancher de caissons inaccessibles (fig. 8). Les désordres sont causés par l'action corrosive des sels de dégel. Par suite d'un déboitement de canalisation, l'eau s'est accumulée dans les caissons. La corrosion du béton a atteint 7 cm de profondeur.

Fig. 8 Dégât de gel au plancher de caissons inaccessibles

Destruction par le gel de l'assise d'appuis glissants; dégâts imputables au défaut de drainage des assises (fig. 9).

Pont du Steilerbach, Sufers, N 13c, altitude 1420 m, section Andeer - Hinterrhein, année de construction 1959. La remise en état, exécutée en 1972/73, a nécessité le renouvellement de l'étanchéité, le drainage des caissons et le remplacement des appuis et des joints de dilatation.

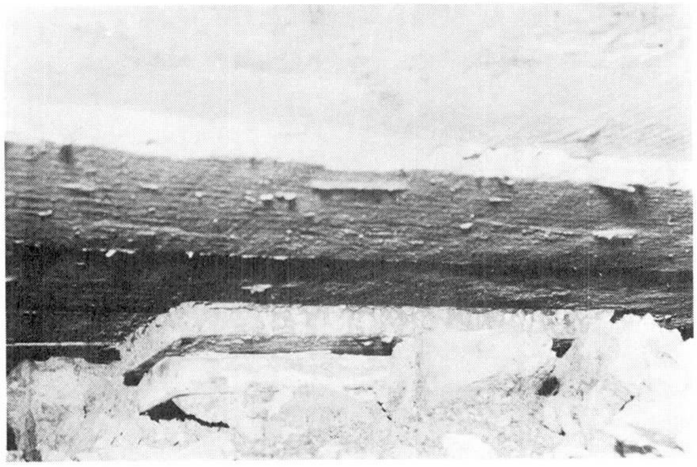


Fig. 9 Désordres aux assises d'appuis

Les joints de dilatation en saillie sur le revêtement sont accrochés au passage par les lames des chasses-neige. Les joints de construction trop légère ne résistent pas aux sollicitations répétées du service hivernal (fig. 10).

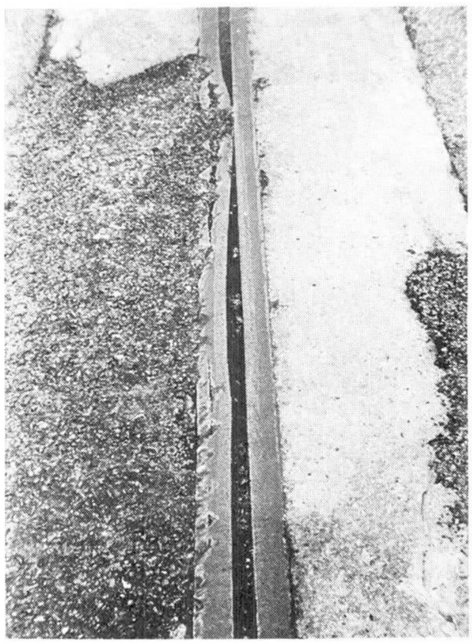
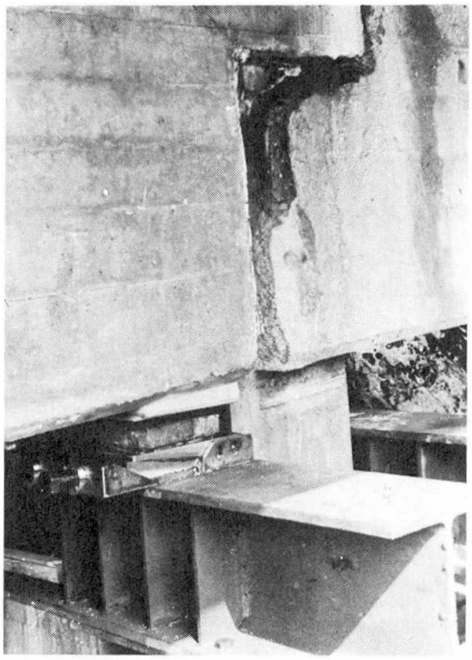


Fig. 10 Joint de dilatation endommagé



En outre, les articulations Gerber du même ouvrage ont été gravement endommagées par suite des infiltrations d'eau de ruissellement pénétrant dans la structure par les ouvertures des joints de dilatation s'ajoutant aux fuites des canalisations (fig. 11 et 12).

Pont de Crestawald à Sufers, altitude 1400 m, N 13c, section Andeer - Hinterrhein, année de construction: 1958

Fig. 11 Détail de l'articulation endommagée

La remise en état de l'ouvrage a nécessité le remplacement des appuis endommagés, la pose de nouveaux joints de dilatation étanches ainsi que le renouvellement de l'étanchéité et du revêtement du tablier. Les désordres ont été réparés en 1972/73.

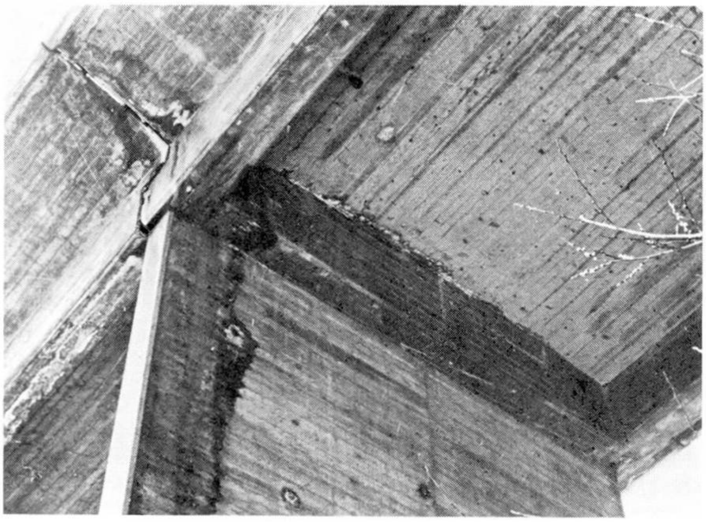


Fig. 12 Vue de l'articulation depuis dessous avant sa remise en état

4. RECOMMANDATIONS POUR L'ETUDE DE NOUVEAUX OUVRAGES

Les recommandations suivantes se limitent à l'énumération des principales mesures à observer en vue d'améliorer la qualité et la durabilité des ouvrages. Elles ne sont pas exhaustives. Pour plus de détails, le lecteur se reportera à la littérature citée en référence, cf. [1], [2], [3].

4.1 Au stade de l'étude

4.1.1 Infrastructure

- On donnera aux ouvrages des formes simples et des dimensions suffisantes, propres à permettre une exécution correcte.

- Le risque de corrosion est particulièrement grand dans les parties d'ouvrage situées aux points de contact entre l'air et l'eau, ou entre l'air et le terrain. Il faut tenir compte de ce fait lors du choix des matériaux et des moyens de protection contre la corrosion.

- Pour assurer aux armatures une protection contre la corrosion, on prévoira un enrobage de béton suffisant.

- Tout enrobage de plus de 50 mm exige que l'on prenne des précautions contre l'éclatement de la surface, p.ex. en mettant en place une armature de fissuration.

- Les faces supérieures des ouvrages doivent présenter une pente continue et suffisante (au moins 8 %). Tous les espaces vides doivent être accessibles; ils doivent donc mesurer au minimum 0,70 m de largeur et 1,50 m de hauteur.

- Les assises d'appui des piles et culées doivent être parfaitement drainées. Il en est de même des zones de transition aux extrémités du pont.

- Les matériaux mis en remblai derrière les culées, murs de soutènement et murs en aile, doivent permettre à l'eau qu'ils contiennent de s'écouler librement sans entraîner de particules de matière.

- Les rigoles et canalisations d'écoulement doivent être largement dimensionnées et aboutir à un exutoire.

4.1.2. Superstructure

Généralités

- Les surfaces qui sont exposées aux intempéries tout en étant dépourvues de protection doivent être aussi limitées que possible.

- L'ouvrage doit être conçu de manière à ce que toutes ses parties soient accessibles aisément et sans frais excessifs, pour les besoins de la surveillance et de l'entretien.

- La transition entre le pont et la route doit se faire à l'aide d'une "dalle de transition".

- Les canalisations d'évacuation des eaux auront un diamètre intérieur d'au moins 200 mm. Elles seront munies, au moins à leurs extrémités, d'une ouverture de nettoyage.

- Tous les espaces libres seront bien drainés et aérés.

Construction massive en béton ou en maçonnerie

- On donnera à toutes les parties de l'ouvrage des formes simples, et des dimensions suffisantes, propres à permettre une exécution correcte.

- Toutes les arêtes du béton doivent être chanfreinées au moyen d'un liteau triangulaire mis dans le coffrage. Il convient d'éviter, dans la mesure du possible, que deux faces de béton forment à leur intersection un angle aigu.
  Les éléments en béton armé doivent avoir une épaisseur d'au moins 0,15 m.

- Les faces supérieures des ouvrages doivent présenter une pente suffisante (au moins 4 %).

- On veillera à ce que les plans d'armature permettent une exécution convenable du ferraillage et une mise en place correcte du béton.

- A l'armature résultant des calculs de résistance, on ajoutera au besoin une armature complémentaire destinée à limiter la fissuration.

- Les barres d'armature et les câbles de précontrainte seront disposés de façon à ménager tous les 50 cm des mailles larges d'au moins 0,12 m, destinées à permettre dans de bonnes conditions la pervibration du béton.

Construction en acier

- Les divers éléments des ouvrages doivent avoir des formes simples et des dimensions suffisantes, de telle sorte que leur fabrication et leur entretien s'en trouvent facilités.

- Dans la disposition des ouvrages en acier, il convient d'éviter les recoins où pourraient s'amasser l'eau et la saleté.

- On choisira la disposition des soudures et l'ordre de leur exécution de manière à limiter dans toute la mesure du possible l'apparition des contraintes internes que la soudure engendre dans l'acier.

- L'improvisation sur le chantier conduit souvent à des dommages entraînant une augmentation des frais d'entretien. C'est pourquoi, au stade de l'étude déjà, l'auteur du projet doit se préoccuper du mode d'exécution et de transport des divers éléments de l'ouvrage.

- Une protection antirouille bien conçue et bien appliquée prolonge la durée de service et réduit sensiblement les frais d'entretien des ouvrages en acier.

- Les surfaces d'acier seront nettoyées au jet de sable ou par un procédé équivalent. La couche de fond de la peinture sera appliquée immédiatement après le nettoyage.

- La couche de fond et les couches suivantes de la peinture doivent être choisies de telle manière qu'elles assurent une adhérence parfaite entre elles. Toute couche de fond qui aura été exposée pendant un certain temps aux intempéries devra être soigneusement débarrassée des parties altérées (p.ex.: du carbonate de zinc).

4.1.3 Tablier et chaussée

- La surface de la dalle du tablier des ponts-routes doit présenter en chaque point une pente nettement marquée (2,5 % au minimum, exceptionnellement 2 %).

- L'ouvrage et en particulier les éléments exposés aux intempéries doivent être protégés par un revêtement étanche pour empêcher les infiltration d'eau ou de solutions agressives.

- Dans les ponts-routes, les parapets et les parties en encorbellement sont particulièrement exposés aux effets des intempéries et du sel antigel. On prendra toutes mesures pour que le béton de ces éléments soit spécialement résistant.

4.1.4 Appuis, articulations, ouvrages de transition

- Les piles, les culées et le tablier doivent être construits de telle façon qu'il soit possible en tout temps de remplacer les appuis défectueux. Ces dispositifs doivent être clairement indiqués dans les plans d'exécution.

- Les appuis et articulations doivent être surélevés de manière à empêcher autant que possible la pénétration de l'eau. Ils doivent être aisément accessibles de tous les côtés afin d'être faciles à surveiller.

- Les éléments exposés à l'usure doivent pouvoir être remplacés facilement.

4.2 Au stade de l'exécution

On utilisera des agrégats non gélifs. Les principaux facteurs qui affectent la durabilité du béton sont avant tout sa composition (y compris les adjuvants et en particulier les entraîneurs d'air), un temps de malaxage correct, le coffrage, le compactage, la cure, et l'enrobage suffisant des armatures. Les bétons de construction fortement exposés au gel et aux sels de dégelage exigent en outre des mesures de protection supplémentaires pour garantir une bonne durabilité. Il s'agit notamment d'imprégnations, de peintures, d'enduits ou de glaçages. Dans chaque cas, le problème crucial est d'obtenir une liaison suffisante avec le béton.

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The Long-Term Behaviour of Viaducts Subjected to Heavy Traffic and Situated in an aggressive Environment. The Viaduct on the Polcevera in Genoa.

Comportement à long terme de viaducs soumis à un fort trafic et dans un environnement agressif. Le cas du viaduc de Polcevera à Gênes.

Langfristiges Verhalten von Brücken unter schweren Lasten und in aggressiver Umwelt. Der Polcevera Viadukt in Genua.

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SUMMARY
After a brief description of the common causes of wear to which a reinforced concrete bridge is most easily subject, the so-called wall cracks phenomenon is considered in detail and one limit case of a viaduct in Italy is described. The state of repair of the Polcevera Viaduct at Genoa (Italy) has also been examined, for which, while the static behaviour of the various members is absolutely normal, there is perplexity about the aggressivity of the local atmosphere on the concrete.

RESUME
Une brève description est faite des causes habituelles de la dégradation subie communément par un pont en béton armé: le phénomène de fissuration superficielle est décrit dans le cas particulier d'un viaduc en Italie. L'état du viaduc de Polcevera, après réparation, est examiné; si le comportement statique des différents éléments est absolument normal, il y a des doutes au sujet de l'agressivité de l'atmosphère locale sur le béton.

ZUSAMMENFASSUNG
Die üblichen Gründe der beginnenden Zerstörung, die bei Stahlbetonbrücken auftritt, werden erörtert. Es handelt sich um Risse an der Betonoberfläche, die im Falle einer Brücke in Italien beschrieben werden. Der Stand nach Wiederherstellungsarbeiten am Polcevera Viadukt in Genua wird beschrieben; das statische Verhalten der verschiedenen Bauteile ist einwandfrei; Zweifel bestehen über die schädliche Einwirkung der Atmosphäre auf den Beton.

Some decades spent in designing, directing and supervising reinforced concrete bridges constructions authorize me to express some opinions about their durability and the frequency of repeated inconvenientes which may occur in the course of time. I shall try to make a synthetic classification of such inconveniente and I shall conclude by reporting the behaviour of two structures, both built and in operation for several years, one with normal and the other with exceptional characteristics. I have chosen them amongst many other because they may arise interesting observations.

As it is well-known, a reinforced concrete bridge, apart from possible troubles due to specific static deficienses, is subject to a slow deterioration because of :

- the effect of movable loads and of the environmental action, especially on the paving, on the supporting structures, on the joints and on the finishes,
- the chemical and mechanical effects due to the metereological action on the concrete and also on the reinforcement.

We must consider, in a particular category, some special phenomena such as the appearance of diffused cracks (the wall cracks) partly due to the insufficient stretching of the concrete compared to that of the steel (when this is subjected to high unit stresses), partly to vibration caused by traffic and partly to an un uneven distribution of the reinforcements within the concrete mass.

In fact, it is well-known that normal reinforced concrete members subjected to bending and shear (especially under the effect of dynamic loads) tend to develop cracks in the course of time, even when design or technological errors must be ruled out.

Let us consider in particular the so-called wall cracks, i.e. those vertical cracks diffused almost all-over the surface of the girder and closer to each other in the intermediate areas between two adjacent supports.

Very often such cracks do not reach the main-steel reinforcements, in other words they remain small and superficial, but give rise regularly to a state of alert, to claims, and to the suspicion that there are defect which will appear in the course of time.

In other words this is a very frequent phenomenon and the positive elimination of it (in consideration of all the causes which contribute to produce it : stresses caused by external loads, by temperature changes, by shrinkage) would require us to introduce in the beam such a quantity of distributed reinforcement to jeopardize the economic conditions of the use of the structure, especially in countries where steel is particularly expensive.

In rather recent times it has been agreed to introduce the concept that the phenomenon of the appearance of cracks could be accepted as a natural behaviour of the structure, as long as it would not cause a decrease in the performance capacity of the structure, even in the long run.

Therefore, it has been agreed to proceed with a series of theoretical and experimental tests in order to find out the maximum crack width (after taking into account the various environmental circumstances) below which the structure would appear fit for service.

The aforesaid maximum values appear by now to have been specified in the codes in force for reinforced concrete structures ; therefore it should be easy, by now, to overcome the worries of the layman (followed in most cases by lawsuits and surveys) when he discovers even a small crack, which he immediately associate with the idea of the crumbling down of the structure.

The study of the determination of the aforesaid maximum crack width tends however to become ever more complex : we have noticed that it is not enough to take into account the maximum working load since we notice more and more that the agreement between the theoretical and actual behaviours of a structure is greater, especially as regards the cracking, if greater has been the investigation on the ratio between the permanent and live loads, on the period of live load permanence and especially on its fluctuating behaviour.

All this, as said beforehand, must be added to the stresses due to prevented geometrical variations under the effect of temperature changes and shrinkage.

Here, however, we must make clear an important point :
The determination of the cracking state of a structure, i.e. the determination of the extent and position of the cracks, may obviously lead us to two different conclusions : if all the cracks are hypothesis or environmental condition (cracking below a certain set limit) in such a case, at least in this respect, the structure is fit for service even in the long run. On the other hand, the structure may show crack openings exceeding the maximum value accepted in design or recognized as acceptable at the time of the survey. In this second case, as a rule, the cracks can, in the long run, cause damage to the preservation of the reinforcement because of the infiltration of humidity or other things and therefore it will be necessary to seal the wider cracks by means of gluing materials and also, in more serious cases, to cover the external surfaces by means of suitable elastomers.

The above, of course, should be done after a through survey by means or direct and indirect tests carried out in order to detect whether the cracks may have damaged the static working capacity of the structure. And, to conclude the matter of the cracks, all what has been said has of course no meaning whatsoever when the structure is subjected to prestress.

In such a case it is enough to think that, for a prestressed beam, the determination of the cracking condition has a very different meaning : in such a case it is not a question to limit cracking to acceptable values but rather to make all the necessary investigations in order to avoid cracking altogether within the limits of the assumed working conditions of the structure.

In this second case, therefore, the fact that cracks are or are not present means the absence or the presence of a defect in design or execution.

Therefore, the presence of cracks in a prestressed structure is a much rarer thing but, as a rule, a much more serious one which requires in most cases an immediate intervention.

On the other hand, in a prestressed concrete structure, the intervention against degradation phenomena caused by environmental factors on the concrete may be more important for obvious reasons, both for the necessity to protect the prestressing cables and to prevent the reduction of the resistant section of the concrete, which usually is rather small.

From all the above, which is quite well-known, the necessity arises which is felt more and more as the technique and technology for reinforced and prestressed concrete becomes more sophisticated, to keep the structures under careful observation as time goes on, to decide about possible remedial works which, in any case, must be carried out quickly and properly.

I know perfectly well that all what I have just briefly summarized is very well known by all those who deal with this subject.

However, I think it will be interesting to bring as an example some extreme cases in which first the survey and then the intervention have been or will be necessary.

The examples refer to structures built and in operation since no less than 12-15 years.

A REINFORCED CONCRETE VIADUCT SERVING A SPEEDWAY IN NORTHEN ITALY.

Of all the cases I had an opportunity to examine, the one I am going to illustrate is perhaps the most difficult to explain thoroughly : since the structure was part of an important super highway it is obvious that, besides any more or less learned considerations on the causes of the phenomena encountered, it was absolutely necessary to answer the imperative whether the structure could or could not remain open to traffic without any limitations whatsoever.

In 1967 I was charged by the Agency of the Italian National Roads with the control of the static conditions of a speed-way viaduct, designed by others, opened to traffic in 1969, consisting of dou ble independent decks (one for each traffic direction) for a total of 36 simply supported spans of average length 19.00 lm, in conventional reinforced concrete.

Figure 1 illustrates the geometry of the standard bay.

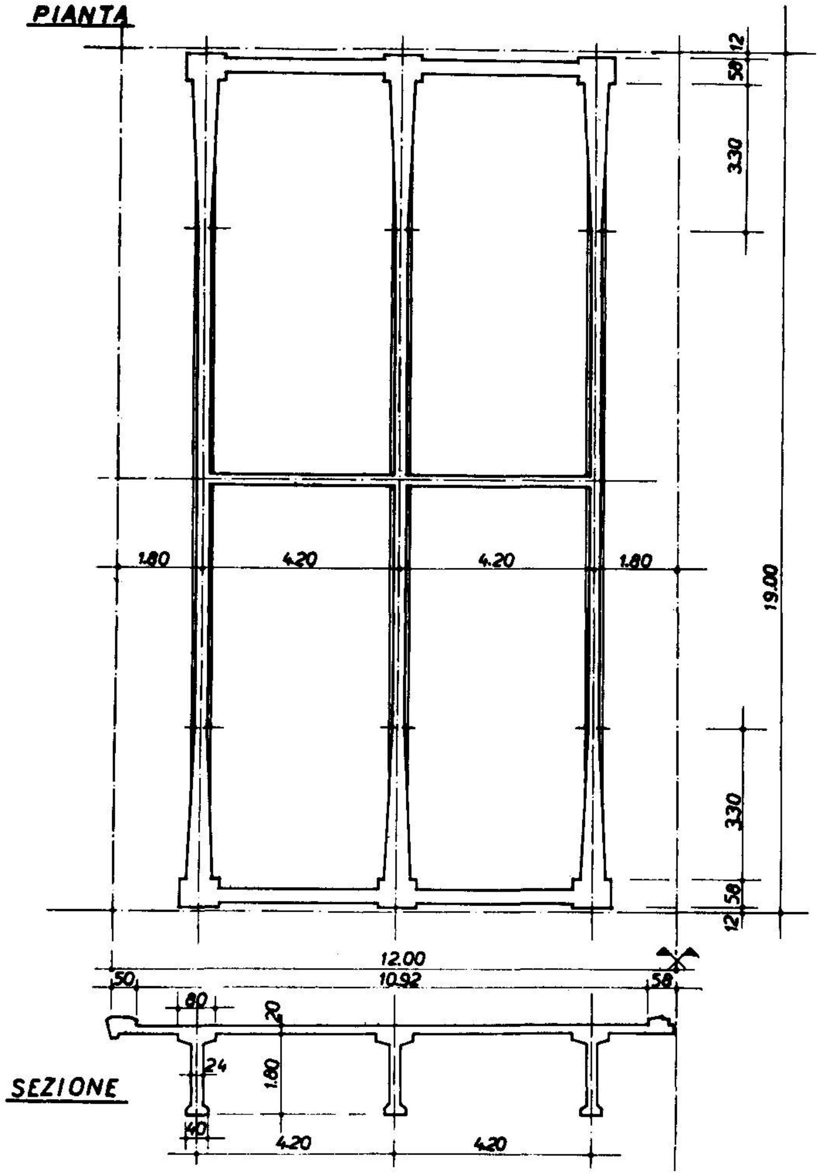

Fig. 1 - Geometry of the standard bay.

During my visit I found some superficial distress in the paving joints, some excessive crushing of the bearings (although nothing really serious) but, most important of all, that all the parallel beams of the deck showed a diffused pattern of cracks of very similar length, frequency and width on all the bays.

As an example, figure 2 shows the cracks of the external rib of bay No. 21.

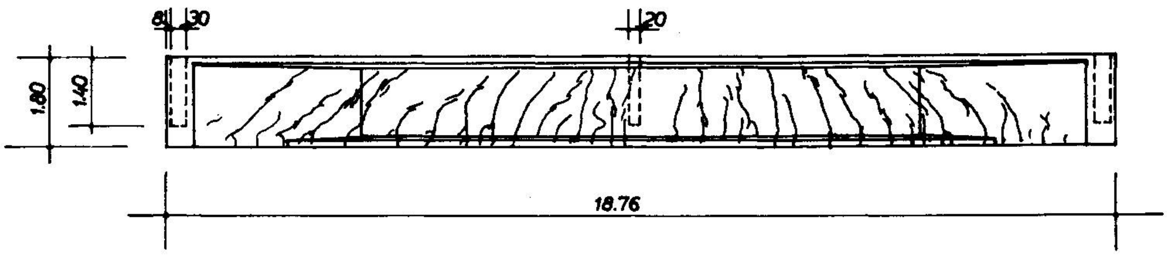

Fig. 2 - Crack pattern on the external rib of bay No. 21

As for the crack width, we can briefly conclude that the inclined cracks near the bearings of the deck reached the maximum width of 0.5 mm, while the width of all the others were between the 0.05 and 0.3 mm, with most of them being less than 0.2 mm.

Having verified and carefully inspected all that, I examined all the documentation in order to single out all the detail characteristics of the project. From that I found out :

- that the forces and the actions taken in consideration in design were in accordance with the existing regulations and with the generally accepted practice,
- that the placement of the reinforcement as resulting from design had been done in a perfect way, except for some imperfections in the positioning of the bent-up bars, which however was unimportant and could not have caused (only by themselves) the cracks I had noticed,
- that the unit stresses of the materials were not over the allowable values,
- that the characteristics strength of the materials were in accordance with those prescribed in the project,
- that the theoretical verification of the width of the crack showed that it should not have exceeded the value of 0.2 mm.

Have done all this, it was necessary - of course - to answer the basic question whether and to what extent the structure, was damaged with respect to its functional capacities, and especially if its safety coefficient had to be regarded as unduly decreased.

The answer to the above question would permit to reach a decision on whether a strengthening of the structure was necessary or a protective action was sufficient so as to avoid that the crack ope -

nings, with the time going on, would damage the internal reinfor cement.

We have preferred to subordinate such important decision to a se cond stage of investigation carried out to establish :

- the causes of the cracking phenomenon, in order to see whether it would have been possible to eliminate them to avoid a further degradation,

- the assestment of the performance conditions by means of a series of tests.

As for the first research (the determination of the causes of such an important cracking phenomenon), we found out after a long series of studies that many factors had occurred accidentally at the same time which could explain the cracking.

The most important of them may be listed as follows :

- the very small ratio beatween the beam width and their height,

- the use of a cement which had not been seasoned long enough in silos, with a consequent increase of the shrinkage value.
The faulty performance of the bearings with consequent temperatu re stresse arisen in the structure.

- Very heavy and fast traffic : the structure is located near an important marble production centre,

- the imperfect position of the bent-up bars for shear reinforce ment.

As for the second question, i.e. whether and to what extent the safety of the structure was effected, we have carried out a series of tests organized as follows :

a) Test of three bays chosen amongst those showing more cracks, with a static load equal to 120 % the maximum design value.
Period of stay of the load : 24 hours.
Determination of the variation of the geometry of the deck and of the size of the cracks. Reversebility characteristics of the above variations. Determination of the average apparent elastic modulus.

b) Testing of the same three bays by means of a vibrating apparatus (vibrodine) for the determination of the dynamic behaviour characteristics and consequent determination, by other means, of the apparent elastic modulus of each deck.

In short, the results of the tests have been the following :

The three tested bays have shown a practically identical behaviour, with almost unnoticeable "dispersion". Such behaviour has appeared to be stable and reversible, not showing any signs of deteriora - tion and unelastic deformations.

The deflections and deformations of the structures have appeared to be realiably estimated by means of an elastic calculation model, assuming a modulus of elasticity equal to about 300,000 kg/cm2 and with begligible residual deformations. In particular, with regards to dynamic tests, the values of elasticity modulus obtained from the comparison between the fundamental experimental frequencies and those calculated theoretically appear to be practically identical to those resulting from the static tests.

Finally, the behaviour of the structures under the dynamic impulses shows a satisfactory level of integrity of the entire assembly. Obviously, after having considered all the above results, we reached the conclusion that it was sufficient to apply coats of various substances (elastomers) on the external surfaces in order to prevent the reinforcements to be reached and damaged by air through the cracks.

I repeat that I wanted to quote this case (to be regarded as rather emblematic) amongst so many other ones, because here a through study has prevented to make recourse to unnecessary and costly interventions or, even worse, to demolition or structural repairs.

May be it will be useful that the experts explain to the laymen that the cracks in a reinforced concrete beam are, within certain limits, a normal phenomenon and should not be considered as the warning sign of a coming disaster.

THE VIADUCT ON THE POLCEVERA, FOR THE GENOA-SAVONA SPEEDWAY.

I shall deal now with one of the biggest reinforced concrete structure built and in operation for more than ten years, which appeares to be surrounded by a particularly aggressive environment.

The viaduct on the Polcevera in Sampierdarena (Genova) marks the junction between two of the most important italian speed-ways, i.e. the Genoa-French Border and the Genoa-Po River valley, crossing a valley in a heavily built-up area with civil and industrial buildings and also including, besides the Polcevera river by a series of very important railway yards.

Therefore, the structure in the whole may be regarded as an example of a big infrastructure within a thick urban and industrial network.

The structure may be subdivided into a main viaduct and four approach lines, the latter being arranged in different ways, altimetrically and planimetrically.

The main viaduct has the following theoretical spans :

- one 43.00 m span
- five 73.20 m spans
- one 75.313 m span

- one 142.655 m span
- one 207.884 m span
- one 202.50 m span
- one 65.10 m span

The spans, of such a different length, find their conceptual link in a series of prestressed concrete decks, all of the same span 36.00 m long, simply supported by a series of special systems, amongst which we may distinguish the following two different basic types :

- The system supporting the amsller spans, consisting of two inclined piers connected at the top by a double cantilever girder of variable length. The whole in reinforced concrete, carried by a foundation raft which in turn rests on drilled piles 110 cm in diameter of a length variable up to 40.0 m.

- The balanced system for the main spans. Such system consists of a three-span continuous girder resting on four supports, with two end cantilevers giving support to the above said 36.00 m beams. The two external support of the three-span girder are provided by the anchorages of two prestressed stay-cables passing over a mast (sospension tower) located on the axis of the system. The mast top is 90.00 m above the ground and about 45.00 m over the roadway deck.

Each balanced system consists of :

1. A reinforced concrete ribbed foundations raft resting on drilled piles 150 cm in diameter.

2. A special reinforced concrete trestle consisting of four "H" shaped bents laid side by side and connected to each other by cross elements. The tops of the trestle give elastic supports to the deck girder.

3. A mast, or suspension tower ("Antenna") made up of four inclined legs with adequate connections in both directions (longitudinal and transversal) so as to form a true and proper frame, but such as to keep independent the tower itself from the trestle-deck sistem.

4. A continuous deck-girder of prestressed concrete, of cellular type, with top and bottom slab and six longitudinal ribs, resting on the trestle referred to under paragraph 2. The connection between the deck and the stay-cables is achieved through a stiff cross girder, also in prestressed concrete, whose projections on each side of the deck provide the anchorage of the two stay-cables passing over the top of the mast at an elevation of 9,000 m above ground. Later on, concrete shells were poured around the cables ; the function of these shells, as it is known, is, besides protecting the steel, also to reduce the cable elongation at the passage of moaving loads because the shells themselves have been prestressed.

Figure 3 - General view of the structure.

The work has been completed in 1966 and was regularly opened to traffic in 1967. Since then it underwent a series of controls about its state of preservation.

Essentially, it has been ascertained that the structure stands the very neavy traffic to which it is continuously submitted without signs of deterioration or static unadequacy.

The balanced systems are behaving in a regular way and obviously, as far as they are concerned, we must not worry about cracks because we deal with structures the parts of which are practically all in compression under the effect of external loads or prestressing.

Some slight cracks of a very'small size (much below 0.2 mm) have been noticed in the secondary connecting cross elements - which obviously had not been prestressed - and which were surely due to the vibration caused by the traffic.

On the other hand, no cracks are noticed on the horizontal elements of the secondary unprestressed "V" piers.

This, evidently, is due to a sufficient distribution of the reinforcement within the concrete and to a non-exceptional shrinkage of the concrete.

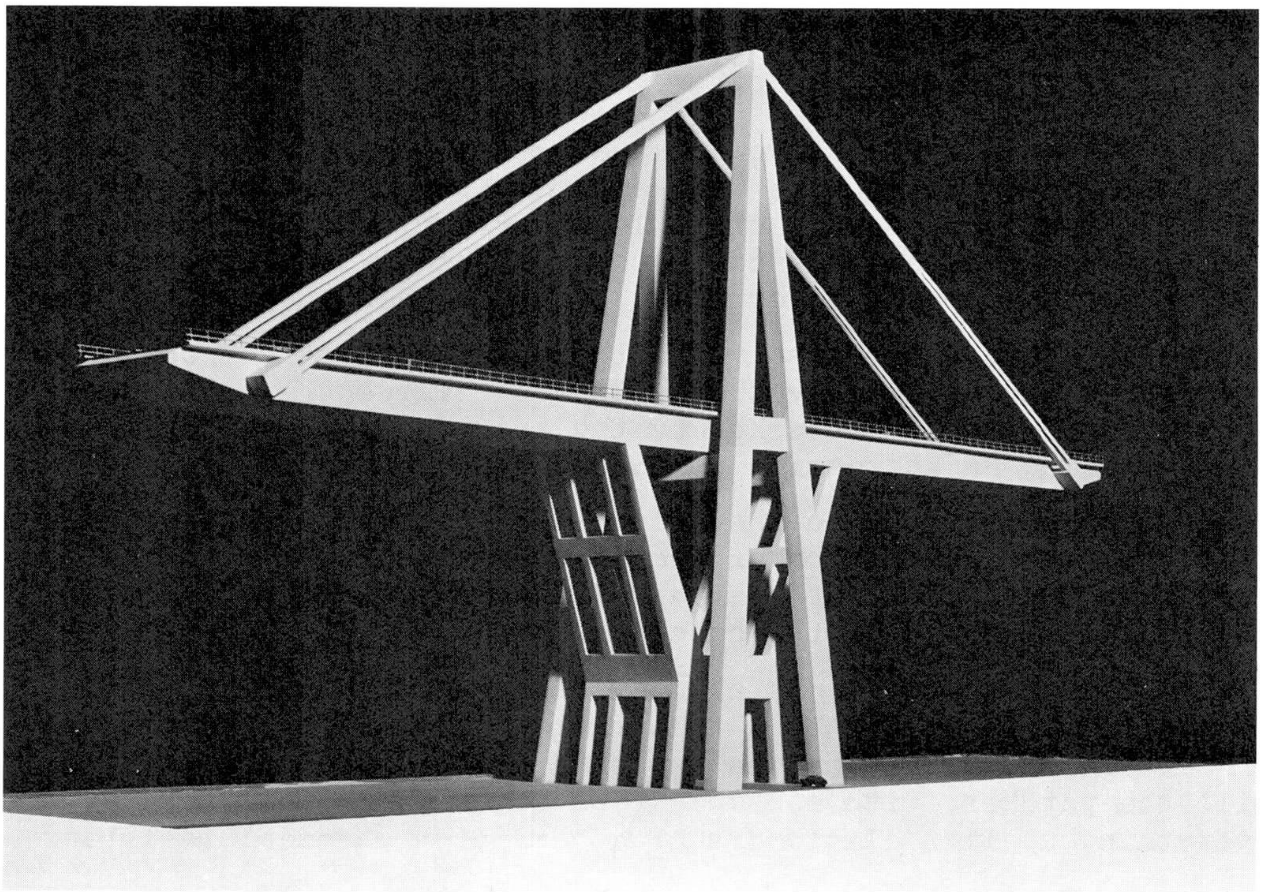

Figure 4 - Photo of the model of a balanced system.

Certainly, in this case, the behaviour of the structure is so different from that we have seen in the vuaduct given as first example that it confirms for the latter the influence of a series of concomitant causes, as it has already been pointed out.

On the other hand, the atmospheric aggressiveness is what represents a definitely negative environmental condition for this structure.

It appears that the structure is struck directly by the marine winds (the sea is about 1 km away), which are canalized in the valley crossed by the viaduct.

Therefore it is a highly saline atmosphere which also finds, on his ways before reaching the viaduct, a curtain of fumes from the chimneys of steel mills and therefore becomes saturated with highly noxious vapours.

In the whall structure, besides some small imperfections of execution which caused some small rust spots to appear on isolated areas due to insufficient end cover of reinforcement, the project has carefully placed within the concrete all steel elements, except, of course, the cadmium-lined plates of the bearings for the simply supported girders.

All these plates have been literally corroded in little more than five years by the extreme aggressiveness of the atmosphere and had to be substituted, with rather complicated processes, with stainless steel elements.

We must think about what would have been the maintenance costs if, instead of a structure made entirely of concrete, a steel solution had been adopted or at least if the solution of the stay-cables embedded in a concrete shell under compression, and therefore not subject to cracking, had not been adopted.

Furthermore, in these last years the external surfaces of the structures and especially those exposed towards the sea and therefore more directly attached by the acid fumes of the chimneys, start showing an aggression phenomenon of a chemical origin.

This is obviously due to the production of soluble salts resulting from the combination of the acids of the fumes with the free lime of the concrete : the well-known loss of superficial chemical resistance of the concrete itself.

I thinck that sooner or later, may be in a few years, it will be necessary to resort to a treatment consisting of the removal of all traces of rust on the exposure of the reinforcements, to fill the patches, with epoxidic type resins and finally to cover everything up with elastomers of a very high chemical resistance.

In conclusion, to sum it up, I wanted to point out with the two examples illustrated above (chosen as border cases amongst many other ones) that, for reinforced concrete structures destined to stay outside, their preservation in the course of time - besides any trouble due to static insufficiency - is subordinated not only to the protection of the reinforcement and therefore the big worry about the effects of cracking, but also to the aggression of the external surfaces of the concrete and this is particularly important under special environmental conditions.

It is also suitable in this case to provide some protection over the external surfaces of the structures in order to increase their chemical resistance and, if necessary, the mechanical resistance to abrasions.

This is especially true for the big infrastructures for which the following interventions will entail at the end very heavy burdens.

Restoration and Widening of the Tasman Bridge

Remise en état et élargissement du pont Tasman

Wiederherstellung und Verbreiterung der Tasman Brücke

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SUMMARY
Three out of 22 river spans of the Tasman Bridge were destroyed by ship collision. The paper describes the method of debris survey in the deep water site conditions and the method of restoration determined therefrom. The unusual engineering tasks involved in the work included temporary repairs, demolition, major steel falsework, unusual vertical piling, the incorporation of a long steel span into the prestressed concrete bridge and erection techniques used. The whole bridge was widened concurrently with the restoration.

RESUME
Trois des 22 portées du Pont Tasman furent détruites par suite d'une collision d'un bateau avec les piles du pont. Le rapport décrit la méthode d'examen des décombres et la méthode qui en résulta pour la remise en état. Les tâches inhabituelles que l'ingénieur rencontra ici comprenaient: des réparations temporaires, démolition, des échafaudages importants en acier, la mise en place de pieux verticaux, l'incorporation d'une longue portée en acier dans le pont existant en béton précontraint ainsi que les techniques de montage utilisées. A l'occasion de cette remise en état, le pont a été élargi dans toute sa longueur.

ZUSAMMENFASSUNG
Von den 22 Flussöffnungen der Tasman Brücke wurden drei durch einen Schiffsstoss zerstört. Das Referat beschreibt die Methoden der Schadensfeststellungen im tiefen Wasser sowie die gewählten Wiederherstellungsmassnahmen. Zu den aussergewöhnlichen Ingenieurarbeiten gehören unter anderem: provisorische Reparaturen, Abbruch, das Erstellen eines Stahlschalgerüsts, ungewöhnliches Bohren von Pfählen etc. Die ganze Brücke wurde gleichzeitig saniert und erweitert.

1. INTRODUCTION

The Tasman Bridge (Fig. 1) crosses the River Derwent and provides the only road link between the eastern and western shores of the City of Hobart which is the capital of the southern Australian State of Tasmania. The bridge was constructed in 1959-64 and design and construction of it was reported by TROLLOPE /1/, BIRKETT /2/ and NEW /3/. It is about 1.5 km in length and when opened in 1964 provided a four lane high level crossing of the river. In 1975, 45,000 vehicles per day were using the bridge and between 1964 and 1975 there had been about 12,000 ship transits.

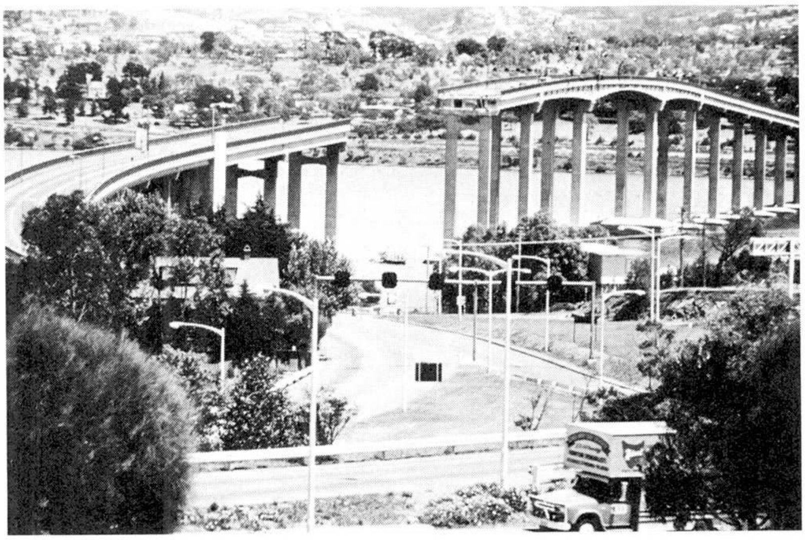

Fig. 1 Tasman Bridge after collapse

At the bridge site the river is wide and deep, water depths generally being in excess of 30 m. The bridge itself consists of three separate structures (Fig. 2), the navigation spans in mid river, 13 x 42.7 m spans in the western main viaduct and 6 x 42.7 m spans in the eastern main viaduct. The original design recognised the possibility of a ship collision although the probability of this was thought to be very small. Accordingly the following provisions were made.

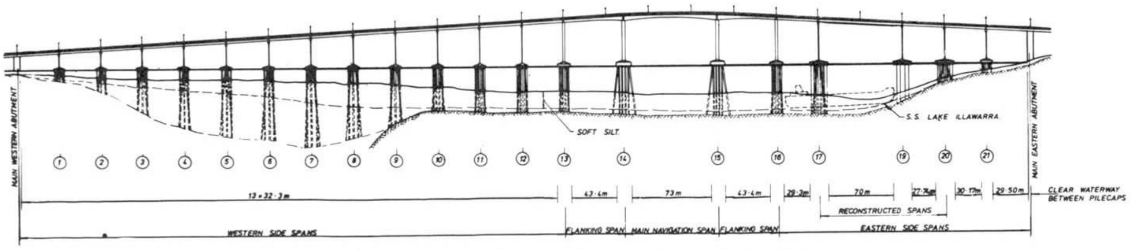

Fig. 2 Elevation of restored bridge

- Gravity fender protection to the navigation span piers.

- A special deck continuity detail at the main viaduct piers, so that in the event of a pier being knocked over the adjacent spans would break away cleanly and fall without damaging the remainder of the bridge.

- Stay bolts between the navigation spans and main viaducts, to provide longitudinal support to the latter in the event of the collapse of some viaduct spans due to a ship collision.

In January 1975, the ore carrier S.S. Lake Illawarra, fully laden and displacing 17,000 tonnes, collided with Piers 18 and 19 of the bridge bringing down these piers together with three spans of the eastern main viaduct and leaving a gap of 128 m. The centre one of the three spans fell on the bow of the ship which quickly sank, the tragedy resulting in the loss of twelve lives. The severence of the only road link also caused quite substantial social disruption, as the eastern shore has mainly dormitory suburbs, with the majority of the city's infrastructure being on the western shore.

The consequences of the accident to the remainder of the bridge were relatively minor as it had performed in accordance with design provisions. Span 16-17, the only remaining eastern main viaduct span on the river side of the gap, had remained in position supported by the stay bolts. Pile Cap 20 had suffered severe damage when it was hit by the falling span 19-20, whose trajectory of free fall was intercepted by one end hitting the bow of the ship. The columns on either side of the opening were deflected towards the gap at the top by up to 200 mm and the lower part of the columns was severely cracked. Otherwise the remaining 19 spans of the bridge were undamaged.

Subsequently a Court of Marine Inquiry /4/ determined that the cause of the accident was an error in navigation.

As the majority of the bridge was intact, the decision to renovate rather than rebuild was a straight forward one, as not only would it be much cheaper but also very much faster, the latter being of prime importance under the circumstances.

2. ADMINISTRATIVE AND CONTRACTUAL ARRANGEMENTS

The complicated physical situation following the disaster was compounded equally by a complicated administrative situation with overlapping jurisdiction by many authorities concerned with the bridge, the ship and the port. This situation was overcome and the work expedited by the formation of a separate authority, the Joint Tasman Bridge Restoration Commission, whose responsibility covered both the bridge and the ship. This has been reported on by KNIGHT /5/.

One of the early and most important decisions reached by the Bridge Commission was that an immediate attempt to salvage the ship was not the best course, that priority should be given to the bridge restoration and that the restoration work should be so devised as to avoid substantial removal of major sections of the ship.

The Bridge Commission appointed Maunsell and Partners Pty. Ltd. as its consulting engineers and the State Government appointed a leading Australian contractor, John Holland (Constructions) Pty. Ltd. to be responsible for the bridge work. The basis of Holland's contract was a mixture of dayworks performed by their own plant and site labour, together with project management provided by them for the procurement of materials and elements fabricated off site. Design proceeded in conjunction with construction as time was not available to do it before work commenced. Design liaison between the consulting engineers and contractor was excellent and this was of considerable advantage in ensuring that design detailing matched construction techniques to be employed.

3. DEBRIS SURVEY

Investigations revealed that the bridge could not be restored without additional piling. Due to the 35 m height of the deck, even a single span truss solution to span the 128 m gap would need additional piles to resist transverse wind loads. Thus with the ship close and nearly parallel to the bridge, and a further 7,000 tonnes of concrete debris in the mud at the bottom of the river, the need to do an accurate survey of the debris was of prime importance. There was no question of trying to remove the debris in this depth of water as the large concrete sizes (pile caps 500 tonnes) would have made the work hazardous, difficult and time consuming. Before commencing this survey a funadmental decision was taken that the new piles should be large diameter vertical cylinders and if necessary, rock anchored at the base to reduce their lateral flexibility. This avoided having to do the survey on a "rake" and the imposed verticality significantly improved its accuracy.

The debris survey required extensive use of divers in difficult conditions of very deep water and a visibility at best of about 1 m using torches. Three different methods were used. The principle method was the use of underwater ultrasonic survey equipment developed by the University of Tasmania and reported on by LAWSON /6/. The equipment consisted of a transmitter carried by the diver and placed against the object to be located, an array of four receivers mounted on a semi-submerged platform and a central control unit providing control signals and outputs. By measuring the time taken for the sound wave to be transmitted to the receivers, which had been accurately located by surveying from the shore, and using simple co-ordinate geometry calculated by a computer, a point on an object was quickly located within 100 mm. The other survey methods involved a probe consisting of 6 m of universal column section attached to a long steel cylinder, and diamond drilling through the debris.

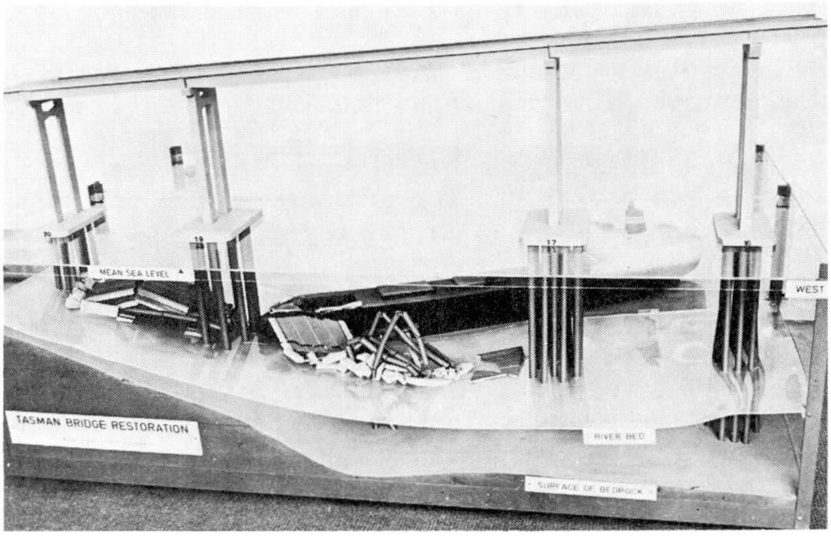

Fig. 3 Debris model

The debris survey took about three months and in the end all the broken bridge pieces were accounted for. A model was built to show the underwater picture (Fig. 3). During the debris survey it is to the credit of the Bridge Commission that pressures to make an early decision on the method of restoration were resisted. Five alternate schemes for reconstruction ranging from a single span scheme to a triple span scheme were examined in some detail. As the underwater scene unfolded the schemes were discarded one by one because debris prevented the placement of new piles, and the selected scheme only became evident at the end of the survey period when a complete picture was available. The only clear space available to establish a new pier was at the site of the original Pier 19 and this involved placement of new vertical piles between the existing broken off raking piles. The decision to delay the selection of a reconstruction method until the survey was completed was vindicated by job performance as no delays occurred due to new piles striking debris.

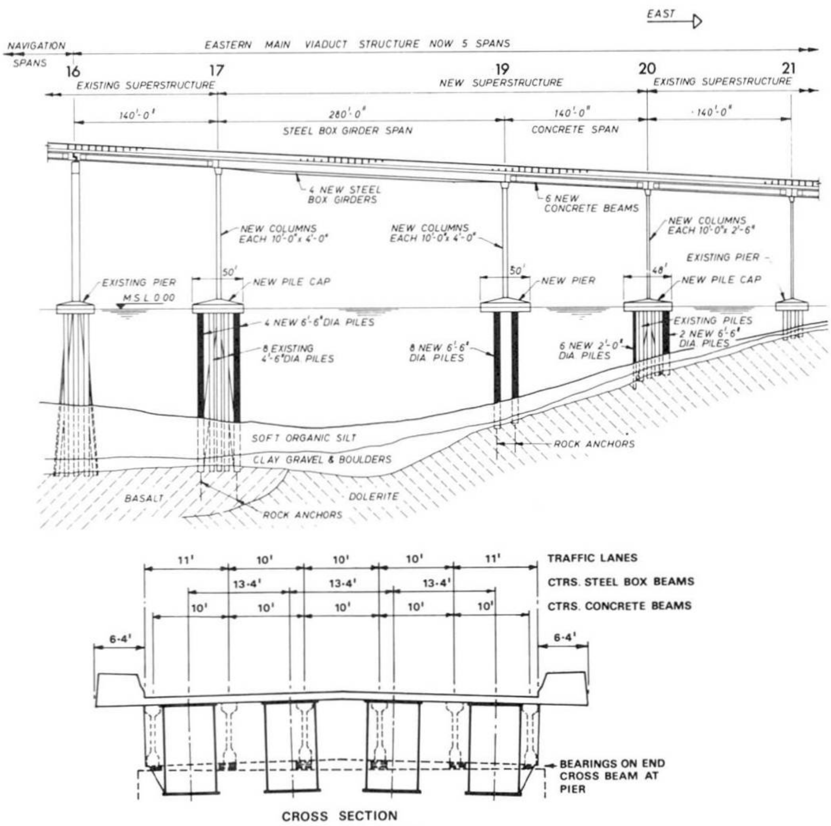

Fig. 4 Details of restoration

4. METHOD OF RESTORATION

The method adopted is shown in Fig. 4 and has been reported on previously by LESLIE /7/ and /8/. As the broken 1.4 m diameter piles at Pier 18 (40 m to bed level being the deepest part of the river), rose 15 m above the mud level this pier was omitted and the method involved the replacement of the three fallen spans by two spans; an 85 m steel span 17-19 and 43 m concrete span 19-20. Cross-sections are shown in Figs. 6 and 7. The scheme involved 14 new 2 m diameter vertical piles rock anchored at the base, some of which were to be used both as falsework piles and permanent piles. Foundation conditions were predetermined using one borehole per pile and were good, consisting of almost fresh dolerite at Piers 19 and 20 and partly weathered basalt at Pier 17. At Piers 17 and 20 the existing columns and pile caps had to be demolished and replaced. At Pier 17 additional piles, working in conjunction with the eight existing 1.4 m diameter raking piles, a bigger cap, and heavier columns were required to resist the greater loads from the double span. At Pier 20 there are no additional loads, but it was considered necessary to replace the badly damaged pile cap and columns, and the falsework piles required to achieve this were incorporated into the permanent structure, although not strictly required from a strength viewpoint. As there was a considerable time saving involved, the replacement of the substructure at these two piers was to be undertaken without demolishing the flanking spans (total mass 1,250 tonnes each) supported by them.

The method of restoration was determined in June 1975 and at that time it was expected that reconstruction would be completed at the end of 1977 after 30 months. At the outset it was hoped that this forecast could be improved upon. The construction relied heavily on the use of water borne equipment supplied by the Contractor. The main items were, Derrick Barge No. 1 displacing 280 tonnes with two triple drum five tonne mooring winches, a 30 tonne Favco stiffleg derrick (32 m jib), Kobe K42 pile hammer and Franki machine; Derrick Barge No. 2 carrying a Favco 1500 tower crane with jib combinations to 48 m and a crane barge with hand mooring winches carrying a 35 tonne Linkbelt Crawler crane with 21 m jib.

5. SECURITY

Before piling work commenced it was necessary to ensure that the damaged existing structure was secure. The work involved the deflected and cracked columns at Piers 17 and 20 and the damaged Pile Cap 20.

For the columns the work consisted of an ultimate load check, incorporating an accurate theoretical assessment of P-Δ effects, to demonstrate that the deflected columns were safe, and arising from this the provision of tie bolts between the deck and crossheads to maintain a pinned connection at the top of the columns. In addition the precautionary measure was taken of welding tie plates across the Pier 16 expansion joint. Some time later when it was possible to examine the Pier 16 stay bolts, some plastic strain and necking was evident and the caution shown earlier proved worthwhile.

The security of Pile Cap 20 was more difficult as it had lost about one third of its volume (Fig. 5) and the remainder was extensively cracked and in some places crumbled. Its load carrying capacity was unknown and although it had to be replaced later it was decided to do temporary repairs before any major work commenced. These were in two parts:-

- Replacement of the concrete which had been removed without attempting to repair or remove and replace the cracked concrete.

- Construction of a temporary steel pile cap above the existing cap (Fig. 6). Its purpose was to enable column loads to by-pass the existing cap and be transmitted directly into the piles. This action was achieved by stressing the steel cap to the columns and by casting concrete plinths on top of the existing cap, directly above the piles, and stressing the steel cap to the piles by means of rock anchors drilled and grouted into them.

These security works contributed significantly to the confidence of the site labour force at the commencement of construction.

Fig. 5 Damage to Pile Cap 20

6. PILING

Details of the new vertical piling are shown in Fig. 7. 2 m diameter was selected as being the largest size which could be

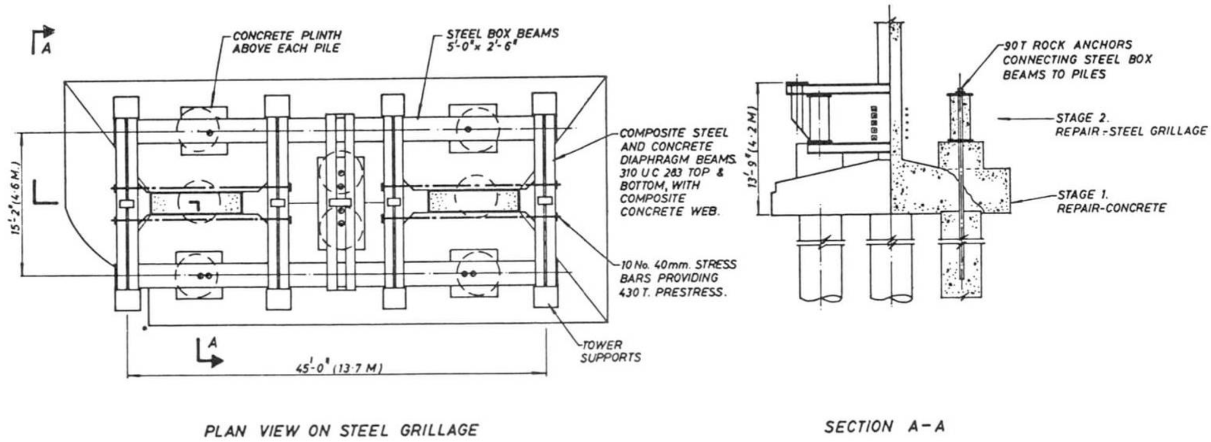

Fig. 6 Pile Cap 20. Temporary steel cap

reasonably handled. Base fixity to the piles was provided by means of rock anchors. At Pier 19 where all piles are vertical deck deflection under a 45 m/sec. transverse wind is 40 mm being 1 in 1,500 of the deck height above foundation level. The maximum loads per pile at this pier were 10,630 kN and 4,650 kNm.

The principle design features of the piles were; casing designed to be dewatered; reinforced concrete tremie plugs (40 MPa) to give F of S against flotation of 1.1; rock anchors designed for maximum corrosion resistance and each one tested in accordance with FIP recommendations. For safety reasons the test load was 70% UTS and lock off load 54% UTS.

During construction advantage was taken of the deep water to tow out full length casings (maximum 50 m long and 60 tonnes mass) and rotate them to the vertical position using flotation principles.

After founding, the tremie plug was poured and rock anchor holes drilled from the top through a casing cluster inserted into the wet tremie concrete.

Back-grouting and redrilling using a rock roller enabled the anchors at Pier 19 and 20 to be placed in the dry using immersion principles.

At Pier 17 due to frequent rock joints the anchor holes could not be sealed and the installation method was modified to enable anchors to be grouted in a flooded pile (Fig. 7). Individual holes were kept grout filled while the grout leaking into adjacent holes was flushed out. A reservoir at the top of the plug enabled the grout quality to be monitored by underwater TV camera while visibility was maintained by downward circulation of fresh water.

The remaining pile operations were straight forward.

7. TEMPORARY WORKS

Major on site temporary steelwork totalling 1,000 tonnes approximately was required. The major items were steel towers at the three piers, arrangements for jacking spans both horizontally and vertically, the underwater support of existing pile groups using 6 m deep steel trusses, and steelwork for temporary diaphragms and articulation.

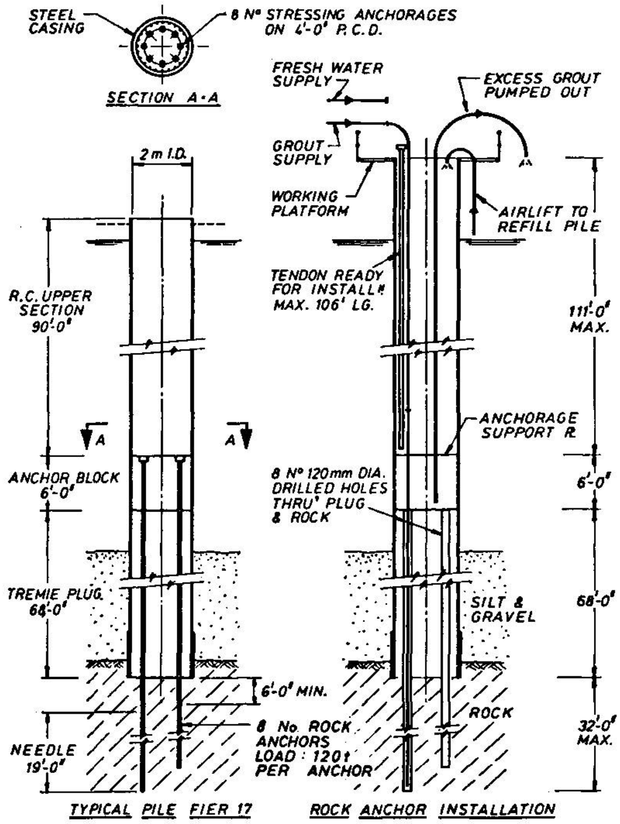

Fig. 7 Pile details

The steel towers fulfilled many functions, including support of existing spans during column demolition, bracing for new concrete columns during construction, supports for the erection derricks and landing points for the new beams. The construction sequence involved in the use of the tower at Pier 17 is shown in Fig. 8.

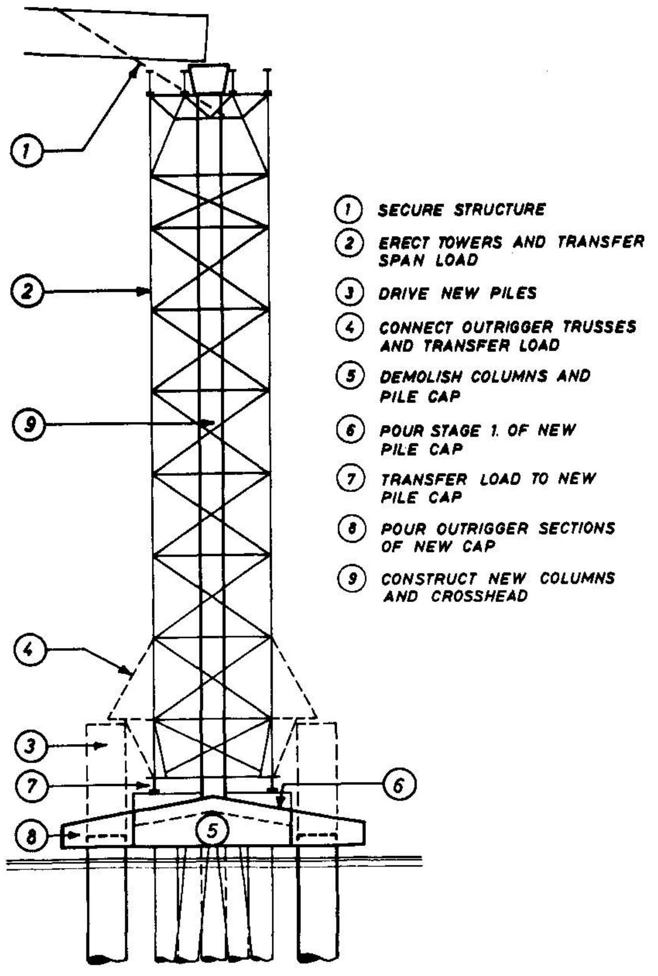

Fig. 8 Pier 17 construction sequence

8. DEMOLITION

Piers 17 and 20 had to be demolished to water level. The original crossheads were heavily reinforced concrete 1.25 m x 1.7 m x 16.5 m weighing 81 tonnes. They were cut into four equal pieces using the thermic lance technique, skidded from under the span across the top of the temporary steel tower and lowered to water level. The process involved burning horizontal holes through the top and bottom reinforcing mats and also through the distribution steel. The section was then split using hydraulic wedge splitters.

The original columns were made up of precast concrete blocks, vertically stressed together with Macalloy bars. They had a cross-section of 3 m x 0.75 m and were cut horizontally using a thermic lance and hydraulic wedge splitters. The 25 tonne pieces were lifted out through the top of the falsework tower (Fig. 9). Twelve working days were required to remove the two legs of the Pier 20 column.

The original pile caps measured 16 m x 7.6 m x 1.8 m and were demolished using track mounted pneumatic equipment working under the falsework towers.

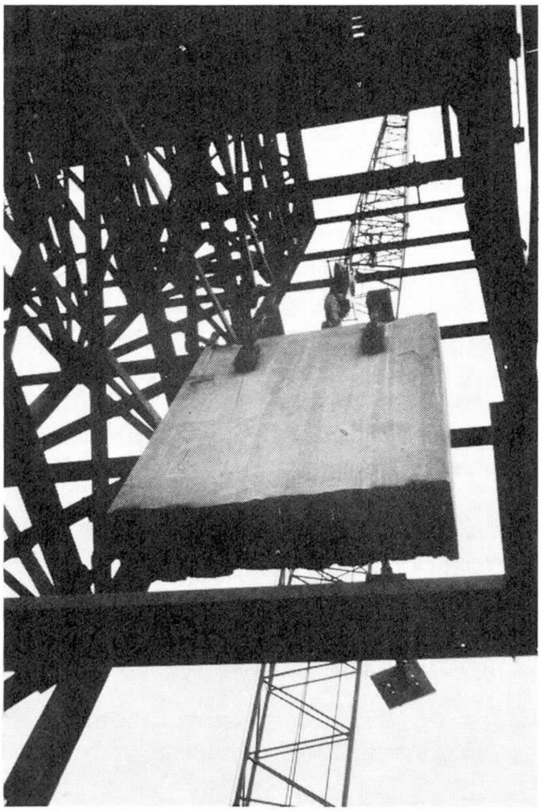

Fig. 9 Column demolition

9. STEEL BEAMS

Steel was selected for the longer 85.4 span primarily because it would be lighter in weight, and therefore pile loading would be less and erection easier than for a concrete span. The boxes were fabricated from high tensile steel (Grade 350 L15) and were fully welded. Plate thicknesses were up to 32 mm and the philosophy adopted, to minimise time and cost, was to use thicker plate sizes and less stiffening. It was necessary to replace the system of balanced rocker and knife edge bearings at each pier, which was done by providing six bearings to match the six concrete beam positions. Therefore the bearing locations generally fall some distance from the steel box centre lines, necessitating stiff steel boxed transverse cross beams at the ends. The "MERRISON" [9] Interim Design Rules were used for the design of the steel boxes and were useful in assessing the results of

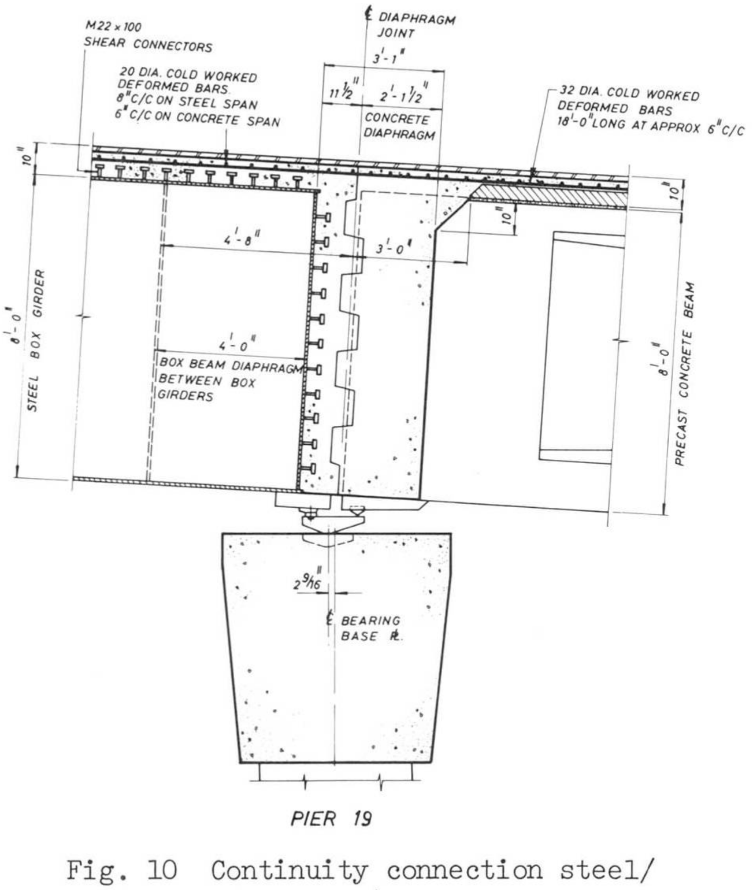

Fig. 10 Continuity connection steel/ concrete spans

the finite element analyses of the end crossbeams. The continuity detail between the steel and concrete spans over the piers (Fig. 10) was similar to that used in the original design.

The depth of the girders was generally 3.6 m tapering to the same depth (2.4 m) at the ends as the adjacent concrete beams. To minimise the appearance problem of the missing pier a light non-structural fascia was attached to the outer boxes, so that the extra depth of the steel beams was in shadow.

10. ERECTION

The erection scheme was dictated by the heavier and larger steel boxes (250 tonnes). Initially, launching was considered but this was ruled out because the support reactions were too heavy for the existing concrete deck. It would also have made the concurrent widening of the bridge impossible.

Fig. 11 Box girder erection

The adopted scheme used luffing 'A' frame lifting derricks specifically designed by the Contractor for the purpose (Fig. 11). By using temporary cantilever extensions at the ends of the boxes, they were lifted outboard of the towers as a single unit and positioned on top of the towers. The boxes were towed to site on a barge and then lifted. The operation from mooring of the barge to lifting the girders 40 m, and positioning them on temporary bearings on the falsework towers took about 20 minutes.

The six precast concrete beams for spans 19-20 were erected in the same manner

11. BRIDGE WIDENING

While the bridge was out of service the opportunity was taken to widen it from four to five lanes over its full length. This permitted the operation of a rather more comfortable 3:2 tidal flow system rather than 3:1 as previously. The widening was achieved by cantilevering new footways and using the existing concrete deck for the five lanes (Fig. 12).

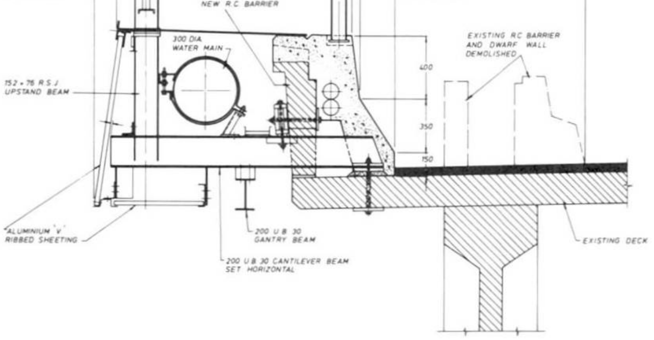

Fig. 12 Bridge widening details

The 42.7 m span, prestressed concrete beams in the main viaducts were constructed segmentally. Due to the extra load from the widening, the outer beams were in tension at working loads and for serviceability reasons it was decided to strengthen them by adding external tendons in a "Vee" shape one on each side of the web (Fig. 13). The tendons were 8 x 12.5 mm strands grouted into a 75 mm steel pipe and were stressed by pulling them down at mid span, and anchored by bolting to the mid span saddle to give an uplift force of 38 tonnes.

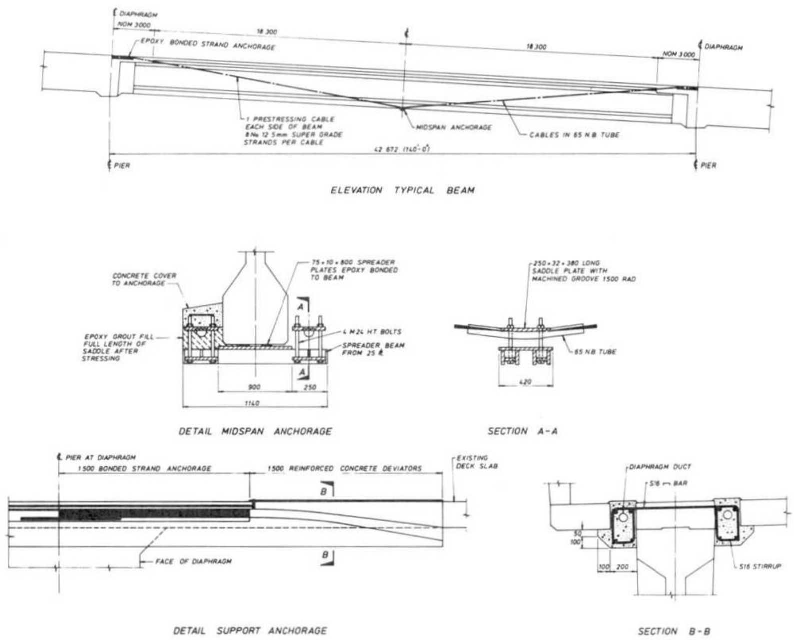

Fig. 13 Beam strengthening details

It was originally intended to anchor the tendons in the deck by breaking out an area of concrete and casting a traditional anchor block. To reduce the required demolition and avoid cutting the main transverse deck reinforcement, the Contractor, Pearson Bridge (Tas.) Pty. Ltd. proposed cutting a slot 3 m x 200 mm (Fig. 14) into which the bare strands were anchored using an epoxy with fillers. The method worked very well.

Fig. 14 Slots

12. CONCLUSION

The bridge was opened to traffic on the 8th October, 1977 ahead of schedule. The restoration consisted basically of demolishing a part of the original bridge and rebuilding "two" bridges, one of steel and the other of concrete. As a result the whole job was on a learning curve with no production runs available. The task involved many unusual engineering features and decisions and a huge number of engineering hours.

The successful completion of the project slightly ahead of time is a tribute to all who were involved.

13. ACKNOWLEDGEMENTS

This paper is published with the kind permission of the Joint Tasman Bridge Restoration Commission and of the Department of Main Roads, Tasmania.

Imperial units have been used on some of the figures in this paper, as all existing information concerning the structure was in these units.

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Renovation of Lions' Gate Bridge

Rénovation du pont Lions' Gate

Erneuerung der Lions' Gate Brücke

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SUMMARY
Rising capital costs of new construction are forcing bridge engineers to give serious consideration to renovation instead of replacement of existing bridges. This paper, which describes the renovation of Lions' Gate Suspension Bridge in Vancouver, Canada, identifies some of the defects common to bridges of 1930's vintage and discusses the steps taken in this case to remedy these defects and prevent their recurrence without unduly disrupting peak traffic flows on the bridge.

RESUME
La hausse des coûts de construction oblige les ingénieurs à examiner sérieusement la rénovation au lieu du remplacement de ponts existants. Cet article décrit la rénovation du pont suspendu Lions' Gate à Vancouver, Canada, identifie quelques-uns des défauts communs aux ponts des années trente et discute les démarches entreprises pour remédier à ces défauts et empêcher qu'ils ne se reproduisent, sans entraver de façon majeure la circulation sur le pont aux heures de pointe.

ZUSAMMENFASSUNG
Steigende Kapitalkosten für Neubauten zwingen Brücken-Ingenieure, Renovationen ernstlich in Erwägung zu ziehen, anstatt existierende Brücken durch neue zu ersetzen. Dieser Beitrag beschreibt die Renovation der Lions' Gate Hängebrücke in Vancouver, Canada, beschreibt einige häufige Mängel von Brücken der Dreissiger Jahre und erörtert die Schritte, die unternommen wurden, um diese Mängel zu beheben und ein Wiederauftreten zu verhindern, ohne den Hauptverkehrsfluss der Brücke unnötig zu unterbrechen.

1. INTRODUCTION

Lions' Gate Bridge was built in 1938 and has a total length of 1518m. The South approach road is located on a bluff and leads directly into the sidespan of the high level Suspension Bridge, which has a total length of 847m and a mainspan of 473m. At the termination of the North sidespan the bridge continues for another 669m over the North Viaduct with 25 spans ranging in length from 13m to 38m.

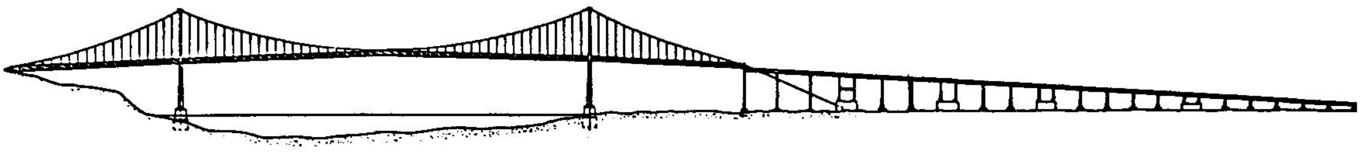

Fig. 1 Elevation Lions' Gate Bridge.

The North Viaduct was built of rivetted steel construction with a reinforced concrete deck supported on cross beams and longitudinal plate girders. The Suspension Bridge comprised a concrete filled steel grid deck supported by steel stiffening trusses and towers.

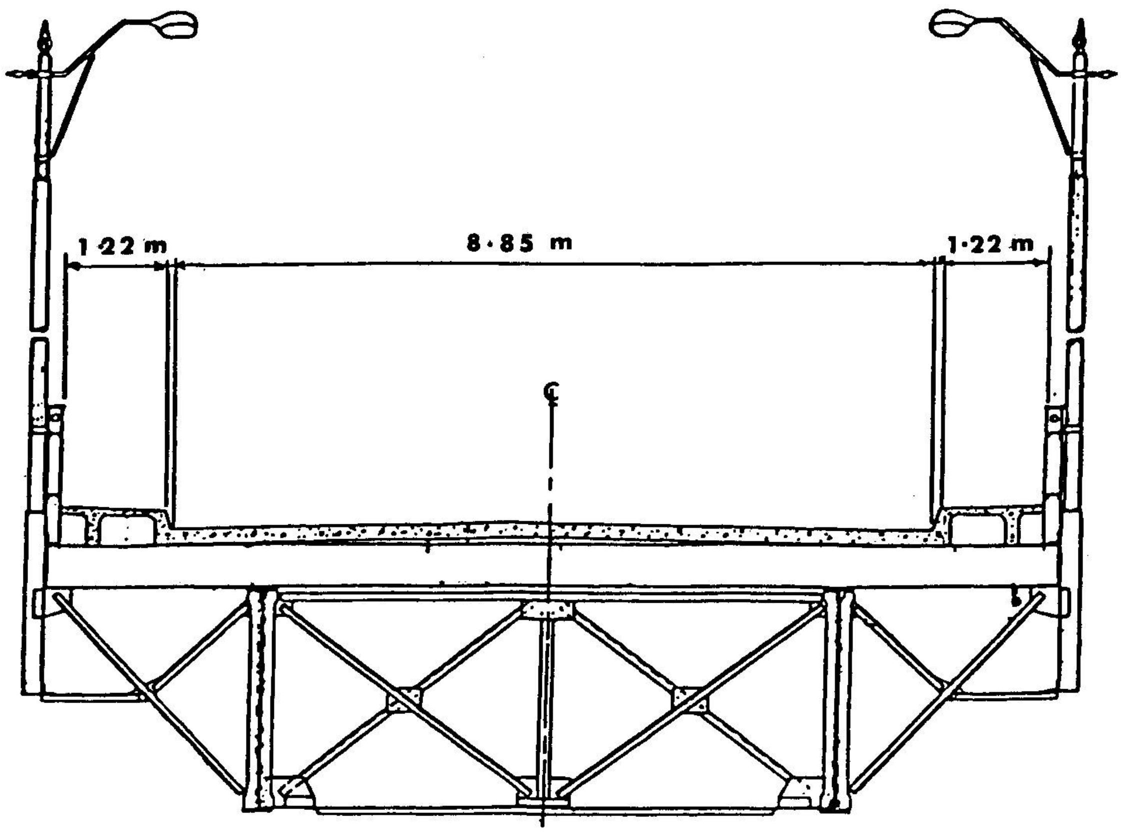

Fig. 2 Cross Section of North Viaduct

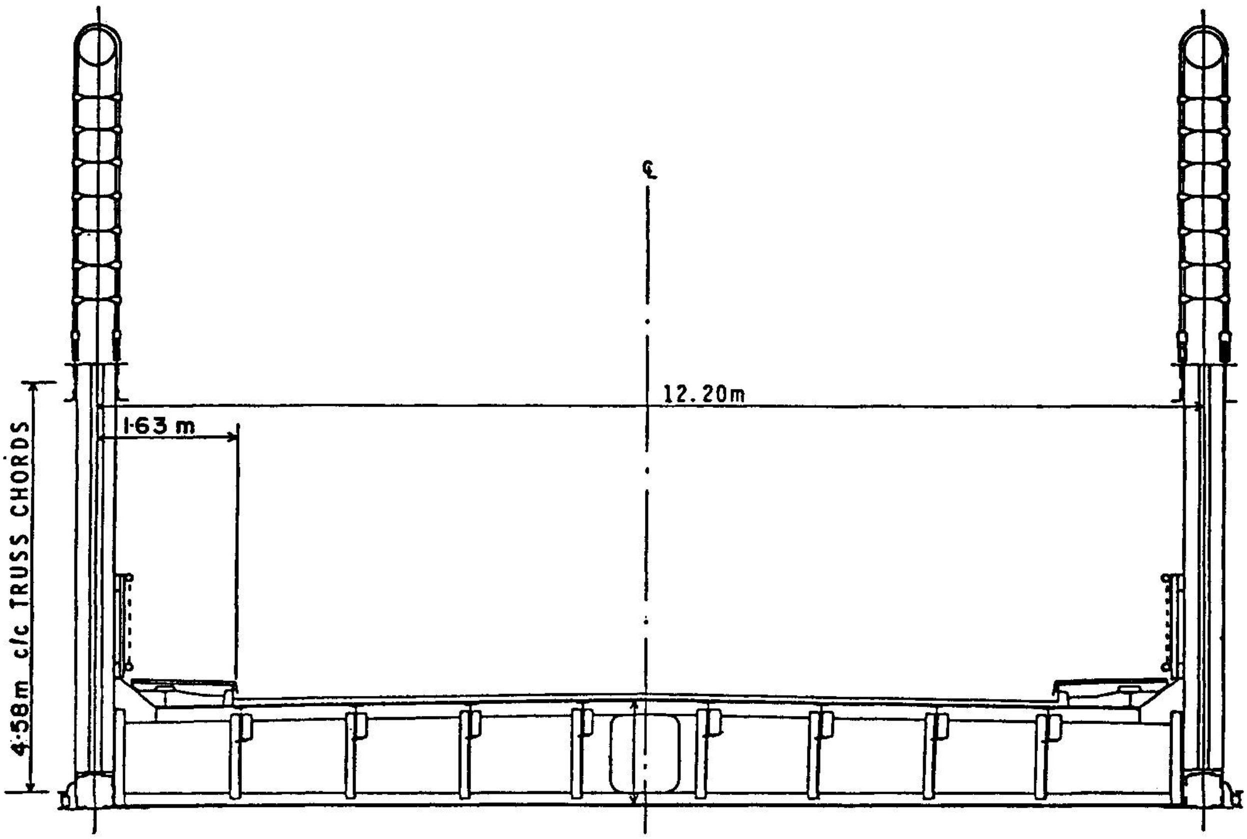

Fig. 3 Cross Section of Suspension Bridge.

The stiffening truss, being of a U shape in cross section (see Figure 3), has a low torsional rigidity. The towers are made of rivetted boxes and the girders and bents of the North Viaduct are rivetted steel plate girders. The South Main Tower is founded on rock, but all other foundations are spread footings on gravel.

The form of construction of this bridge is common to many medium span suspension bridges built in the 1930's in North America and elsewhere. Bridges of this vintage are now approaching a point in their lives where renovation or reconstruction must be considered.

This bridge forms an essential artery between the suburbs and downtown Vancouver. Construction of a new crossing was considered in the early 1970's, but was rejected by conservationists. With replacement ruled out, it became necessary to consider renovation.

Renovation of Lions' Gate Bridge is proceeding in three Phases:-

Phase 1	-	1974-5	Renovation of North Viaduct
Phase 2	-	1978-9	Repairs to North Cable Bent and Footing
Phase 3	-	1980-3	Renovation of Suspended Spans.

2. TRAFFIC ON THE BRIDGE

The bridge as built had a roadway width of 8.84m and carried two lanes of traffic. This was later increased to three lanes each of 2.95m as traffic volumes built up. The traffic comprises only automobiles and buses (trucks were banned in 1973 in order to reduce live load) carrying commuters to and from downtown Vancouver. Due to user familiarity with the bridge and also to the high degree of control exercised by the Bridge Patrol Officers, who direct tidal traffic flow by lane signal lights, the traffic volumes passing over the bridge are very high (about 60,000 vehicles per day) despite the narrow traffic lanes. In winter about 60% of the vehicles on the bridge use tire studs. This fact, coupled with the high traffic volumes and the uphill grade for most of the bridge length, combine to create very severe wear of the bridge deck paving.

3. PLANNING FOR RENOVATION

Renovation of a bridge may be carried out under any one of three traffic conditions:-

1. - Renovation with normal traffic flow on the bridge.
2. - Renovation with the bridge closed to traffic.
3. - Renovation with one or more traffic lanes closed.

Certain bridge components may be renovated under condition 1, but any major work affecting the bridge deck requires condition 2, or 3. The choice between these alternatives depends upon the deck structure itself. Where a deck is supported by longitudinal stringers, the deck may usually be renovated or replaced one lane at a time, although a second lane will normally be required for access by construction machinery.

Thus on a bridge having 4 or more lanes, progressive lane renovation can be carried out at off peak periods without total bridge closures. However, if the deck structure is supported on transverse beams, as is the case for Lions' Gate Bridge, then full bridge closures are necessary for replacement of these components.

Because this bridge is a critical urban link, prolonged total bridge closures were out of the question. It was considered essential to maintain weekday peak commuter traffic flows in all lanes. Thus bridge renovation was planned around 6.50 hour night closures (23.30 to 06.00 hrs.) for work in the North Viaduct and 48 hour weekend closures for the Suspension Bridge renovation. This time constraint, requiring complete sections of bridge deck to be renovated and opened for traffic in such short time periods, added considerably to the design and construction problems of this project.

4. NORTH VIADUCT

4.1 Components Requiring Renovation

After 35 years of wear, the deck of the North Viaduct was in very poor condition. It comprised a 178mm reinforced concrete slab without wearing surface. Abrasion by traffic had removed the concrete cover and the top reinforcing steel entirely in places and furthermore cores showed cracks extending through the full slab depth. The supporting floorbeams were severely corroded while the plate girders and bents showed local corrosion where de-icing salts used on the road had accumulated.

4.2 Design for Renovation of the North Viaduct

Basically all of the problems in the North Viaduct stemmed from the bridge deck. Its main shortcomings were:-

1. - lack of wearing surface.
2. - insufficient road width.
3. - crevice corrosion due to drainage of de-icing salts from the deck onto rivetted support steel.

It was desirable to correct all of these deficiences within the constraint of short bridge closures (as mentioned earlier), while at the same time increasing live load capacity of the bridge to bring it in line with modern design loads for a three lane bridge. Heavier traffic loads on a wider deck, acting at larger eccentricities from the girders and riding on a wearing surface which didn't previously exist, all tend to increase the forces on the support structure. Conversely, corrosion losses in the support steelwork required that the total effective load be reduced for safety. These opposing structural constraints had to be resolved by the design for renovation.

Consideration of the constraints on a structural solution for the North Viaduct led to two conclusions. Firstly, the new deck must have a much lower dead weight than the original in order to afford margins for wearing surface, increased width, traffic barriers etc. and secondly, the new deck and girder system must have increased live load bending capacity to handle a higher live load with a girder capacity reduced by corrosion.

A suitable solution was found in the form of an orthotropic steel deck connected by diaphragms to the existing longitudinal girders in order to ensure composite behaviour under live load. This effectively reduced dead load well below previous values and also reduced girder live load bending stresses.

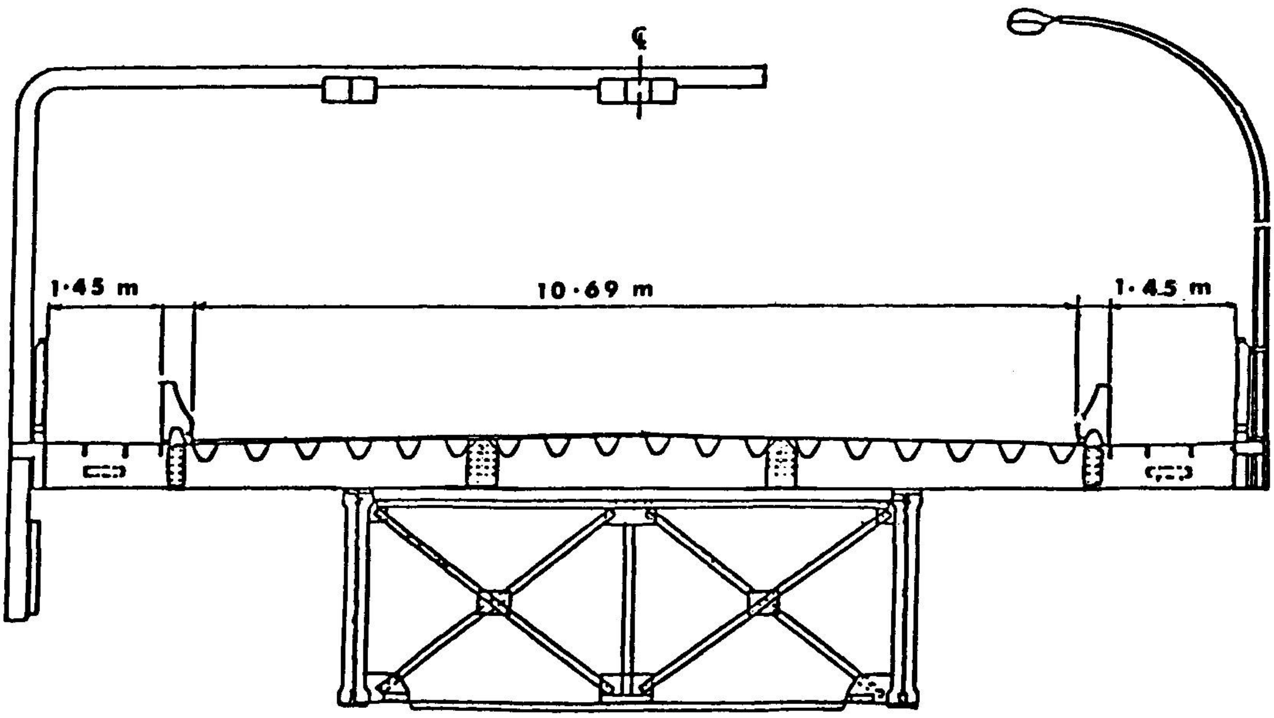

Fig. 4 Cross Section of Renovated North Viaduct

Fortunately, there was excess shear capacity in the existing girder webs. A 37mm wearing surface of epoxy asphalt was specified in the traffic zone with a 2mm epoxy coating on the sidewalks. A cross section of the renovated North Viaduct is shown in Fig. 4.

Epoxy asphalt has been found by experience in North America to be compatible with the flexibility requirements of orthotropic steel decks and to offer improved abrasion resistance in areas of high wear.

Many schemes were considered for combatting the corrosion attack on the viaduct support girders and bents. This had resulted in crevice corrosion of alarming appearance at the interface of the 8mm flange cover plates between rivets spaced up to 175mm apart. The corrosion product forming between the plates had forced the outer ones to bulge with permanent deformations up to 7mm. Fortunately, the most severe damage was in the tension flanges of the girders. Corrective measures considered for this problem included sealing every crevice (a lifetime's work!), encasing the steel structure in concrete and using sophisticated de-icing compounds on the bridge deck instead of salt.

All of these schemes had major drawbacks or limited chance of success. Finally, it was decided to measure actual corrosion losses at a few typical sections on the girder and to attempt to predict probable future section losses from this data. This approach proved useful. Despite the alarming appearance of the corrosion products, the actual section losses were found to be less than anticipated, ranging from 2 to 4% loss. Based on the most pessimistic corrosion predictions it was shown that a modest reduction in the corrosion rate, coupled with live load composite action by the orthotropic deck, was sufficient to ensure adequate load carrying capacity in the girder flanges for several more decades. The reduced corrosion rate was achieved by sealing the new deck with joint seals and ducting the de-icing salts away to ground level through a closed drainage system. Thus an impermeable "roof" was provided to reduce corrosion, furthermore the quality of the paint protection was increased by sandblasting and recoating the steelwork to a uniform high quality.

Other improvements incorporated in the renovation design for the North Viaduct included a protective guard-rail between automobiles and pedestrians (made of galvanized steel to minimize weight), a new electrical distribution and emergency power system and improved access beneath the bridge for maintenance.

5. SUSPENSION BRIDGE

5.1 Components Requiring Renovation

The defects occurring in the Suspension Bridge were of a more complex nature than those in the North Viaduct. Bridge components were in general in better condition, but the structural defects were more significant.

Starting with the foundations, it was discovered that the footing of the North Cable Bent, located at the North end of the Suspension Bridge, had settled unevenly over the years and was inducing large secondary stresses in the tower legs. Furthermore, continued slippage of the main cables in the saddle at the top of this tower had aggravated the situation. See Figure 5.

The stability of this tower is critical to the safety of the entire bridge. Other significant structural defects in the Suspension Bridge included live load bending overstress in the stiffening truss and inelastic stretch in the main cables which caused main tower out of plumb and secondary bending in the stiffening truss. The suspended spans also suffered from the same basic problems as the North Viaduct due to crevice corrosion in the support steelwork and towers plus decay of the deck structure due to corrosion and wear. In this case the deck construction is steel tee grid infilled with concrete. This deck has no wearing surface and is unacceptably slippery when wet. Furthermore, the deck is not waterproof and corrosion between the tees and on the top of the floorbeams tends to cause the tees to pry loose from one another and the support steel.

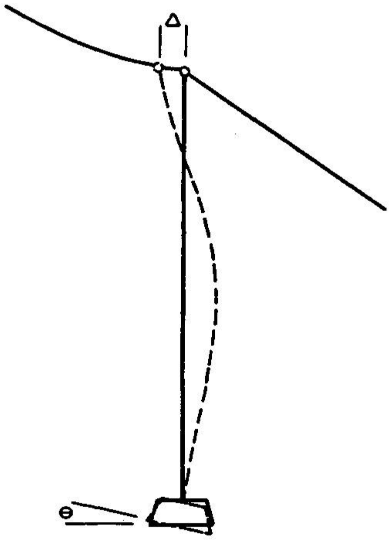

Fig. 5 Schematic View of North Cable Bent

The objectives of the renovation in this case were:-

1. - to secure the North Cable Bent in a safe and stable condition.
2. - to provide a new wider, waterproof deck with a wearing surface and improved drainage.
3. - alleviate the bending overstresses in the towers and stiffening trusses.

5.2 Design for Renovation of the Suspension Bridge

The first objective of this phase of the renovation was to restore the North Cable Bent to a stable condition prior to commencing other work in the suspended spans. Bridge strand tiebacks were installed between the North anchorage and the top of the bent in order to prevent further slippage of the clamp among the main cables. After this was completed, the footing was underpinned to prevent further settlement and then rotated to relieve secondary bending in the column. The steps in this process are illustrated schematically below:-

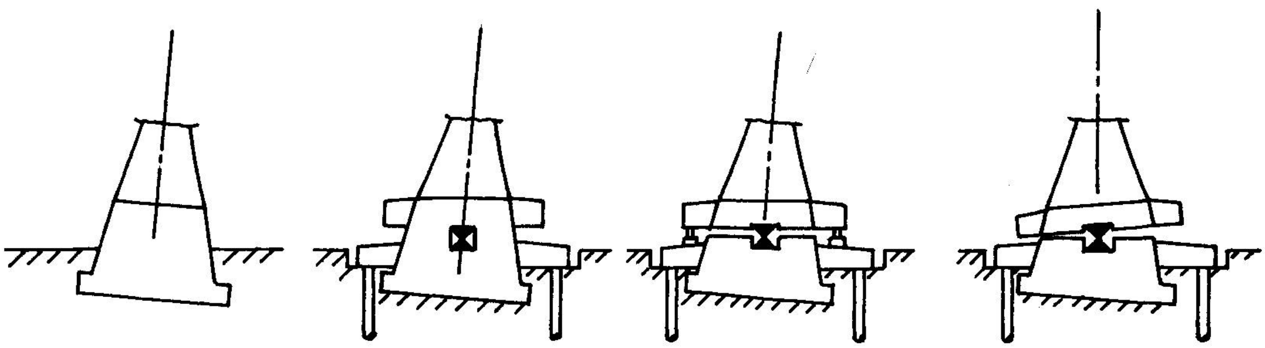

Fig. 6 Stabilization of North Cable Bent

It seemed natural to renovate the deck in the suspended spans in a similar manner to the North Viaduct using an orthotropic deck and epoxy paving, and this is in fact being done. However, there were a number of difficulties in framing this deck into the existing stiffening truss and floorbeam system within the constraints of short bridge closures and the requirement to provide adequate corrosion resistance and long term strength in an already overloaded stiffening truss. Furthermore, widening the road deck could only be achieved at the expense of cantilevering the sidewalks outside the stiffening truss, where they significantly reduced the aerodynamic stability of the cross section. In light of the poor behaviour in wind of U shaped trusses, decreases in aerodynamic stability were regarded as undesirable. Aerodynamic model tests of the full bridge, carried out at the National Research Council of Canada [1], showed that aerodynamic stability could be attained, but at the price of fairings over some of the mainspan.

Upon consideration by the Owner of all these factors, including aerodynamics, maintenance, strength, ease of renovation etc., it was decided not to install a renovated deck within the existing truss but to design a new truss integral with a wider orthotropic deck. This decision permitted the new truss to be sized and detailed specifically to suit the requirements of renovation. It was dimensioned sufficiently shallow to fit entirely below the deck, which participates in the vertical bending of the truss. The bending stiffness of the new truss system is less than the original. However, the aerodynamic stability of the new section is made superior to the existing by closing the new truss into a box configuration and thus improving the torsional stiffness by a large factor. A cross section of the new configuration is shown below.

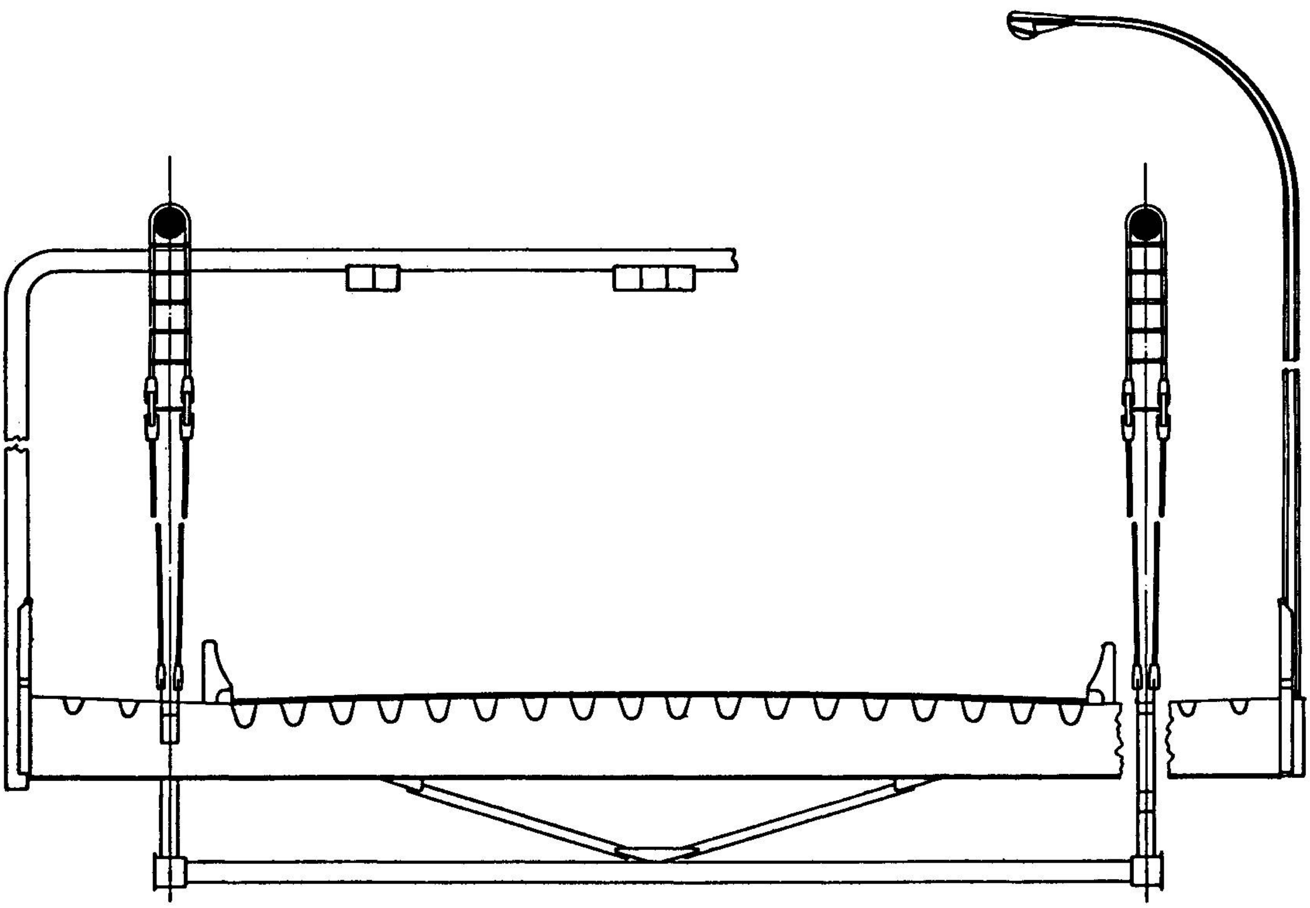

Fig. 7 Suspension Bridge New Stiffening Truss Section

In order to save enough weight to permit asphalt paving to be added, and also to minimize corrosion and simplify maintenance, truss members are welded rectangular hollow sections. Welded splice design posed some problems due to the anticipated low fatigue strength of one sided welds inevitable with tubular construction. However, prototype joint fatigue tests [2] conducted at Universitat Karlsruhe indicate that adequate fatigue resistance can be obtained, in most cases with one sided fillet welds.

At first analysis, the tower out of plumb due to cable stretch appeared to be a serious problem. Combined tower stresses including secondary bending exceeded design allowables. However, after a careful assessment of ultimate loads and displacements, it was demonstrated by a load factor approach that adequate live load factors against collapse could be attained provided appropriately low load factors were used on the predominant dead load components of tower load. This approach avoided the necessity of massive reinforcement of the towers. It only proved necessary to replace some rivets with high strength (A490) Bolts to increase the capacities of some connections.

6. DESIGN OF CONSTRUCTION FOR RENOVATION

For intermittent renovation of this type, the traffic closure is short and finite and the bridge must be ready to receive traffic at the end of the closure. This constraint dictates that:-

1. - the new deck be prefabricated to the maximum extent.
2. - the new deck be pre-paved.
3. - connection details be simple and tolerant of adjustment.

Maximum prefabrication implies completing everything possible before the actual closure commences. In this case paving, drains, traffic barriers, sidewalks, lighting, fences etc. were all installed on the deck sections ready to lift into place before commencement of the closure.

New orthotropic decks must be pre-paved before erection because traffic cannot pass safely over unpaved steel. For the North Viaduct, the panels were fully pre-paved prior to erection against steel paving stops at the end of each section. This discontinuity will be avoided in the suspension bridge by pre-paving to half thickness only prior to installation and then applying the remaining paving upon completion of the whole length of deck.

The method by which the new deck panels connect to the existing structure requires careful detailed design. For successful performance they must be quick to install but tolerant of adjustment in several directions. This is particularly necessary when connecting to existing structure because "as built" dimensions may differ from those shown on the original drawings or, as was discovered on Lions' Gate Bridge, settlement of the foundations may have caused distortions and misalignments in the structure.

For the North Viaduct, connection to the top flange of the existing plate girders was made by removing four existing rivets and bolting a nominal 40mm thick shim pack to the top of the girder with countersunk bolts. The height and tilt of the top surface of the shims could be adjusted by adding or removing shims, tapered if necessary. Once at the correct level and bolted down, the countersunk bolts afforded a flat surface, permitting the new deck cross beams to be moved laterally or longitudinally to the appropriate position before welding down. This simple device thus afforded linear and rotational movement in each of the three axes and permitted complete freedom of adjustment.

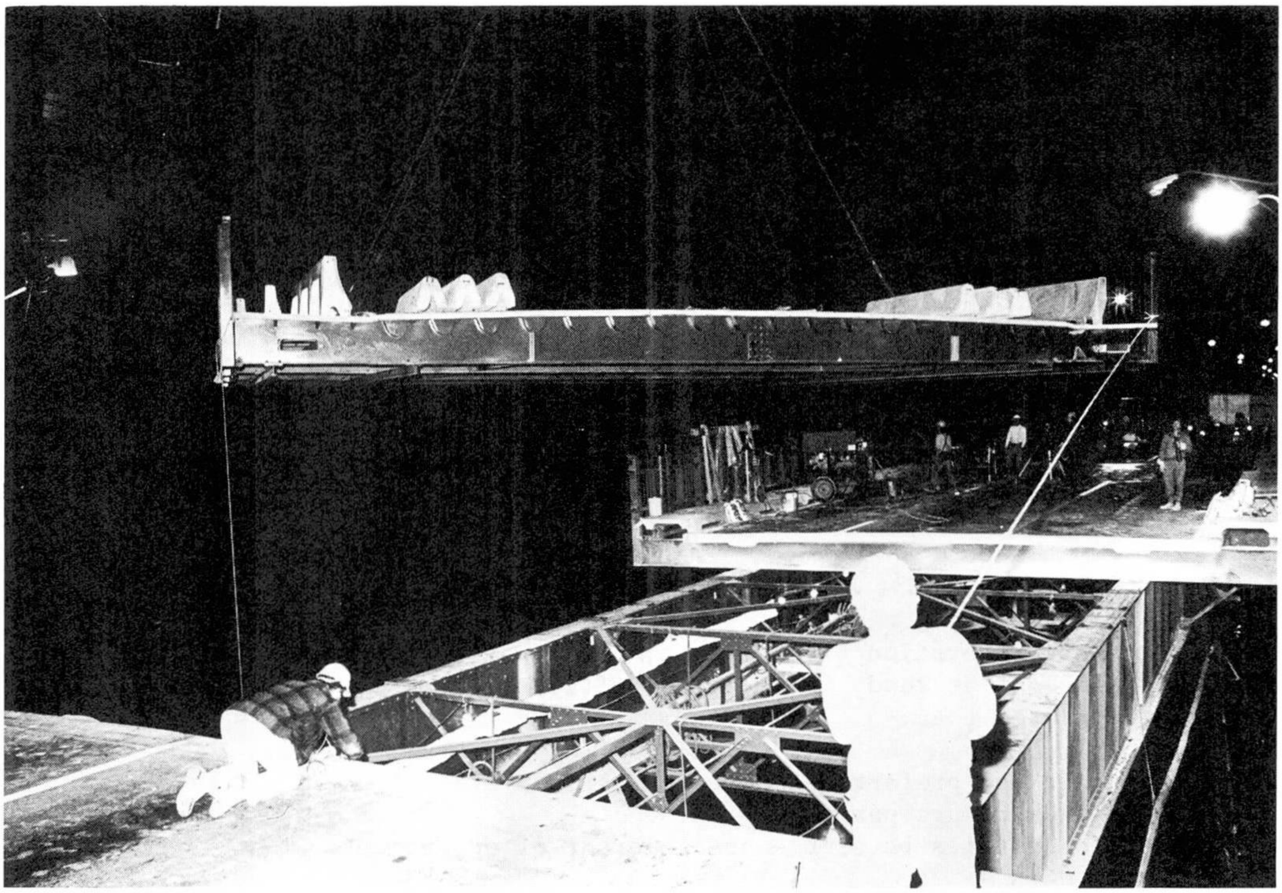

Fig. 8 Section of North Viaduct during Installation.

Other detail problems peculiar to renovation included:-

- maintaining load carrying capacity in the partly renovated deck system. This problem is particularly acute when replacing the stiffening truss of a suspension bridge.

- assuring smooth traffic flow over the joint between old and new sections. It was discovered that, when running at capacity, bridge traffic is very sensitive to minor discontinuities in the deck and serious traffic back ups can develop if a bump, perceived or actual, is introduced.

- precautions against major equipment failure during bridge closures. When working against the clock in short bridge closures the problem of machinery breakdown is very real and potentially serious. One approach is to demand back up machines for all key items of equipment. However, this is very expensive and perhaps unrealistic. One alternative, which was used on the North Viaduct renovation, is to have light modular temporary deck sections available adjacent to the site ready for immediate installation by a light crane should an emergency develop. This approach at least ensures that the bridge will be capable of carrying traffic after an equipment breakdown, although the vehicle flow over the temporary sections may be considerably less than over the permanent deck system.

7. PUBLIC INFORMATION

The importance of this aspect of renovation cannot be overemphasized. The bridge user is always curious about what is happening to the bridge, and if he is made aware of precisely what changes are being made and what the end result will be, then he becomes involved in the process and is more tolerant of minor delays and irritations. For all stages of renovation of Lions' Gate Bridge, a three fold public information system is being used. This consists firstly of media advertising giving a full description of the work to be done and defining the extent of traffic closures. The second phase comprises mailings to local businesses containing calendars of bridge closures. Finally, bulletins are supplied to local radio stations at 15 minute intervals as the end of the closure approaches so that the public is informed of the anticipated opening time for the bridge.

8. CONCLUSIONS

After forty years of wear and corrosive attack, this bridge had reached a point where maintenance problems were mounting and it was necessary to review how many more years it would remain servicable. Removing the bridge from service for repair for more than a few hours was ruled out, as was total replacement.

This paper shows that with ingenuity and careful design it is possible to successfully renovate a bridge under these circumstances.

With rising capital costs of new construction and an unwillingness by the public to support large scale new projects, it is becoming more important to consider the benefits of recycling major bridges.

When renovation of this structure is complete, it can anticipate an extended useful life as long or longer than its life to date. Furthermore, future decay will be minimized by careful attention to corrosion free details and improved access for maintenance.

9. ACKNOWLEDGMENTS

This paper is published with the permission of the British Columbia Ministry of Transportation Communications and Highways, W. A. Bowman, P. Eng., Director of Bridge Engineering.

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