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SESSION 2

Rating and Evaluation of Remaining Life of Bridges

Evaluation de la durée de vie des ponts

Schätzung der Lebenserwartung von Brücken

Development of a Comprehensive Computerised Bridge Rating System

Système informatique pour l'évaluation des ponts

Entwicklung eines umfassenden computerunterstützten Brückenbewertungssystems

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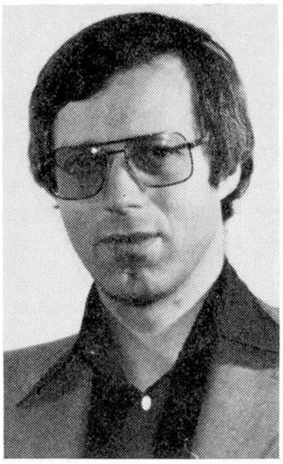

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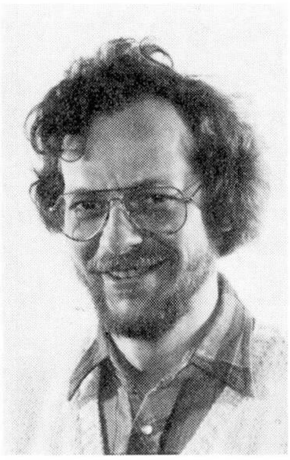

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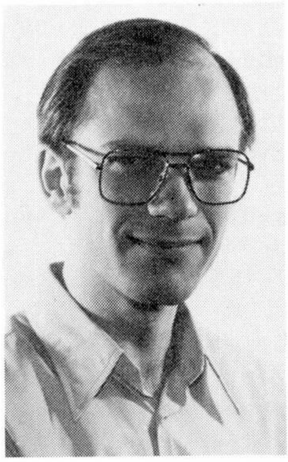

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SUMMARY

The accelerating deterioration of bridge structures cause major problems to bridge authorities all over the world. To establish priority of rehabilitation and replacements, and to manage special permits, all structures must be rated. When rating, the realistic load carrying capacity should be utilized, with due regard to the present physical state of the structure. In the contribution, the requirements to a comprehensive computerised rating and evaluation system are defined. Further, the practical approach to planning, development and maintenance of such a system under development in Denmark is described.

RESUME

La dégradation progressive de structures de ponts est un grave problème posé aux autorités, de tous les pays. Pour établir des programmes prioritaires de réparation et de reconstruction, ainsi que pour l'attribution de permis spéciaux, toutes les structures doivent être classées. Une telle classification doit être basée sur la capacité portante réelle en tenant compte de l'état physique actuel de la structure. La contribution définit les conditions à remplir par un système général de classification et d'évaluation de structures de ponts par ordinateur. De plus, elle décrit les méthodes pratiques de planification, d'établissement et d'entretien d'un tel système, en cours de réalisation au Danemark.

ZUSAMMENFASSUNG

Die zunehmende Zustandsverschlechterung von Brückenkonstruktionen verursacht erhebliche Probleme für die Brückenverwaltungen in aller Welt. Für alle Konstruktionen müssen Bewertungen geschaffen werden, um die Reihenfolge für Instandsetzung und Austausch festzusetzen und die Erteilung von LKW-Genehmigungen zu ermöglichen. Die Festsetzung der Belastungen muss aufgrund einer realistischen Traglastkapazität erfolgen unter Berücksichtigung des augenblicklichen Standes der Konstruktion. In diesem Beitrag wird ein umfassendes Computersystem für die Belastungs- und Bewertungsfestsetzung beschrieben. Weiter wird der praktische Vorgang bei Planung, Entwicklung und Unterhalt eines solchen Systems beschrieben, welches in Dänemark zur Zeit entwickelt wird.

1. INTRODUCTION

During the sixties and seventies the Danish highway network was rapidly developed to serve the increasing traffic demands, resulting from the economic and commercial progress.

This road construction, which as a backbone comprised a national divided highway system, has caused an unprecedented activity in bridge construction. The bridges vary from long span bridges in steel and concrete, crossing major waterways, to a significant amount of highway overpasses and underpasses, interchange bridges, etc.

The great majority of bridges applied the - at that time - rather new prestressed concrete technology which opened for new slender structural concepts and use of high strength concrete.

At that time most engineers were fully occupied by the planning and implementation of new construction activity, and little attention was paid to the durability problem. This for a good reason, as the reinforced concrete bridge - essentially built during the pre-world war construction boom in the thirties -had proven excellent and virtually maintenance free performance throughout their 20-40 year service life.

However, as the bridge inventory increased it was realized during the latter part of the seventies that the recently built concrete bridges suffered from rapid deterioration even after few years of service. The cause of this has been the subject of substantial research activity, and although this is still not concluded it is conclusive that changing traffic patterns (intensity and load), introduction of deicing salts, changing material characteristics (cement and aggregates), higher material strength utilization and less careful (accelerated) construction - and curing practice etc., are major contributory factors.

The drastically increasing obligation to highway administrations from increasing bridge inventory and accelerating deterioration simultaneously with further limited funding, imposes strict and selective management for max. benefit/cost ratio [1].

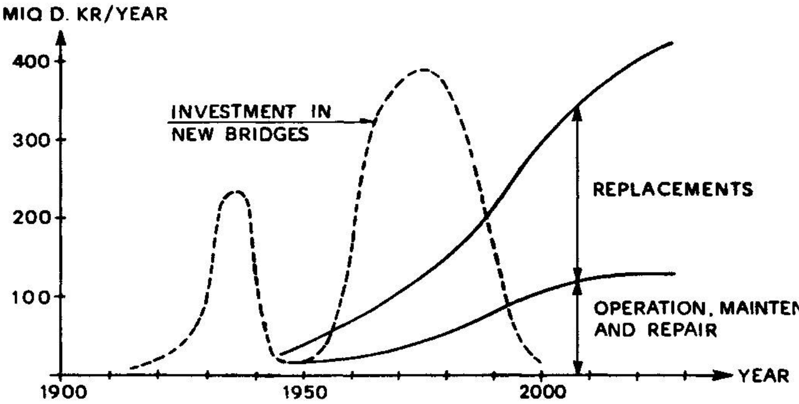

Fig. 1 Total yearly bridge investments and expenditures on the Danish national highway system [1].

In order to handle expenditures as indicated in Fig. 1, The Danish Road Directorate has started a methodical approach to an effective management system and Cowiconsult has been involved in this work.

2. BASIS FOR A COMPREHENSIVE COMPUTER SYSTEM

The increasing traffic intensity, in particular heavy transports, and a new traffic code with increased allowable vehicle/axle loads has required a complete rating of the existing Danish bridge inventory with respect to actual load carrying capacity. This has led to the development of a first generation of a computerised management system, as illustrated in Fig. 2. The system comprises databanks for storing general bridge data, reported deficiencies and data from inspections, in accordance with principles outlined in the OECD bridge inspection report [2]. Structures and vehicles are rated according to a matching standard system. Furthermore, special permits for exceptional vehicles can be issued either on the basis of the actual rating system or after supplementary structural analysis.

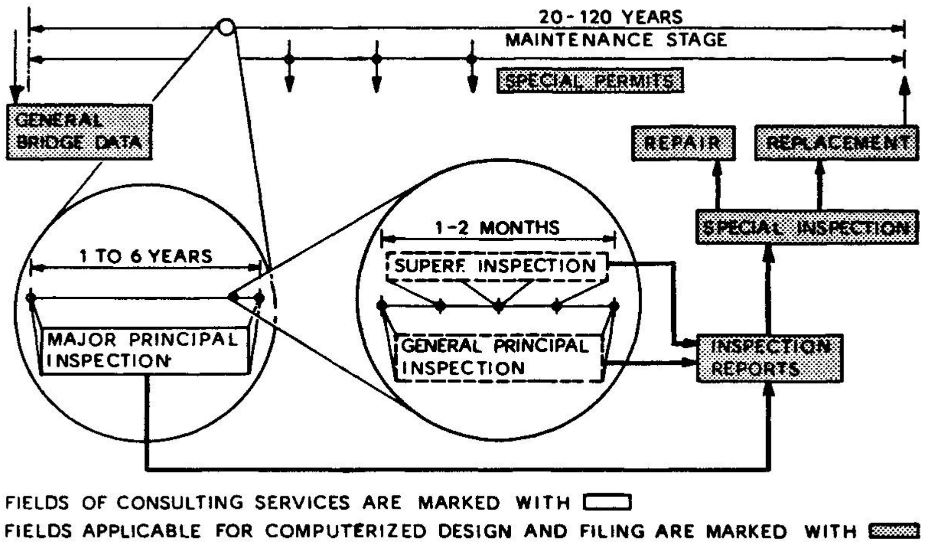

Fig. 2 Bridge maintenance system

However, due to the number of bridges to be classified/rated within a short period of time, only simplified methods on the safe side have been possible. A more accurate approach will be able to disclose a significant and realistic additional load carrying capacity, which - if documented - will permit higher ratings, and postponement of strengthening/replacement with optimum economic results.

Management of priorities for bridge repair, strengthening and rehabilitation, as well as the heavy transport road system and administration of special heavy transport permits are closely interrelated with structural analysis and design. Thus, it is natural to organize all requirements - which necessitates handling of large amounts of data - in a comprehensive computerised system based on a modern data base file.

Cowiconsult's existing bridge program library, used for the design and rating of more than 1,000,000 sq.m of bridges, and general programs from external sources will constitute the background and software for the system.

3. DEVELOPMENT OF THE COMPUTER SYSTEM

During planning and implementation of the computer system, existing bridge rating and evaluation facilities are incorporated. Thus, the following routines will be easily accessible and in most cases automatically processed:

- Standard rating of structures (inventory and operation rating)
- Standard rating of vehicles, i.e. ranking of an actual vehicle in a system of standardized vehicles used in rating of structures.
- Posting and special permit analyses of structures
- Evaluation of structurally deficient bridges, e.g. to investigate consequences of damage and risk of collapse
- Evaluation of effects of rehabilitation work (upgrading of structures)

The existing Danish bridge maintenance system leads to a considerable number of reports with a heavy flow of data during the lifetime of a bridge. Consequently, adequate updating facilities must be available in order to ensure that bridge data is always in accordance with the actual physical state of the object.

The most important part of the system will be comprehensive data base facilities enabling access to the latest information about each specific bridge. The information basically includes

- project data,
- data from inspection reports,
- information concerning repair of the bridge throughout its lifetime,
- information concerning standard ratings and heavy vehicle permits.

The project data is either original design data, or data produced by renewed structural analyses of existing bridges.

Facilities also feature reference to the DS, AASHTO, DIN and BS Standards, as well as most common construction material characteristics.

In order to manage the data efficiently and to minimize redundancy, the operation of existing bridge rating and design systems has been analysed. This work indicated that bridge data may be arranged into coherent, logical groups of data in a natural way. The following selected types indicate the principle:

- "Site information data": Any information directly related to the site. Such information is stored and updated independently. Several bridges may refer to the same site information. Typical site information comprises soils data, climatic data etc.

- "Loads": Frequently used international standard loads are stored and updated, as required

- "Materials": Library of materials and their characteristics corresponding to relevant codes including long term behaviour. Updating by introduction of results from new research is possible.

- "Geometry": Basic geometrical data for the actual bridge.

- "Statical models": Statical models covering all structural systems for each particular bridge during construction and final operational phases.

The database is divided into a total of 13 such groups of data. A specific piece of data is stored in a predestined logical part. Information is therefore retrieved from different parts of the database, as required.

Individually independent program modules carry out the dataprocessing in the system. The detailed requirements for these modules have been clarified from experience with other systems. Therefore, existing computer bridge programs in the firm have been decomposed into program routines whose abilities match the requirements of the program modules. To complete the system, several new program modules are being developed.

Design, rating, and evaluation programs for structural analyses include the following facilities:

- Composition from one or more structural materials
- Regard to effects from variation of material properties with time. The variation is either based on actual reports, theoretical work, or standards.
- Regard to all concepts of structural systems.
- Regard to influence of the construction process used.

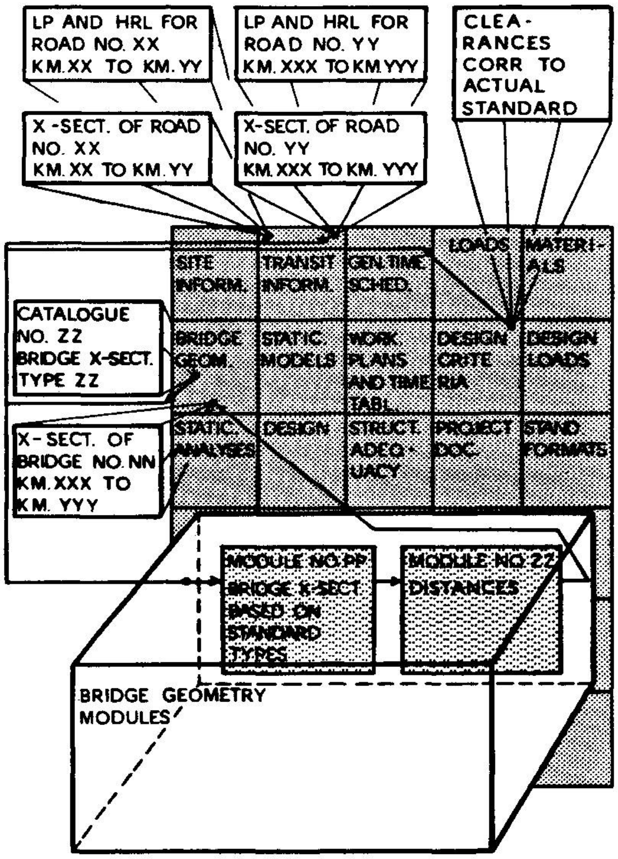

Fig. 3 Main Principles of the computer system

Furthermore, it will be possible to compare the feasibility of different rehabilitation projects in order to optimize the remaining life and costs of the structure.

Data is documented by using program modules and results may be printed in a standard format or as plotted construction drawings and bar schedules.

The above-mentioned structure may be illustrated by the dataflow in the system at a preliminary design stage, as indicated in fig. 3. Original road geometry data is combined with a proposed standardized cross section in a program module which then produces data describing the actual bridge cross section.

On completion, the Danish system will comprise facilities for rating, evaluation and design of the most common types of bridges in steel and concrete , whether cast-in-place, precast, segmentally constructed (balanced or span by span), or incrementally launched. The bridges may be simple span, continuous single or multiple frames and contain external members, e.g. cable stays.

The development work is now in progress, scheduled to be completed by end of 1983.

The programming language is standard FORTRAN 77, and the development is being carried out on a computer system comprising PRIME 550 and PRIME 850 computers.

4. MAINTENANCE AND FUTURE DEVELOPMENT

The modular concept of the computerised bridge rating system ensures that a continuous updating can take place in the future. Code revisions, new codes, as well as changed design procedures and other future bridge management requirements will be introduced as required by special staff, who will also provide the technical support - and back up services to external users.

5. REFERENCES

[1] H.H. GOTFREDSEN: "Economic aspects in planning of bridge rehabilitation and repair",
IABSE Symposium on Bridge Rehabilitation and Repair, 1982.

[2] OECD-Road Research Programme Research Group CM 2, "Evaluation of Load Carrying Capacity of Existing Road Bridges".
Final Report, Paris, 1979.

[3] SCHELLING D.R., "Survey of Bridge-Orientated Design Software", Office, Chief of Engineers, U.S. Army, Washington D.C., 1980.

[4] "Inspection, maintenance and repair of road and railway bridges", International Conference on Gestion des Ouvrages d'Art.
Volume I and II, Bruxelles - Paris, 1981.

[5] "The Deterioration of New York State Highway Structures", New York State Department of Transportation, New York, 1980.

Collapse of the Viennese Reichsbrücke: Causes and Lessons

Effondrement du pont Reichsbrücke à Vienne: causes et leçons

Einsturz der Wiener Reichsbrücke: Ursachen und Lehren daraus

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SUMMARY
The paper describes in a simplified manner the essential peculiarities of the Viennese Reichsbrücke which led to increasing local splitting forces in the piers for decades. Supervening temperature stresses at the critical location in an unloaded state finally produced the collapse of the whole bridge.

RESUME
L'article résume les caractéristiques principales du pont Reichsbrücke à Vienne, lesquelles ont conduit – au cours des décennies – à une accumulation de tensions d'éclatement dans les piles. Les contraintes thermiques, en augmentation aux endroits critiques, ont finalement causé l'effondrement total du pont, et ceci en l'absence de charges de trafic.

ZUSAMMENFASSUNG
Der Aufsatz schildert in vereinfachter Form die wesentlichen Besonderheiten der Wiener Reichsbrücke, die örtlich zu jahrzehntelang zunehmenden Spaltzugkräften in den Pfeilern geführt haben. Den Einsturz der gesamten Brücke im unbelasteten Zustand lösten schliesslich an der kritischen Stelle hinzukommende Temperaturspannungen aus.

1. SPECIAL FEATURES OF THE STRUCTURE

Sunday, August 1^{st}, 1976, a quarter to five a.m. the suspension bridge covering 10 000 m^2 collapsed practically without live load - only two automobiles were on the bridge - and claimed the loss of one human life. It seemed that the collapse originated from the failure of a river pier. In figure 1, the cross section through the bridge and the concerned river pier, this fact is displayed by omission of the plunged parts of concrete and ashlar.

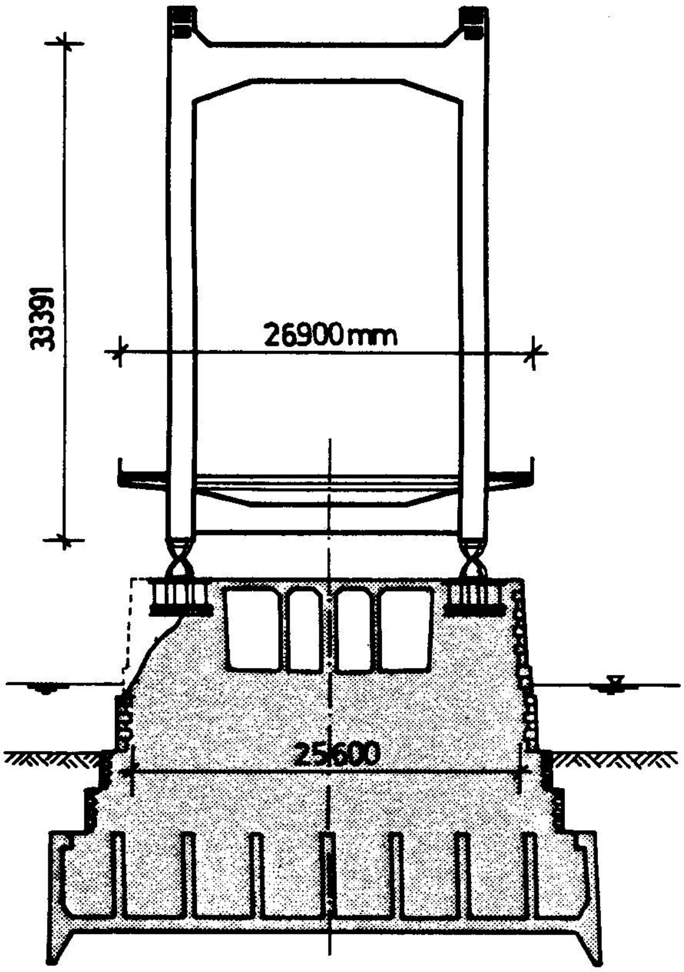

FIG. 1: CROSS SECTION AT PIER XVIIa

Yet, before the essential cause of the collapse will be treated, a few peculiarities of the structure and of the pier are to be described, which finally led to the catastrophy (in combination with the influences of creep and temperature).

1.1 The statical system (fig. 2)

The iron suspension bridge, built in 1936, was originally planned as a genuine suspension bridge. Only during the erection of the

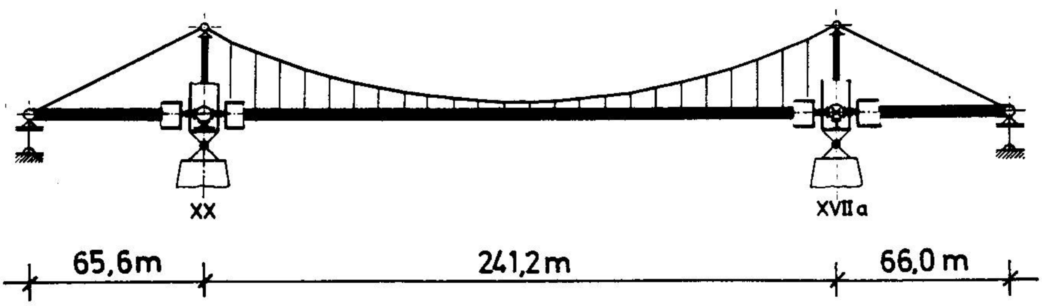

FIG. 2: STATICAL SYSTEM

stiffening girder in the main span the statical system of the bridge was changed to a self anchored suspension bridge due to rather strong differences in the judgement of the soil properties; therefore it was necessary to anchor the horizontal tension of the chains to the suspended stiffening girder and strengthen it correspondingly.

This measure led to an unusual statical system with hinged connections of the stiffening girder at the piers, penetrating the pylons which acted as columns hinged at both ends. For the main loads this system was statically determinate. Failure of one essential member was sure to lead to the collapse of the whole structure.

1.2 The bearings under the pylons

Each of the bearings was made of cast steel, designed for a working load of 90 000 kN of which 70 000 kN were dead load. The hinge function in any direction was achieved by the contact of two spherical surfaces with different radius of curvature. In order to distribute the load over an adequate contact area the radii of curvature of both spherical surfaces were only slightly different, that is 6 000 mm and 9 000 mm. Therefore, the bearings were sensitive to inclinations of the pylons responding with relatively great excentricities of the load.

1.3 The river piers

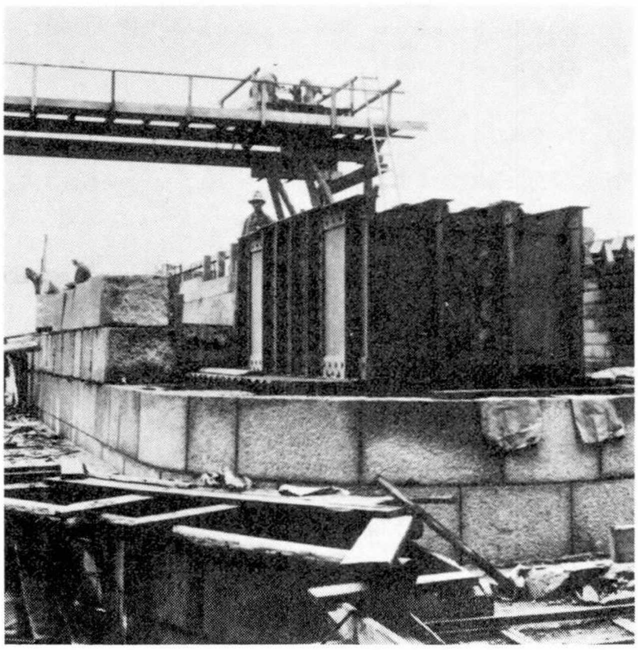

FIG. 3: PIER XVII a DURING ERECTION (1935)

Under each pylon bearing a heavy grid of bolted steel girders of 1,60 m hight was concreted into the head of the pier. For better distribution of the load the grid rested upon rolled I-beams layed closely to eachother (Fig.1 to 3). No other reinforcement of the piers was provided. The external dimensions of the pier shaft were just so large that the steel grid and the mat of rolled beams found enough space between the headers of the ashlar.

The unreinforced concrete just under the rolled beams had the relatively high cylinder strength of 55 N/mm^2, measured after the disaster. (The maximum pressure under full load below the mat of 4,5 N/mm^2 was comparatively small). Lower concrete layers showed various cylinder strengthes down to 24 N/mm^2. There is no doubt that these relatively high compression strengthes developped only in the course of the 40 years coninuous hardening of the cement paste. Certainly the cement qualities of those days showed the characteristic of longtime hardening to a great extent as everybody knows.

2. THE CAUSES OF THE COLLAPSE; SIMPLIFIED ESPECIALLY BY REGARDING THE PIER AS A PLANE SYSTEM

(The complete wording of the commission's expertise is given in [1]).

2.1 Creep of concrete

As subsequent measurements revealed, the stones of the granite ashlar must have been broken from a zone close to the surface of the quarry. They had a modulus of elasticity which for granite was very low indeed. The order of magnitude was about the same as the one of the neighbouring concrete. Therefore, the flow of forces underneath the grid at first corresponded rather well to that one which can be expected in a homogenious material.The dashed line in fig. 4 indicates this behaviour.

Since the granite has no ability to creep in the course of decades redistribution of forces occurred in the pier's head: Because of the unyielding granite a steadily increasing amount of the load flowed from the concrete into the granite ashlar at the front surface of the pier (indicated by the dash-dotted line in fig.4). Some time or other the splitting forces developping at the same time produced an initial crack in the concrete (heavy line in fig. 4) so that the forces due to deflection of the load path charged the squat bar developped at the left side of the initial crack in bending. With increasing redistribution of the load flow the bending stresses of the squat bar and the length of the initial crack became greater and greater.

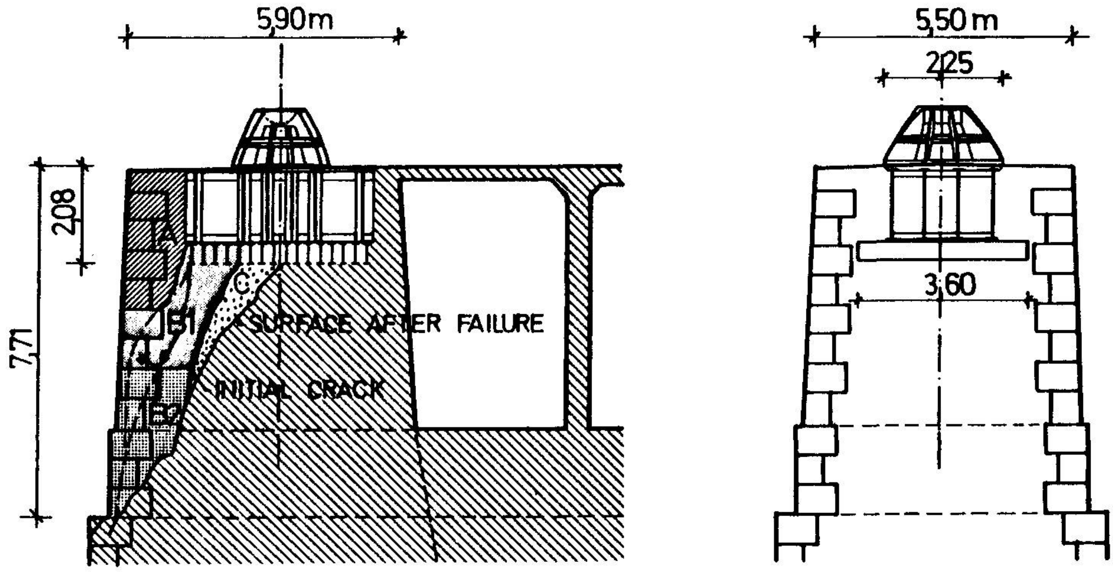

FIG. 4 : PIER XVIIa CRACK FORMATION AT FAILURE
A, B1, B2 FRACTIONS C CRUSHED PART

2.2 Temperature

Until ten days before the collapse Austria had been struck by a longlasting heatwave which had been superseded by unusually cold weather. The slowly penetrating cooling of the pier, still warm in its interior, led to the maximum of tensile stresses in the zones close to the surface after several days. Combined with the bending tensile stresses due to the redistribution of the load flow as a consequence of creep they led to the failure in bending of the squat bar to the left of the initial crack - fortunately in an unloaded state of the bridge. The mechanism can be realized from the parts B1 and B2 (fig. 4) salvaged from the Danube seven months after the disaster.

3. SUCCESSION OF FAILURES UNTIL THE TOTAL COLLAPSE

After a portion of the grid's bottom area had become unsupported due to the plunge of the granite and concrete parts the pressure at the edge of the remaining supporting area under the grid crushed the concrete. The whole grid tilted and produced great excentricity of the pylon's load in the spherical contact surfaces of the pylon bearing which bursted immediately and introduced the collapse of the total superstructure. The kinds of damages and the final position of the parts as well as the sequence of the further events are explained and interpreted in the expertise mentioned before.

4. LESSONS FOR MAINTENANCE OF BRIDGES

4.1 Unreinforced piers with stone ashlar can be endangered by time dependent splitting forces, especially when bearing loads are introduced directly into the concrete or the like between the ashlar.

4.2 Larger structures generally should be investigated from time to time with respect to new scientific findings.

For reasons of conpleteness it may be noted that the first mathematical treatment of creep was shown by Dischinger in 1937 [2] on arch bridges. He also was the first to calculate the redistribution of loads from concrete to the ashlar in a bridge pier in 1939 [3] .

REFERENCES

1. LÖTSCH, K., PAUSER, A., REIFFENSTUHL, H., SATTLER, K. and STEIN, P.: Bericht der Expertenkommission zur Beurteilung der Ursache des Einsturzes der Wiener Reichsbrücke. Der Aufbau, 1977, page 303

2. DISCHINGER, F.: Untersuchungen über die Knicksicherheit, die elastische Verformung und das Kriechen des Betons bei Bogenbrücken. Der Bauingenieur, 1937, page 487 to 621

3. DISCHINGER, F.: Elastische und plastische Verformungen der Eisenbetontragwerke und insbesondere der Bogenbrücken. Der Bauingenieur, 1939, pages 53 to 63, 286 to 294, 426 to 437 and 563 to 572

Evaluation and Improvement of Bridge Foundations

Evaluation et amélioration des fondations de ponts

Beurteilung und Verbesserung von Brückenfundamenten

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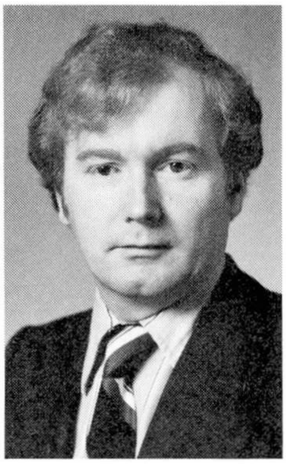

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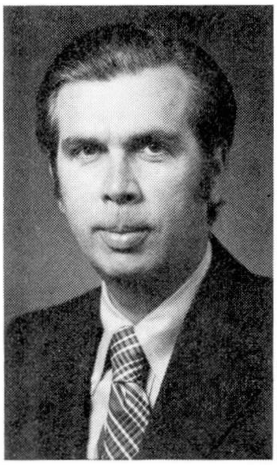

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SUMMARY
This paper reviews methods for evaluation and improvement of bridge foundations. A brief review of causes for damage in concrete structures is presented. Guidelines are provided for making a systematic evaluation of concrete substructures. Evaluation techniques including laboratory tests on concrete cores, and nondestructive test methods are described. A brief discussion of recent developments in nondestructive testing is included.

RESUME
Ce rapport traite des méthodes pour l'évaluation et l'amélioration des fondations de ponts. Un bref résumé est présenté sur les causes des dommages de constructions en béton. Des directives pour une évaluation systématique des structures en béton sont données. Des techniques d'évaluation comprenant des essais en laboratoire d'éléments en béton et des méthodes d'essai non-destructifs sont décrits. Les récents développements d'essais non-destructifs sont présentés.

ZUSAMMENFASSUNG
Dieser Bericht fasst die Methoden zur Beurteilung und Verbesserung von Brückenfundamenten zusammen. Ein kurzer Überblick über Schadenfälle an Betonbauwerken wird gegeben. Richtlinien für eine systematische Berechnung von Betonunterbauten werden aufgezeigt. Beurteilungsverfahren einschliesslich Laborversuche an Betonbohrkernen und Versuchsmethoden werden beschrieben. Weiter werden die neueren Entwicklungen in der zerstörungsfreien Materialprüfung beschrieben.

1. INTRODUCTION

There is a great need for rehabilitation, upgrading, or replacement of bridges throughout North America. Because bridge superstructures normally deteriorate faster than substructures, it is frequently possible to retain the existing substructure for supporting a replacement or upgraded bridge.

To determine the feasibility of reusing the substructure, an evaluation must be made of its present condition, need for and method of repair, load bearing capacity, and methods of increasing load bearing capacity if required. Such an evaluation can provide the basis for a decision on the technical and economical feasibility of reusing the bridge substructure.

This paper reviews causes of damage and deterioration in concrete foundation elements for bridges. A systematic approach to evaluation of condition is described. In addition to reviewing available evaluation techniques, recent developments in nondestructive testing are discussed.

2. CAUSES OF DAMAGE IN BRIDGE SUBSTRUCTURES

Damage in concrete structures can be categorized as follows:

- Freezing and thawing
- Aggressive chemical exposure
- Abrasion
- Corrosion of embedded materials
- Chemical reactions of aggregates
- Fire
- Mechanical overloading

The first five causes are discussed in detail by ACI Committee 201 [1]. Mather [2] suggests that the last two can be added to the list given by ACI 201. Concrete in bridge foundations, piers, and abutments can be affected by each of these causes.

3. STRUCTURAL EVALUATION

To determine existing condition of a concrete bridge substructure, a detailed evaluation should be made. The purpose of the evaluation is to determine the condition of the structure so that a judgment can be made of the need for and possibility of repair to return the structure to its design capacity. A judgment can also be made of the estimated life of the repaired structure. Based on the capacity of the repaired structure and requirements for the replacement superstructure, determination can be made of the need for increased load bearing capacity of the substructure.

3.1 Visual Examination

A thorough visual examination of all components of the bridge substructure is the first step in engineering evaluation. Sketches, drawings and/or photographs should be utilized so that no part of the substructure components will be overlooked in the visual examination. All accessible surface areas of the structures should be examined closely to detect signs of distress. A complete record of all observations should be made using photos, sketches, and notes describing observations. All conditions that deviate from those expected in a new structure should be reported. These include surface discoloration, surface erosion, surface deterioration, surface cracking, popouts and spalls.

Original drawings and specifications of the structure should be obtained if available. Comparison of information found in these original documents should be made with information developed during evaluation of the structure.

A general examination of the substructure should be made to determine measurements and over-all alignment of members. This can be done by use of a level and transit. Notes should be made of any evidence of settlement, deflection, misalignment of joints, or any other unplanned movements of members of the superstructure.

3.2 Tests on Cores

To analyze the substructure and determine its suitability for future use, it is necessary to develop data on concrete strength, cracks, and voids within the concrete members. In addition physical or chemical deterioration that would affect long time durability should be determined.

Direct measurement of compressive strength of concrete in an existing structure is determined by testing cores taken from the structure. Testing of cores provides a direct measurement of the in-place strength of the concrete. This procedure should be done in accordance with requirements specified in ASTM Designation C42, "Obtaining and Testing Drilled Cores and Sawed Beams of Concrete"[3]. In addition to compressive strength, the method describes procedures for determining splitting tensile strength and flexural strength if information on these properties is desired.

Enough cores should be obtained to permit petrographic and, if required, chemical analysis. Testing of drilled cores will yield information on condition of concrete to the depth of the core. For thick concrete members, it is usually more cost effective to supplement core test with nondestructive test methods that yield information on properties of concrete in the interior of a member.

3.3 Nondestructive Tests

Several procedures are available for relatively rapid determination of concrete quality near the surface of a structure. Currently available methods can be classified as rebound, penetration, and pull-out techniques [4]. Accuracy of these procedures is considerably improved if the equipment is calibrated frequently on concrete of known compressive strength and of materials similar to the concrete being evaluated.

To evaluate soundness of relatively massive concrete members such as bridge piers and foundations, knowledge of condition of concrete throughout the volume of the member is necessary. Currently available techniques used in field applications include ultrasonic pulse velocity methods and radiographic methods [4]. Corrosion activity can be evaluated by taking electrical potential measurements in the structure.

4.0 RECENT DEVELOPMENTS IN NONDESTRUCTIVE TESTING METHODS

New evaluation techniques currently under development, include pulse-echo, tomography, acoustic emission, radar, and electrical earth-resistance techniques.

4.1 Pulse-Echo

The pulse-echo method, also known as microseismic technique, is based on the properties of sound waves in solids. The major difference between this procedure and ultrasonic testing using through-transmission techniques is that access is required to only one side of the member.

Test arrangements for through transmission and pulse echo methods are shown schematically in Fig. 1. In the through transmission techniques, a mechanical pulse is generated at one face of the member and picked up by a receiving transducer at the opposite face of the member. In the pulse echo method, a receiving transducer is placed against the face of the member. The member is struck with a hammer adjacent to the receiver. The first wave to reach the receiver by the shortest path available produces a signal on the oscilliscope screen. A second wave reflects from the back face of the member, is picked up by the transducer, and produces a second signal on the oscilloscope screen. An electronic time measuring circuit measures the delay between the first and second signals representing the time taken for the signal to pass through the member twice. Total distance traveled divided by time taken gives the pulse velocity for the member.

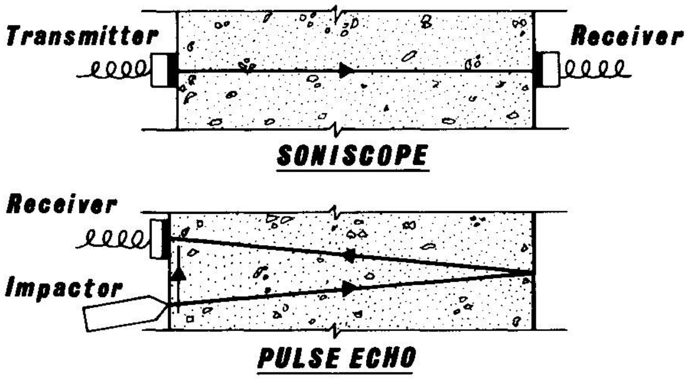

Fig. 1 Test Set-up for Through Transmission (Soniscope) and Pulse-Echo Techniques

In addition to reading of pulse velocity, discontinuities such as cracks and voids can be identified by examining the signal displayed on the oscilloscope screen. As the sound wave passes through an interface between materials of different density, part of the energy is reflected at the interface and the remainder passes through to the back face of the member. The location of the discontinuity is identified by the position of the intermediate signal on the oscilloscope screen.

The strength of the signal received from the back face gives an indication of relative crack widths. As the width of crack increases, less energy is able to penetrate to the back face. At a particular crack width, all of the energy will be reflected at the interface.

Attenuation of signal strength with no evidence of discrete discontinuities indicates porosity or low density of the cement paste causing gradual absorption and scattering of energy as the signal passes through the member.

Pulse-echo equipment for concrete testing is not widely available. However, the method has been successfully applied in evaluating many types of concrete structures including nuclear power facilities [5]. Development work is underway at a number of organizations including the U.S. Army Engineer Waterways Experiment Station [6].

4.2 Other Methods

Other methods that may have application to evaluation of bridge substructures include computerized tomography (CT) [7], acoustic emission [4], radar [8], and earth-resistance technique [9] for pile-soil systems.

5. Repair Methods

A decision to proceed with repair of concrete or strengthening of a bridge substructure should be based on a thorough evaluation using techniques and procedures described under the heading "STRUCTURAL EVALUATION". Methods for repair and strengthening of concrete structures are described in detail in Refs. [1] and [10]. Repair methods applicable to bridge foundations, piers and abutments include:

- Dry-pack mortar
- Replacement concrete
- Replacement mortar
- Preplaced aggregate concrete
- Thermosetting plastic (epoxy)
- Reinforcement installation
- Addition of new members
- Post-tensioning

Case histories of repair projects are described in Refs. [11] through [19].

PCA R&D Serial Number 1706

REFERENCES

1. ACI Committee 201, "Guide to Durable Concrete", ACI Journal, Proceedings Vol. 74, No. 12, December 1977.
2. MATHER, B., "Concrete Need Not Deteriorate", Concrete International, Vol.1, No. 9, September 1979.
3. ASTM Designation C42, "Obtaining and Testing Drilled Cores and Sawed Beams of Concrete".
4. MALHOTRA, V.M., "Testing Hardened Concrete: Nondestructive Test Methods", ACI Monograph No. 9, American Concrete Institute, Iowa State University Press, Detroit, 1976.
5. Nuclear Regulatory Commission Marble Hill Inspection Report No. 50-546/79-07; 50-547/79-07, September 1979.
6. ALEXANDER, A.M., "Development of Procedures for Nondestructive Testing of Concrete Structures; Report 2: Feasibility of Sonic Pulse-Echo Technique", Paper C-77-11, U.S. Waterways Experiment Station, Vicksburg, Miss., April, 1980.
7. MORGAN, I.L., ELLINGER, H, KLINKSICK, R., and THOMPSON, J.N., "Examination of Concrete by Computerized Tomography", ACI Journal, Proceedings Vol. 77, No. 1, January-February, 1980, pp. 23-27.
8. ANONYMOUS, 'Radar Spots Substratum Voids Without Drilling Slab", Engineering News Record, September 7, 1978, p. 15.
9. McCARTER, W.J., WHITTINGTON, H.W., and FORDE, M.C., "An Experimental Investigation of the Earth-Resistance Response of a Reinforced Concrete Pile", Proceedings, Institution of Civil Engineers, London, Vol. 71, December 1981, pp. 1101- 1128.
10. Concrete Manual, U.S. Department of the Interior, Water and Power Resources Service, Washington, D.C., Eighth Edition, 1981.
11. LAMBERTOM, B.A., "Fabric Forms for Erosion Control and Pile Jacketing", Concrete Construction, May, 1980.
12. KOERNER, R.M., and Welsh, J.P., "Fabric Forms conform to Any Shape", Concrete Construction, May, 1980.
13. ANONYMOUS, "Special Problems Feature Epoxy Repairs on Soo Line Bridge", Railway Track and Structures, September, 1977.
14. ANONYMOUS, "Old Bridge Dressed Up and Down", Engineering News Record, March 6, 1980.
15. ASHTON, N.L., "New Bridges Found on Old", Civil Engineering - ASCE, November, 1968.
16. ANONYMOUS, "85-Year-Old Bridge Saved by New Technology", Civil Engineering - ASCE, October, 1976.
17. ANONYMOUS, "Epoxy Injection Solves Unique Pier Problems", Railway Track and Structures, March, 1978.
18. ANONYMOUS, "Stay-in-Place Forms Protect Bridge Piers", Highway and Heavy Construction, May, 1978.
19. SALKOWSKI, A.S., "Face Lifting for a New Bridge", Public Works, December, 1964

Rating and Evaluation of First-Built Composite Girder Bridge in Japan

Estimation et évaluation du premier pont à poutre mixte au Japon

Beurteilung und Wertung der ersten Verbundträger-Brücke in Japan

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SUMMARY

The paper presents a continued study on the first-built composite girder bridge in Japan, which was removed in 1978 after 25 years' service. A probability of failure at the time of the removal and the remaining life of the bridge are evaluated on the basis of probabilistic analysis from the test results of full-size specimens of structural members cut off from the bridge and the results of traffic measurements.

RESUME

Le rapport présente une étude continue du premier pont à poutre mixte au Japon, qui a été démoli en 1978, après 25 ans de service. La probabilité de ruine au moment de la démolition et la vie restante du pont sont évaluées sur la base d'analyses probabilistes de résultats d'essais d'éléments originaux du pont, découpés, ainsi que sur la base des résultats de mesures de trafic.

ZUSAMMENFASSUNG

Dieser Bericht stellt eine über längere Zeit unternommene Studie über die erste Verbundträger-Brücke in Japan, welche 1978 nach 25 Betriebsjahren abgebrochen wurde, dar. Die Versagenswahrscheinlichkeit zur Zeit des Abbruchs und die Restlebensdauer der Brücke wurden mit Hilfe einer Wahrscheinlichkeitsanalyse und aufgrund von Versuchsresultaten die an herausgetrennten Brückenbauteilen erhalten wurden sowie anhand von Verkehrsmessungsergebnissen berechnet.

1. INTRODUCTION

The paper presents a comprehensive study on the rating of deterioration and the evaluation of existing static strength and remaining life of an actual highway bridge. The bridge was Kanzaki Bridge, Osaka, Japan, consisted of 18 spans of 12 m length of composite I-beams, 7 spans of 12 m length of composite welded girders and 2 spans of 10 m length of non-composite I-beams. It was the first composite girder bridge in Japan built in 1953 and was removed in 1978 after 25 years' service to meet a further increase in traffic and to provide against the control of flood tide in the river. In 1969, the width of roadway was enlarged from 6.0 m to 8.0 m plus a footway of 2.0 m. The original design live load was 127.4 kN trucks as one of the 1st-class bridges.

On the occasion of removal, full-size specimens, such as full-span composite welded girders, reinforced concrete slabs supported at the spacing of 1.5 m and push-out specimens of block-type shear connectors, and coupons of steel and concrete cylinders for material tests, were cut off from the bridge. Simultaneously, measurements of traffic loads and volume at the site were carried out, too.

The authors tried to investigate about deterioration, existing static strengths and remaining lives of various structural members of the bridge from the test results and probability analysis. Lessons from the composite girders after 25 years' service seem to have a great significance for future plans of maintenance and rehabilitation of bridges.

2. STATIC STRENGTHS

Mechanical properties of materials and ultimate strengths of structural members are summarized in Tables 1 and 2, respectively. In Table 2, the theoretical values and the previous test results[2] obtained at the time of the original construction are shown. The test results of the members are summarized as follows:

(1) All test composite girders showed bending failure forming a full-plastic hinge at a cross section under a loading point. So, the theoretical values of the girders at the ultimate bending failure were obtained by assuming the full-plastic stress distribution at the section.

(2) One of the slab-specimens and the slab of a two-web composite test girder showed punching shear failure under a rectangular pad. Generally, the ultimate load-carrying capacity of a slab may be defined by such punching failure. But, there is no rational theory for estimating the ultimate punching load under the rectangular pad. The authors proposed a new formula of Eq.(1) by assuming the failure pattern as shown in Fig.1. The effectiveness of the equation was evaluated by the existing data of about 100 specimens. The mean value and standard deviation of the ratios of test values to theoretical ones were 0.984 and 0.078, respectively.

Table 1 Material properties

(μ : Mean value, σ : Standard deviation)

Material	E (GPa)		σ_y (MPa)		σ_b (MPa)		σ_c (MPa)	
	μ	σ	μ	σ	μ	σ	μ	σ
Steel plate	193	6.9	257	12.5	426 (402~490)	34.1	-	-
Reinforcement ϕ=12mm	208	10.4	265	23.9	380	20.5	-	-
Concrete	28.0	3.58	-	-	-	-	35.4 (24.8)	3.33 (2.13)

( ) : Previous tests

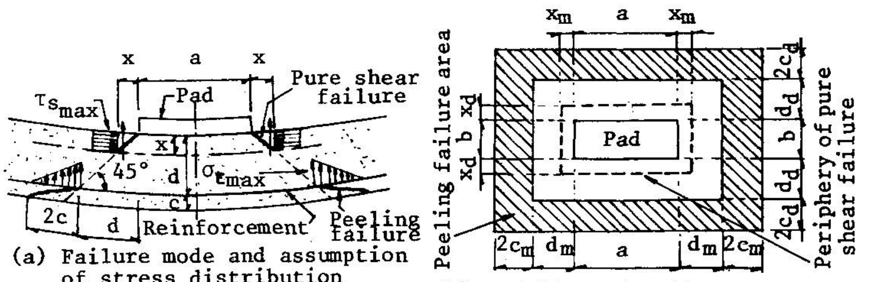
Table 2 Ultimate strengths of structural members

Structural member		Ultimate strength Exp. v.	Ultimate strength Theo. v.	Failure mode	Loading pattern
Girger	Single web	1940 kN·m (1955 ″)	1940 kN·m (1700 ″)	Flexural failure	
	Single web	2097 kN·m	1970 kN·m	Flexural failure	
	Two webs	760 kN	742 kN	Punching shear f.	
	Two webs	2528 kN·m	2764 kN·m	Flexural failure	
Shear connector		1651 kN (1658 ″)	- -	Bearing failure	
Concrete slab		608 kN	575 kN	Punching shear f.	

$$P=\tau_{s_{max}}\{2(a+2x_m)x_d+2(b+2x_d)x_m\}+\sigma_{t_{max}}\{(b+2d_d+4c_d)2c_m+(a+2d_m)2c_d\} \qquad (1)$$

(3) Seven push-out specimens were tested to examine the ultimate shear strengths of block-type shear connectors. All the specimens showed concrete crushing at the ultimate state.

At first, steel plate materials of the girders were judged to correspond to SS34 steel which was a mild steel designated by the Japan Industrial Standards. Strengths of the concrete had increased about 43 % more than the initial 28-days strength.



(a) Failure mode and assumption of stress distribution

(b) Modeling of failure zone

Fig.1 Modeling of punching shear failure.

Then, regarding the ultimate strengths of the structural members, comparisons between the present test results and the theoretical values, and between the present and previous test results gave the following conclusion. Namely, the reduction of load-carrying capacities of the structural members can be scarecely seen even after the 25 years' service. The soundness seems to have been retained by inexperience of severe cracking in the slabs, of serious corrosion and large traffic loads and volume. The main reasons for the constraint of severe slab cracking seem to be due to a small spacing of the girders of 1.5 m and a comparatively thick slab of 18 cm. In the slab tests, the average bending crack depth was estimated to be only 5 cm from the bottom surface.

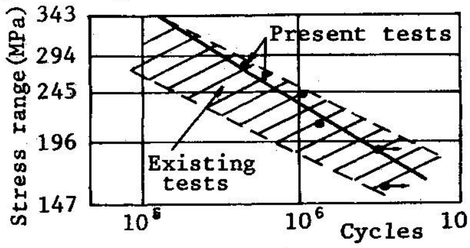


Fig.2 S-N relations of parent material.

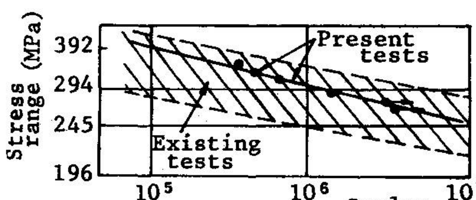


Fig.3 S-N relations of fillet weld of flange.

3. FATIGUE STRENGTHS

To investigate influences of traffic loads during 25 years on the fatigue strengths and to get S-N relations, fatigue tests of steel coupons and structural members were conducted. The coupon specimens were of parent materials, of fillet welds connecting a flange to a web plate, and groove welds of two flange plates different in thickness. The specimens of the structural members were a full-span single-web composite welded girder, three RC slabs and four push-out specimens of shear connectors. The test results were obtained in terms of S-N relations as follows:

(1) Steel plates

Three kinds of the results were plotted on S-N diagrams with existing similar data as shown in Figs.2 to 4. From the fairly good agreement between the present results and the existing data as seen in Figs.2 and 3, accumulation of fatigue damages in the parent materials and fillet welds could be neglected. For only groove welds, a clear reduction of the fatigue strengths can be seen. It seems to be not due to cumulative damages, but due to the influence of bending caused by a difference of thickness and a stress concentration on the surface of the as-welded metal. In conclusion, Kanzaki Bridge can be rated as it had not suffered from

Table 3 Loading for fatigue test of full-span girder

σ_{range} (Mpa)	Cycles ($\times 10^6$)	Loading pattern
98.0	1.5	2m
107.8	2.65	1m
112.7	0.50	

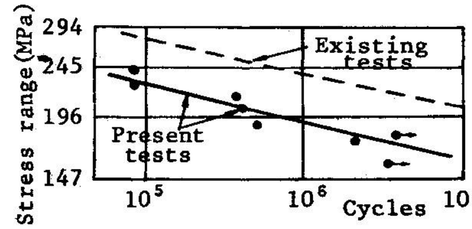


Fig.4 S-N relations of transverse groove weld of flange.

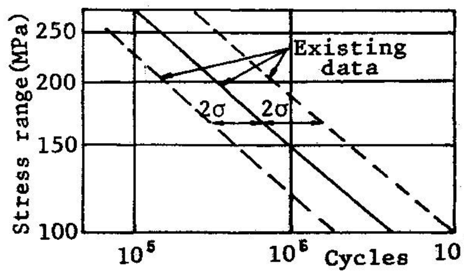


Fig.5 S-N relation of transverse non-load carrying fillet weld.

fatigue damages until the removal.

(2) Full-span composite girder

Loadings, stress ranges and loading cycles, for the test girder are shown in Table 3. At the total cycles of 4.65×10^6, a fatigue crack occurred from the toe at the lower end of fillet welds for a vertical stiffener. This fatigue failure can be evaluated with the existing data as shown in Fig. 5,[3] for transverse non-load carrying fillet welding joints.

(3) RC slabs

One of the three specimens, which was subjected to a fatigue load of 235.2~19.6 kN, showed a fatigue failure at about 2.65×10^6 cycles. This result was plotted on a S-N diagram with existing data as shown in Fig.6, which were defined by fracture of reinforcements. The other two specimens subjected to loads of 156.8~19.6 kN or 196.0~19.6 kN did not show any clear indication of fatigue even after 5×10^6 or 2.5×10^6 cycles' loading, respectively.

(4) Shear connectors

Four test values in terms of S-N relation were obtained as shown in Fig.7. Existing fatigue data for such block-type shear connectors were hardly found. So, a S-N curve was decided from these four values.

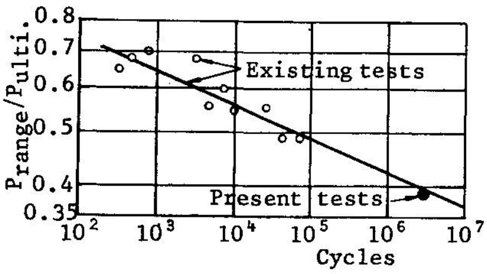


Fig.6 S-N relation of RC slab.

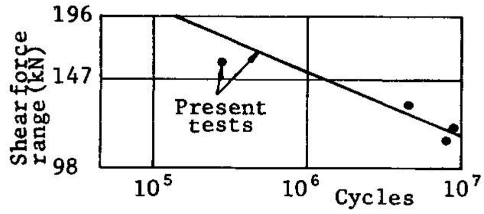


Fig.7 S-N relation of shear connector.

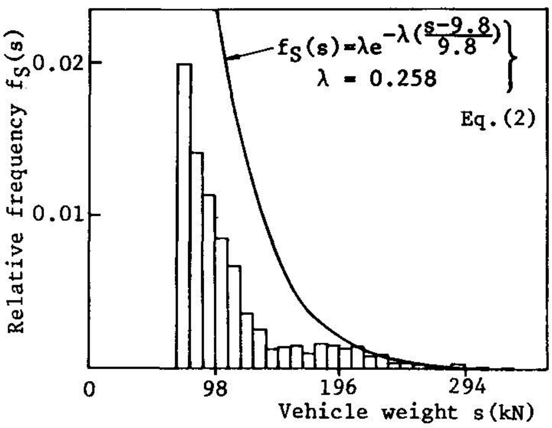


Fig.8 Observed load spectra(1980).

4. TRAFFIC MEASUREMENTS AND LOAD SPECTRA

For evaluation of probabilities of failure and remaining lives of members of a bridge, its own load spectra, loading history and traffic volume are important variables in conjuction with the resistances and S-N relations. Therefore, in 1980, traffic measurements were conducted on a temporary rampway approaching to the new Kanzaki Bridge, which has almost the same width of roadway and could be presumed to have the same traffic pattern, as the old Kanzaki Bridge. The load spectra and daily traffic volume history were obtained as shown in Figs.8 and 9. For the obtained spectra, Eq.(2) shown in Fig.8 was found to give the most fitting probability density function.

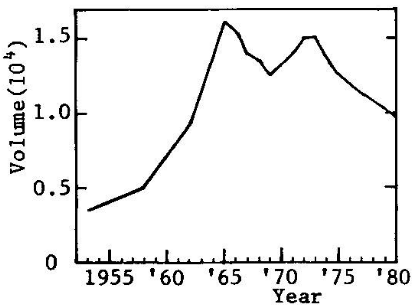


Fig.9 History of daily traffic volume per one way.

5. PROBABILITY OF FAILURE AND REMAINING LIFE

5.1 Probabilities of static failure of structural members

Characteristics of the static resistance R of the main structural members were already defined by the tests. Generally, in the field of probability of failure, serviceability states besides ultimate states have to be considered. Here, they were evaluated in terms of an allowable deflection of a composite girder and a critical slip between slab and girder. The allowable deflection is L/(20000/L) m specified at the Bridge Specification in Japan, where L is the span length of a girder. 55% of the ultimate push-out load as proposed by Johnson[4] could be used for the critical slip load. The mean value μ_r and standard deviation σ_r regarding the resistance of each member are summarized in Table 4. A normal distribution may be adopted for a probability density function to express each of the resistances.

Table 4 Resistance, load and probability of failure of structural members of Kanzaki Bridge

Member		Resistance μ_r	Resistance σ_r	Load μ_s	Load σ_s	p_f	$\beta=\frac{\mu_r-\mu_s}{\sqrt{\sigma_r^2+\sigma_s^2}}$
Girder (Moment)	Ultimate strength	kN·m 1799	kN·m 254	kN·m 21.3	kN·m 16.7	1.2×10^{-12}	6.98
	Deflection	kN·m 519	kN·m 51.8	kN·m 21.3	kN·m 16.7	5.8×10^{-10}	9.14
Shear connector (Load)	Ultimate strength	kN 1812	kN 272	kN 12.4	kN 9.7	6.1×10^{-12}	6.62
	Critical slip	kN 979	kN 82.1	kN 12.4	kN 9.7	1.8×10^{-11}	6.57
RC slab (Load)	Ultimate strength	kN 738	kN 142	kN 15.2	kN 11.2	4.9×10^{-8}	5.13

Subsequently, the load effect S for each member in the bridge has to be determined by conversion of the vehicle load spectra with distributions of path of wheels and proper influence lines for aimed points of the members. For the former, the data obtained by the Ministry of Construction in Japan as shown in Fig.10 are available. For the latter, reasonable influence lines are obtained from the three-dimensional analysis. The mean value μ_s and standard deviation σ_s regarding the load effect for each member are obtained as shown in Table 4. The probability density functions for the load effects become exponential distributions. Finally, the probabilities of failure p_f of all the members are determined together with their safety indexes β as shown in Table 4. The probabilities of failure for all the members are very small compared with conventional probabilities of failure of $10^{-4}\sim10^{-6}$. These results seem to show that Kanzaki Bridge were almost sound at the time of removal.

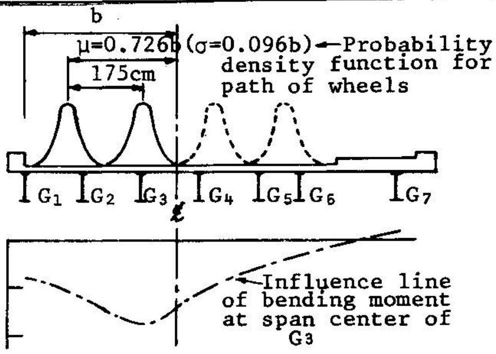


Fig.10 Distributions of path of wheels and influence line.

5.2 Probabilities of fatigue failure of structural members

A reliability function L(n) for fatigue proposed by Ang[5] is

$$L(n)=\exp\left[-\left\{\frac{n}{\bar{n}}\cdot\Gamma\left(1+\Omega_n^{1.08}\right)\right\}^{\Omega_n^{-1.08}}\right]. \tag{3}$$

Using the S-N curves in Chapter 3 and Eq.(3), each reliability for fatigue of the main members could be calculated as in Fig.11. From the figure, the reliability for fatigue, L(n), probability for fatigue, (1-L(n)), at any time, or the remaining life under a certain reliability can be calculated. Table 5 is the results of probabilities of fatigue failure of the members at the time of removal. It can be seen from the results that the Bridge had had also a sufficient safaty for fatigue at the time of removal.

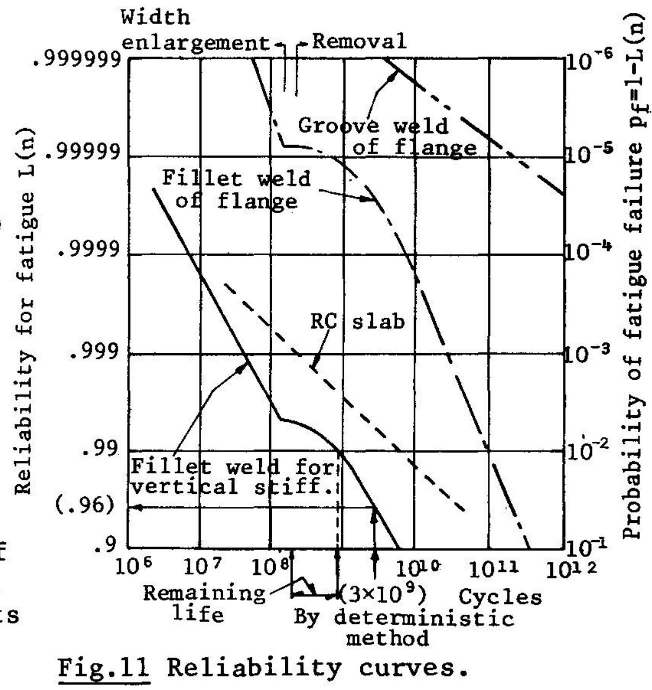


Fig.11 Reliability curves.

5.3 Probability of fatigue and remaining life of Kanzaki Bridge

To tell the truth, probability of failure and remaining life of a bridge has to be evaluated against its system failure. However, mechanism of the system failure is unsolved yet and existing data are hardly found out. Therefore, the authors have to consider at this time that probability of failure and remaining life of a bridge may be defined by the maximum probability of failure and the smallest remaining life among those obtained indpendently of each member.

So, the probability of failure of Kanzaki Bridge is evaluated to be 4.87×10^{-8} in terms of the ultimate strength of RC slab. The probability seems to be less by 2-order than the conventional values of $10^{-4}\sim10^{-6}$.

Table 5 Reliabilities L(n) and probabilities p_f for fatigue of members

Member	L(n)	p_f=1-L(n)
Flange (Parent)	.999999999	3.0×10^{-9}
Fillet weld of flange	.9999925	7.5×10^{-6}
Groove weld of flange	.99999970	3.0×10^{-7}
Fillet weld of v. stiff.	.9955	4.5×10^{-3}
Concrete slab	.99952	4.8×10^{-4}
Shear connector	.99999995	5.0×10^{-8}

Table 6 Remaining life for fatigue at toe of fillet weld of v. stiff.

	L(n)=0.999	L(n)=0.99
Remaining life	-20 years	100 years

The remaining life can be considered with the occurrence of a fatigue crack at the toe of the lower end of fillet weld for the vertical stiffener at the span center of G_3-girder. Because the probability of fatigue failure at the toe is the maximum and the occurrence of the fatigue crack was recognized at the full-span girder test. However, here, a simple question comes out that how high reliability level has to be taken for evaluation of the remaining life. To answer the question, the authors calculated the fatigue life under the actual load spectra at the reliability for fatigue of 95 % about the fatigue crack at the toe in the girder test by a deterministic method using Miner's rule. The obtained fatigue life was about 3×10^9cycles as seen in Fig.11. Then, the reliability level by the reliability function corresponding to the life was found to be about 0.96. It showed that the both reliabilities obtained by the deterministic method and by the reliability function were almost coincident with. From this experience, the authors think that the reliability level for evaluation of the remaining life from the reliability function is enough with 0.99 from a viewpoint of safety side. As the result, the remaining life of Kanzaki Bridge could be estimated to be about 100 years.

6. CONCLUSIONS

Through the tests of full-size girders or specimens and the probability analysis, the followings are concluded for Kanzaki Bridge:

(1) Deterioration, reduction of the static strength and cumulative fatigue damage of Kanzaki Bridge were scarecely observed after 25 years' service.
(2) The probability of failure of Kanzaki Bridge is less by 2-order than the conventional values of $10^{-4}\sim10^{-6}$.
(3) The remaining life of Kanzaki Bridge seems to be 100 years more under the same traffic load spectra and traffic volume as observed in 1980.
(4) The soundness of Kanzaki Bridge was dependent on the careful design to choose a small spacing of girders and a thick slab, the use of quality assured materials and rather smaller traffic loads and less volume.

REFERENCES

1. MAEDA, Y. *et. al.*, Behavior of First-Built Composite Girder Bridge in Japan, Proc. IABSE 11th Congress, Vienna, 1980.
2. KONDO, K. and A. MATSUNAGA, Design and Fabrication of Composite Girders of Kanzaki Bridge, Osaka, J. Civil Engineering, Vol.8, No.9, 1957. (in Japanese)
3. MEADA, Y., Fatigue Cracks of Thin-Walled Plate Girders, TRR 664 Bridge Eng., Vol.1, TRB, 1978.
4. JOHNSON, R.P. and R.J. BUCKBY, Composite Structures of Steel and Concrete, Vol.2, Constrado Monographs, GRANADA, 1979.
5. ANG, A. H-S., Structural Reliability and Probability-Based Design, Lecture Note at Osaka University, May 1976.

Displacement-Induced Fatigue Cracking of a Box Girder Bridge

Fissures de fatigue dues aux déformations d'un pont-poutre

Durch Verformungen verursachte Ermüdungsrisse in einer Kastenträgerbrücke

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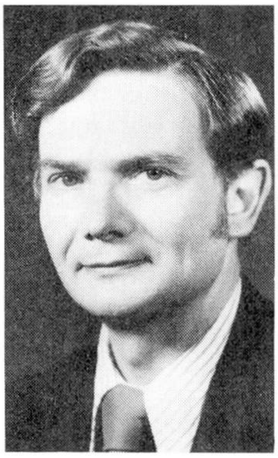

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SUMMARY
Unanticipated out-of-plane distortions as a result of the three dimensional behavior of structures have caused fatigue problems in welded bridge structures. This paper assesses the nature and causes of the distortion of the relatively small web gap of the web of curved box girders in the Ramp C Viaduct at the intersection of Interstate 695 and 83 near Baltimore, Maryland. The ultimate extent of cracking is evaluated, and a retrofit procedure is presented for the fatigue-damaged structure.

RESUME
Des déformations inattendues — hors de leur plan — résultant du comportement tridimensionnel de constructions ont causés des problèmes de fatigue dans des constructions soudées de ponts. Cet article traite de la nature et des causes du développement d'une étroite fissure dans l'âme d'une poutre en caisson courbe, d'un viaduc près de Baltimore, MD, USA. L'état final des fissures est évalué. Une technique de réparation est présentée pour la structure endommagée.

ZUSAMMENFASSUNG
Unerwartete, unebene Verzerrungen als Resultat von dreidimensionalem Verhalten von Bauwerken haben Ermüdungsprobleme an geschweissten Brückenbauten verursacht. Dieser Bericht behandelt die Art und Gründe der Verzerrung der relativ engen Stegspalte des Stegs an gebogenen Kastenträgern eines Viadukts in der Nähe von Baltimore, MD, USA. Die letzte Rissgrösse wird ermittelt und ein rückwirkendes Verfahren für ermüdungsgeschädigte Bauwerke wird beschrieben.

1. INTRODUCTION

Since the mid-1970's, fatigue problems with welded bridge structures have developed which are associated with out-of-plane displacements causing secondary web bending stresses in floor beam-girder bridges, and at cross frames and diaphragms in multiple girder bridges [1,2]. When such web bending stresses are sufficiently large and cyclical, fatigue cracking can result. Such displacement-induced cracking was recently observed at several internal diaphragm connection plates of a curved continuous box girder bridge in Baltimore, Maryland at the intersection of Interstates 695 and 83, in the Ramp C Viaduct. The characteristic cracking associated with out-of-plane movement of the web in a relatively small gap at the ends of transverse connection plates was detected after eight years of service, in the negative moment region where the transverse connection plates were not welded to the tension flanges.

Diaphragms and cross-framing are frequently used in curved box girders to prevent the cross-section from grossly distorting. It has been the practice to utilize transverse connection plates shop welded to the girder web and compression flange to connect the diaphragm members between the box girder webs. The end of the transverse connection plate is cut short or coped and fitted creating an unstiffened web gap adjacent to the tension flange. Figure 1 shows a cross-section of the curved box girder with the internal diaphragm in the negative moment region.

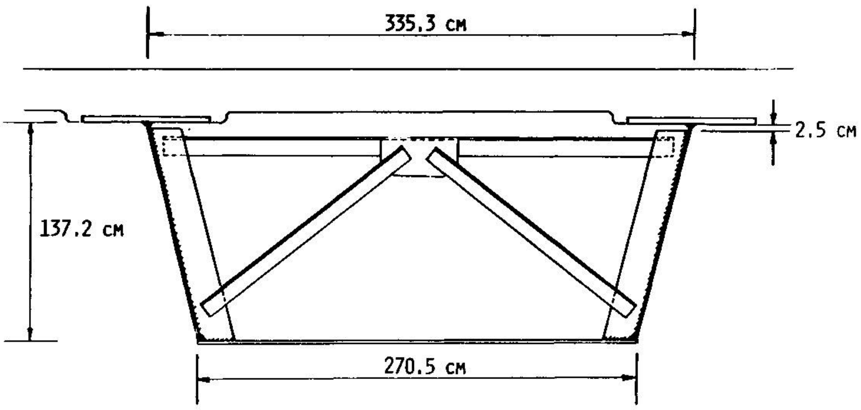

Fig. 1 Cross-Section of Curved Box Girder

2. CRACK DEVELOPMENT

The observed cracks all exhibited the characteristics that are associated with out-of-plane movement of the web in the relatively small gap at the end of the transverse connection plate. Figure 2 shows a photograph of a typical crack that was observed in the box on the outside web under the southbound lanes. Cracking has occurred in the web gap at the upper end of the transverse connection plate where no attachment was made to the tension flange in the negative moment region. This cracking is directly comparable to cracking that has been observed elsewhere in bridges with longitudinal girders and transverse floor beams.

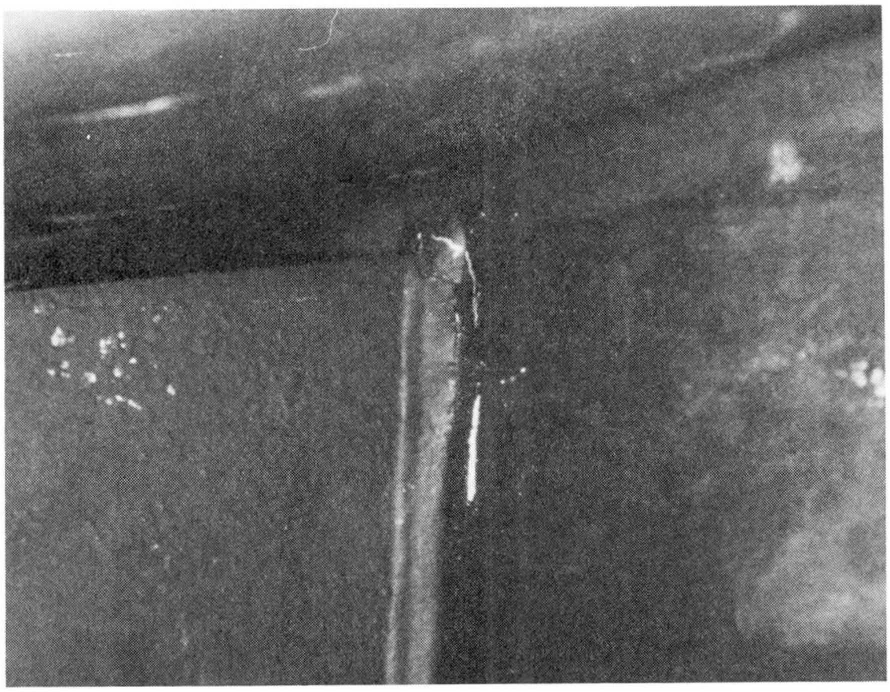

Fig. 2 Typical Crack in Transverse Connection Plate-Web Weld

When the web gap is relatively small, i.e. 2.5 cm, the movement of the diaphragm introduces large stresses on the upper end of the fillet weld connecting the diaphragm to the girder web and in the web gap. This results in cracking of the diaphragm-web weld connection at smaller welds, as illustrated in Fig. 3. If this cracking is not arrested through some retrofitting procedure, eventually between 2.5 cm and 5.0 cm of the weld will crack introducing enough additional flexibility into the girder such that the web gap is forced into greater double curvature. This distortion is shown schematically in Fig. 3. Cracking along the web-flange weld connection and longitudinal cracking in the girder web at the lower end of the diaphragm-web connection crack result from this cyclic distortion. At several locations, cracks formed in the web without significant weld cracking.

This observed cracking results from the distortion of the curved box section and subsequent loading of the diaphragms due to this distortion. This causes forces to be introduced into the girder web adjacent to the top flange, since the transverse connection plate is not welded to the flange. At the bottom end of the transverse connection plates, fillet welds positively attach the plate to the bottom flange which is acting in compression. Therefore, no cracks can develop at this welded end.

Most of the cracking was detected in the outside web of the curved box section and not in the inside web. This is a result of the direction of the out-of-plane movement. The transverse angle that is attached to the upper ends of the web connection plates is relatively stiff compared to the web gap. The outside web is pulled in on one side by the transverse angle producing cyclic tension stresses and subsequent cracking. On the other side, the web is pushed out resulting in cyclic compressive stresses. Since the effective stress range is less, severe cracking was not as extensive prior to retrofitting the structure.

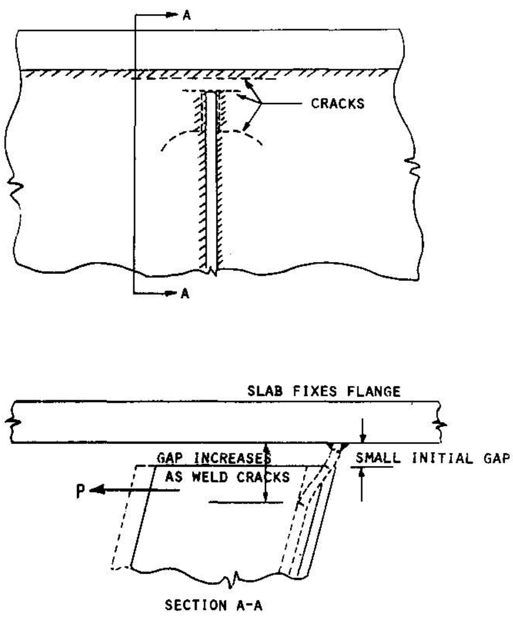

Fig. 3 Schematic of Crack Development

The cracking that has developed is a detail problem, not conceived of when this structure was designed and fabricated. Web gap cracking has only revealed itself during the past eight years. First observations of such cracking was made in floor beam-girder bridges. The cracking experienced in the Ramp C Viaduct is the first known case of displacement-induced fatigue cracking in a curved box girder structure. The development of these cracks is also related to the very high volume of truck traffic using the structure (estimated average daily truck traffic is 5100).

3. THEORETICAL ANALYSIS

One unit of the three unit Ramp C Viaduct structure was analyzed using the finite element method. SAP4 - A Structural Analysis Program for Static and Dynamic Response of Linear Systems [3] was used to perform the two phase analysis. The first phase consisted of analyzing the entire structure utilizing a gross discretization to model overall structural behavior. The resultant displacements from this and subsequent analyses were used to load the boundaries of substructure models of the region of the structure adjacent to the point of detected fatigue cracking. This second phase produced a better representation of the web gap behavior while conserving computer resources. One such substructure is shown in Fig. 4.

Using an HS 20-44 truck statically placed on the structure to represent live load, the uncracked web gap was distorted sufficiently to produce cyclic stresses varying from -7.13 MPa to +52.53 MPa. The maximum stress range exceeds the fatigue limit of the weld toes and crack propagation is unavoidable. An

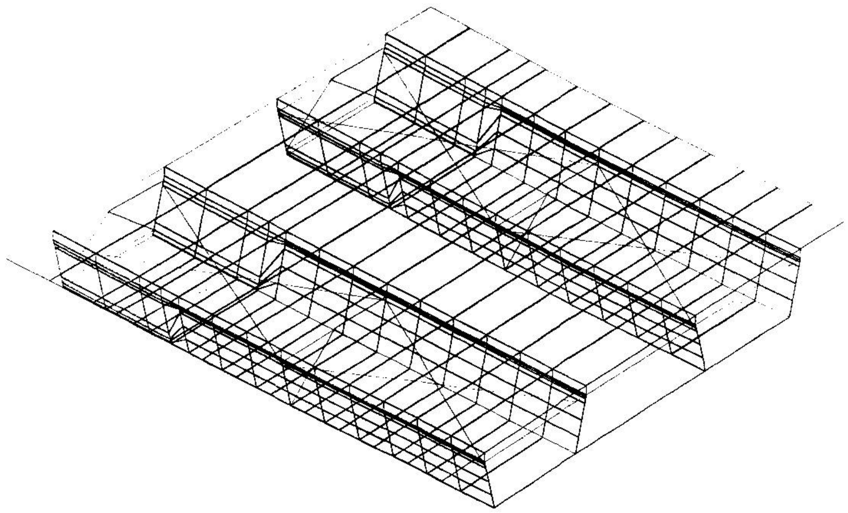

Fig. 4 Typical Finite Element Substructure Model

ongoing more extensive investigation suggests that the gap condition may be even more severe than this preliminary study indicates. The stress range in the gap, during passage of the HS 20-44 truck, may be as high as 90 MPa. Under random variable truck traffic, an effective stress range of 45 MPa ($0.35^{1/3}$ times stress range H20) would result in visible cracking from the estimated 15 million vehicles that have crossed the structure since it was placed in service. Hence, the observed behavior is in agreement with experiment test data and predicted crack propagation.

4. RETROFIT PROCEDURE TO IMPROVE FATIGUE LIFE

The only conceivable possibility for retrofitting this detail in the negative moment region is to provide a positive attachment between the transverse connection plate and the flange where web gaps now exist. This was accomplished by welding a shear tab to the transverse connection plate and flange, as shown in Fig. 5. This attachment provides a Category C fatigue condition for the flange plate which is not significantly different than the connection plate-web weld.

It is not feasible to cut back the connection plate, increasing the gap length to provide additional flexibility as an alternate solution to the out-of-plane movement problem. The diaphragms in the curved box section are carrying load while maintaining the cross-sectional shape. Cutting the connection plates back would alter the structural behavior and would likely result in more adverse behavior than is currently being experienced and continued fatigue cracking in the girder web.

The corrective action at the Ramp C Viaduct at the intersection of Interstates 695 and 83 has arrested continued crack propagation. Since the existing flange-web weld cracks are parallel to the normal cyclic stress, no further crack propagation is possible once the web gap distortion is prevented. This ensures that the desired life of the structure can be achieved without further adverse cracking.

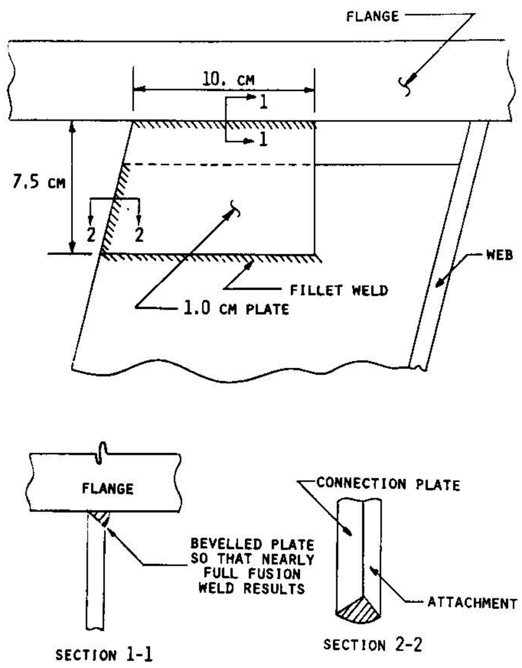

Fig. 5 Detail for Retrofit Procedure

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REFERENCES

1. FISHER, J. W., Fatigue Cracking in Bridges from Out-of-Plane Displacements. Canadian Structural Engineering Conference, 1978.

2. FISHER, T. A., FISHER, J. W., KOSTEM, C. N. and MERTZ, D. R., Design and Retrofit for Fatigue Damage in Web Gap. IABSE Colloquium, Lausanne, March 1982.

3. BATHE, K.-J., WILSON, E. L. and PETERSON, F. E., SAP IV - A Structural Analysis Program for Static and Dynamic Response of Linear Systems. EERC Report No. 73-11, University of California, Berkeley, California, 1973.

Fatigue Life of Australian Railroad Bridges

Résistance à la fatigue de ponts-rails australiens

Ermüdungsfestigkeit australischer Eisenbahnbrücken

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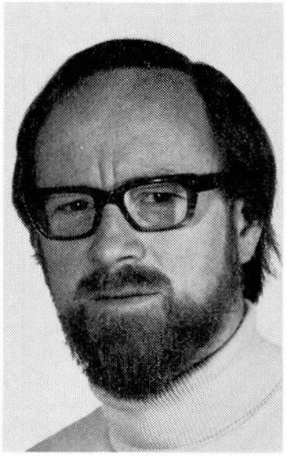

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SUMMARY
A series of Australian railroad bridges with identical welded coverplate details are reviewed for expected fatigue life. A method of determining complete sample stress histories from traffic records is developed, with the key factors of influence line and impact being confirmed by field measurements to give a reliable estimate of fatigue loading. Difficulties are encountered in identifying the fatigue category of the detail. Laboratory tests are used to give guidance on crack growth characteristics. Although confident estimates are made of expected life, the risk of premature failure of any one detail in such a large number remains difficult to estimate.

RESUME
Une étude de résistance à la fatigue est entreprise sur une série de ponts-rails australiens présentant les mêmes détails de couvre-joints soudés. Une méthode est proposée pour l'établissement des contraintes en fonction des charges de trafic, prenant en compte la ligne d'influence. La méthode est contrôlée par des mesures in situ et permet ainsi de calculer avec précision les contraintes dues à la fatigue. Des essais en laboratoire ont permis de prévoir les caractéristiques et l'évolution des fissures. Bien que la durée de vie restante peut être prévue avec une bonne précision, il reste le cas de la rupture prématurée de n'importe quel détail constructif, qu'il est difficile de prévoir.

ZUSAMMENFASSUNG
Eine Anzahl australischer Eisenbahnbrücken mit gleichen geschweissten Gurtplatten wird bezüglich ihrer Ermüdungslebensdauererwartung untersucht. Ein Verfahren für die Bestimmung der Spannungsgeschichte aus den Verkehrsaufzeichnungen, unter Einbezug der Einflusslinien und der Stosszuschläge die durch in situ-Messungen bestätigt werden, wird präsentiert. Schwierigkeiten werden in der Bestimmung der Ermüdungskategorie der Details angetroffen. Laborversuche an Schweissdetails werden verwendet, um Auskunft über die Charakteristik des Risswachstums zu erhalten. Obwohl zuverlässige Schätzungen über die erwartete Lebensdauer der Brücken gemacht werden, bleibt das Risiko des vorzeitigen Versagens eines Details, infolge der grossen Anzahl, schwierig abzuschätzen.

1. INTRODUCTION

In 1962 a new rail track was opened between Melbourne and Albury in the State of Victoria, completing the standard gauge link between Sydney and Melbourne shown in Fig. 1. Nearly all the bridges were of three standard spans of approximately 7m, 10m and 13m, consisting of stringers and open decking. The designs use 24" x 7.5" x 95 lb/ft. taper flange girders, which were the largest available, with cover plates. As was fashionable, the coverplates were terminated where they were no longer required to keep stresses within allowable limits. A typical coverplate termination detail is shown in Fig. 2, and a photograph of one ready for fatigue testing is shown in Fig. 3.

Recognising the potential fatigue problem, in the light of more recent research, Victorian Railways began an investigation to establish the risk of fatigue failure, and to establish the remaining life before remedial work should begin. [7-10]

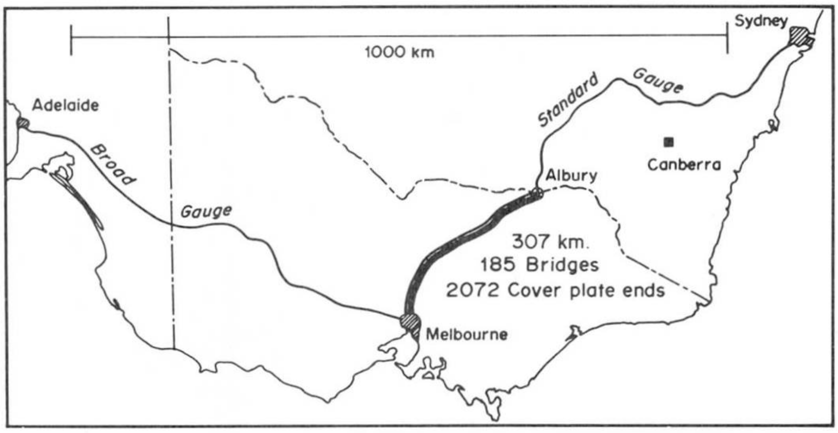


Fig. 1 Location of Bridges under Review.

A noteworthy aspect of the problem is the large number of similar structures, presenting over 2000 locations at which a fatigue crack can form, each on a nonredundant load path. Further, each structure is subjected to the same traffic. The rewards for establishing the fatigue reliability of each representative girder with confidence are high.

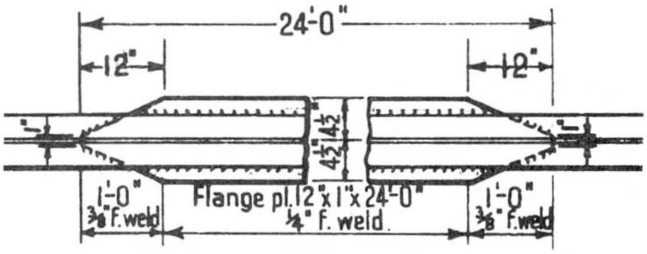


Fig. 2 Detail of Coverplate (from the original drawing).

This paper describes the steps taken to quantify the loads, their fatigue damage effect, the fatigue resistance of the detail, and hence the remaining fatigue life in probabilistic terms.

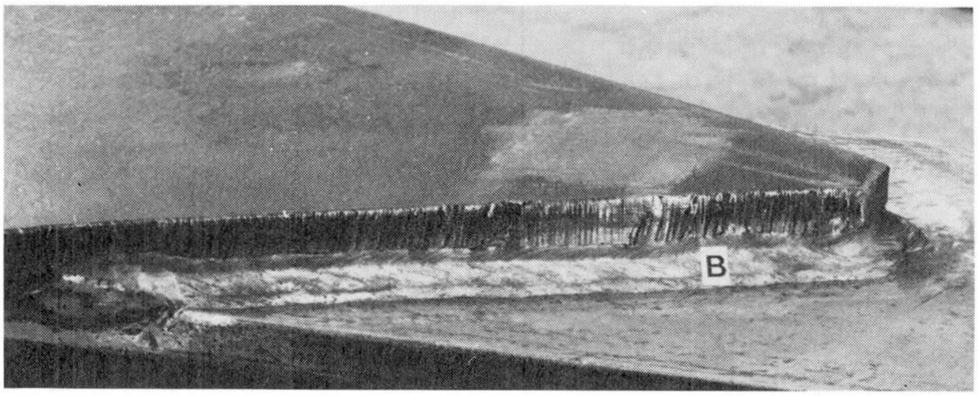


Fig. 3 View of Coverplate terminations

2. REQUIREMENTS FOR PREDICTING FATIGUE LIFE

The two primary requirements for predicting fatigue life are correct estimates of the damage index or fatigue loading, and of the damage sensitivity or fatigue resistance. The relationship between these two parameters determines the reliability of the system in terms of remaining life with an appropriate probability of exceedance [1-4,12].

The fatigue damage index is here defined as $\Sigma D(\Delta\sigma)$ at a specified location on a structure over a specified period, where D is the function relating $\Delta\sigma$, stress amplitude, to damage per cycle, and the sum applies to all stress amplitudes occurring in the specified period. Typically $D(\Delta\sigma) = K(\Delta\sigma)^3$. The index can be determined by field measurements or from traffic data subjected to calculated transfer functions to transform it into a fatigue damage index. There are limitations to either approach, most of which can be eliminated by using both.

In the context of bridges, fatigue damage sensitivity is dealt with by classification of structural details into categories [5], found in all codes of practice. Stress concentration, the use of welding, and other factors influencing fatigue resistance are all encompassed by the category, so that only the ambient stresses need be established by conventional structural analysis.

When the damage index and damage sensitivity have been established, it is possible to estimate the remaining design life of the structure at the detail under review [4]. Design life is determined from fatigue resistance curves, which are lower confidence limit curves, not mean life curves. If the estimate is correct, it implies a probability of prior failure of 0.05 in American and Australian Codes, and of 0.023 in European Codes. The owner of a railway system is reluctant to accept such a risk unless a failsafe program of inspection and maintenance can be implemented. These considerations are compounded by the large population of similar details at risk, where the cost of a failsafe program can become significant. It is possible to estimate the expected time to the first fatigue failure, but in the context of no failure being permitted the owner would like to know the expected time to the probability of first failure exceeding a low threshold, say 0.023. Such a prediction, based upon assumed functions describing the lower tail of probability distribution, would lie outside the database of laboratory tests, and would therefore carry little credibility.

3. ESTIMATION OF FATIGUE DAMAGE INDEX

For the bridges under review there exists remarkably complete documentation of all trains to the extent that the entire history of axle loads, spacing and sequence can be reconstructed for the whole life of the bridge. In practice sample days are chosen to represent typical traffic to give a daily damage rate. The annual gross tonnage is compared with the sample day to confirm the representativeness of the sample. The procedure, detailed elsewhere [7-11], is

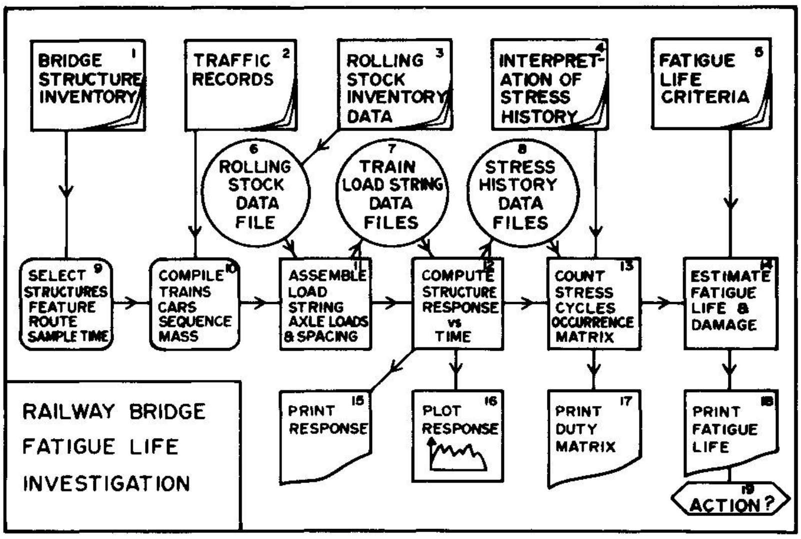


Fig. 4 Bridge Fatigue Life Flow Chart.

summarised in Fig. 4, where items 1-5 are documentary sources, items 6-8 are computer data files, items 9-10 are user inputs, items 11-14 are computer programs processing the data, and items 15-18 are outputs. Items 14 and 18 require an input on fatigue resistance. Apart from these, the entire program develops a general damage index for the structures on the chosen route.

For the given load history, item 7, the corresponding stress history is computed for a given span (influence length) and shape of influence line. The influence line is readily determined for these statically determinate structures. However, the effects of continuity of track and the dispersion of load from the top of the rail to the bottom of the girder where the critical details were located needed to be taken into account. The influence line was deduced from strain measurements with a locomotive stationed on the bridge (Fig. 5).

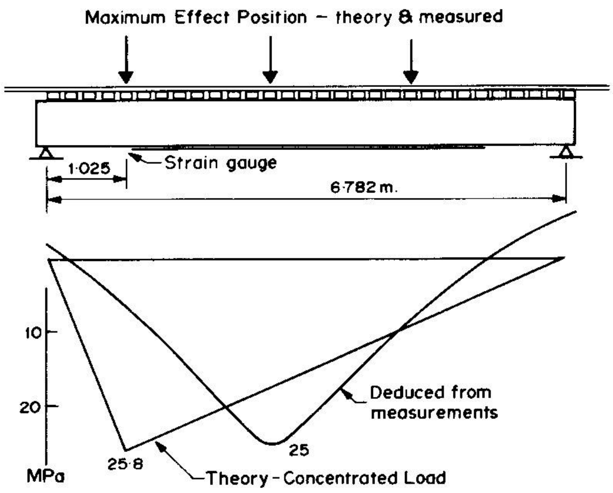


Fig. 5 Influence Lines for Stress 1.025m from support for 115.7kN wheel load.

Although the measured peak value almost agreed with the computed one, the positive influence length is shorter and there are adjacent negative influence regions as a consequence of the structural continuity of the track. For the span shown in Fig. 5, the measured maximum static stress for the locomotive was only 79% of the computed value. If the computed stress amplitudes on this simple statically determinate structure were used without calibration by field measurement the damage index would have been overestimated by nearly 100%.

4. DYNAMIC EFFECTS

In estimating stress amplitudes it is necessary to apply the correct impact factor for dynamic effects. Since it is the average impact factor which is important for cumulative damage, the maximum values used in design codes are of no use. Since data gathered on impact factors on one type of bridge of a given span cannot be applied to a different type or different span of the same type, it is invariably necessry to measure inpact factors for the bridge in question.

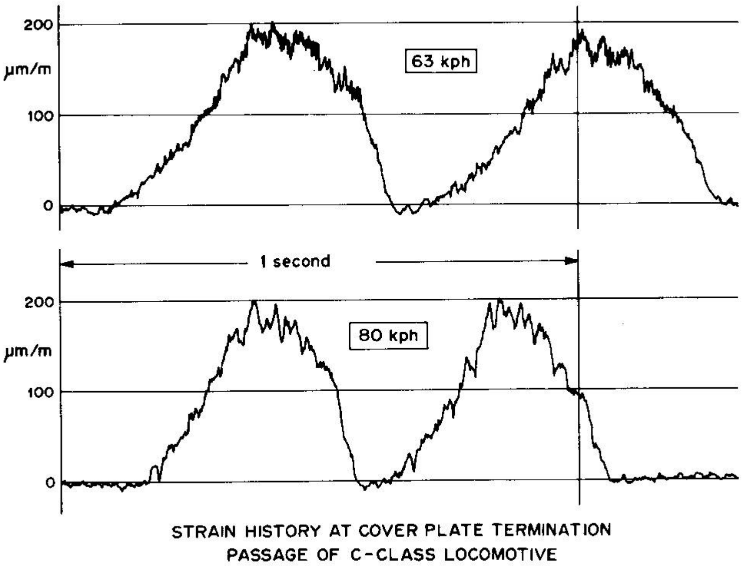
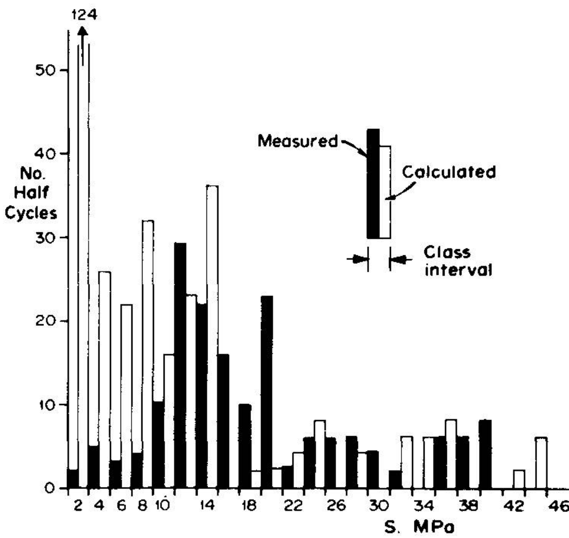


Fig. 6 Measured Response to Locomotive.

Two methods of measurement were employed in this study. In the first, dynamic traces of the of the strain at the ends of the coverplates were made for the locomotive previously used in static calibrations traversing the bridge at various speeds. Two such traces are shown in Fig. 6. Superimposed on the quasistatic response, which is not amplified, is a vibration associated with the suspension of the locomotive. This vibration adds on about

8% to the maximum static stress amplitude, and it creates a significant number of minor stress cycles not previously present. If the latter are disregarded, the dynamic effect can be included by amplifying the static stress amplitudes uniformly 8%. The method is imprecise, but vastly superior to using the impact factor specified in the relevant Code, which is 1.58.

The second method integrates the dynamic effect for a whole train in terms of fatigue damage. The strain history was recorded for two trains and then subjected to analysis by the Rainflow method [3]. For all stress cycles, the sum $\Sigma(\Delta\sigma)^3$ was formed as a measure of damage, D_{meas}. The procedures of Fig. 4 were then followed for the same two trains to generate the corresponding stress history, and the measure of damage, D_{calc} was formed in the same way, without using the 0.93 static calibration factor derived from field measurements. The expression $\sqrt[3]{D_{meas}/D_{calc}}$ is an integrated estimate of impact factor for all stress amplitudes, which includes the calibration factor relating measured static stress to computed stress.



STRESS AMPLITUDE OCCURRENCE HISTOGRAM
– 2 TRAINS

Fig. 7 Stress Amplitudes by Rainflow Method.

When applied to the 7m span under review, the second method yields a nett impact factor of 1.01. If the calibration factor remains at 0.79 for all axle configurations, the real average impact factor becomes 1.27 - much higher than the value from the first method. One reason for the difference is that the vibrations noted create more stress cycles at lower amplitudes, and the amplification factor is greater for stress cycles of lesser amplitude. Histograms of the measured and calculated occurrences of stress half-cycles are shown in Fig. 7. The number of major stress cycles corresponded between theory and measurement, with the measured maximum being 0.91 of calculated maximum. This is consistent with the lower impact factors measured with the locomotive. Low amplitude cycles were filtered out of the field measurements, but in the middle range many more cycles were measured than calculated.

It is noted that 90% of damage is done by stress cycles exceeding 16 MPa. This confirms the observation that the small cycles are not significant, and that fatigue damage estimates are not sensitive to the shape of the S-N curve at low stresses under variable amplitude.

In assessing the fatigue damage index, more confidence was placed in the second method. Taking all documentary evidence with the confirmation of field measurements into account lead to a confident prediction of the fatigue damage index. A similar effort was required to establish the fatigue sensitivity of the coverplate detail.

5. FATIGUE RESISTANCE OF COVERPLATE ENDS

Although coverplate ends have been extensively studied [5,6,13], it is evident from Figs. 2 and 3 that this series does not readily compare with those tested by others. The overlapping thick coverplate leads to a choice of USA fatigue category E' [6], proposed to allow both for overlapping coverplate and for size effect. In this case the manual fillet weld is continuous through the overlap. It is undersize by modern standards. The choice between E and E' represents a factor of 4.1 on estimated life. Two girders removed from service are being tested to give four results - not enough for statistical predictions of life, but useful for monitoring fatigue crack growth.

So far one test has run out and two have failed in the same way, by a crack propagating outwards from the root of the fillet weld (Fig. 8). The crack surfaces almost simultaneously across the full width of the coverplate end and then turns down through the side fillet welds into the top flange (Fig. 9). The two specimens which failed had virtually identical crack growth rate and form, but very different times to the initial visible crack. There appears to be insufficient data to support a change from E' classification to E, but the stable crack growth gives confidence in implementing a program of visual inspection, since the cracks are large and easily seen over a long period of the fatigue life.

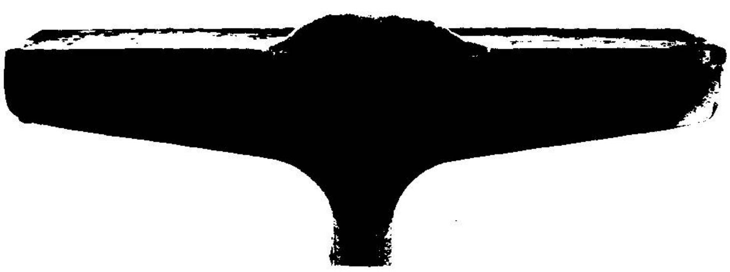


Fig. 8 Fracture Surface.

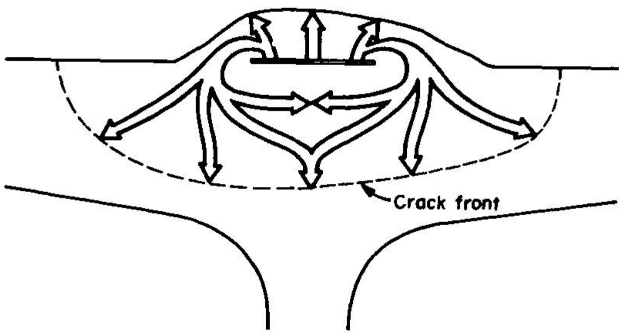


Fig. 9 Key to Crack Growth of Fig. 8.

6. CONCLUSION

This paper describes an investigation in which, unlike most situations, it has been possible to document load effects more accurately than resistance. It has resulted in a predicted life of 40 years for the coverplate end on the short span beams [10-12], based upon E' classification. As this life represents 95% probability of survival, it is practically certain that at least one beam out of the large population will have failed. Attempts to translate this finding into a credible estimate of current risk of failure, after 20 years of service, have been unsuccessful. If the detail is Class E, the estimated fatigue life becomes acceptable.

7. REFERENCES

1-4 Fatigue Reliability. Four papers by the Committee on Fracture Reliability of the Committee on Structural Safety and Reliability. Journal of the Structural Division, ASCE, Vol. 108, No. ST1, Jan. 1982. Introduction, pp. 3-23. Quality Assurance and Maintainability, pp. 25-46. Variable Amplitude Loading, pp. 47-69. Development of Criteria for Design, pp. 71-88.

5. FISHER, J.W., Bridge Fatigue Guide. American Institute of Steel Construction, New York, 1977.

6. FISHER, J.W., Retrofitting procedures for fatigue-damaged full-scale welded bridge beams. NCHRP Research Result Digest 101, April 1978.

7. GRUNDY, P., Generation of Railway Bridge Loadings of Real Trains and Their Structural Effects. Monash Uni. Civ. Engg. Res. Report 1/1980.

8. GRUNDY, P., Criteria for Evaluating Fatigue of Railway Bridges. Monash Uni. Civ. Engg. Res. Report 2/1980.

9. GRUNDY, P., Structural Response of a Standard 6.883m Span Railway Bridge to Real Trains. Monash Uni. Civ. Engg. Res. Report 5/1980.

10. GRUNDY, P., An Evaluation of the Fatigue Life of Specific Structures on the Melbourne-Albury Standard Gauge Line. Monash Uni. Civ. Engg. Res. Report 3/1980.

11. GRUNDY, P., Evaluation of Fatigue Life of Some Australian Railroad Bridges. Proc. Conf. Gestion des Ouvrages d'Art, Brussels-Paris, 1981, pp. 69-74; also Inst. Eng. Australia Metal Structures Conf., Newcastle, pp. 125-130, 1981.

12. GRUNDY, P., Fatigue as a Design Limit State for Bridges. IABSE Colloquium. Fatigue of Steel and Concrete Structures, Lausanne, pp. 69-76, 1982.

13. GURNEY, T.R., Fatigue of Welded Structures, 2nd. ed., C.U.P., 1978.