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SESSION 4

STRUCTURAL EVALUATION BY TESTING

KEYNOTE SPEAKER

Analytical and Observed Responses of Steel Girder Bridges
Comportement analytique et observé des ponts à poutres d'acier
Berechnetes und beobachtetes Verhalten von Stahlträgerbrücken

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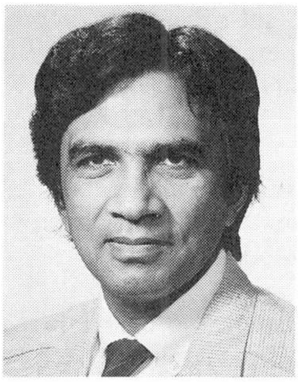

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SUMMARY
The introduction over the past thirty years or so of ever more powerful methods of analysis of bridges has brougth with it an increasing acceptance by designers of the position that "the results of the analysis must be right". This paper shows that even the most refined methods of analysis can lead to very serious errors in the prediction of bridge behaviour because of the presence of non-qualifiable behavioural factors, which by their nature cannot be included in the mathematical model on which the analysis is based.

RÉSUMÉ
Au cours des trente dernière années, l'introduction de méthodes d'analyse de plus en plus puissantes pour le dimensionnement des ponts a suscité l'acceptation de la part du concepteur que "les résultats de l'analyse doivent être corrects". Cette communication montre que même les méthodes les plus poussées peuvent conduire à de très graves erreurs de prédiction du comportement du pont, à cause de la présence de facteurs de comportement non-quantifiables, qui, par leur nature, ne peuvent pas être inclus dans un modèle mathématique sur lequel l'analyse est basée.

ZUSAMMENFASSUNG
Die Einführung immer wirksamerer Rechnenmethoden im Brückenbau in etwa den letzten dreissig Jahren hat die Statiker vermehrt zu der Einstellung veranlasst, dass "die Ergebnisse von Berechnungen richtig sein müssen". In dieser Abhandlung wird gezeigt, dass selbst die genauesten Rechenmethoden zu schwerwiegenden Fehlern führen können, weil doch immer unfassbare Verhaltensweisen mitspielen, die ihrer Natur nach im mathematischen Modell der Berechnung nicht erfasst werden können.

1. INTRODUCTION

Very significant advances in analytical techniques during the past three decades have led to the development of extremely powerful and versatile methods of structural analysis that are generally computer-based. Especially within the linear elastic range, these methods are known to predict accurate and consistent sets of results. Even methods developed from different fundamental considerations predict virtually the same set of results, thereby lending credence to each other's accuracy. Because of their rigor and perceived capability to predict the actual behaviour of structures, these advanced methods of analysis are used extensively for both the design and strength evaluation of bridges.

Predictions from rigorous analyses are usually accepted with confidence as representing the actual behaviour of bridges, and reliance on the ability of analysis to predict the actual behaviour of a bridge increases with the rigor of the method.

Jointly between them, the authors have spent more than 60 years on research related to different aspects of bridge engineering; much of this research has been conducted in the analytical aspects of bridge engineering, e.g. [1], [2], [3], [4] and [5]; they also had the opportunity of being involved in a large number of field tests on short and medium span bridges, e.g. [6], [7], [8], [9] and [10]. Through such involvement, many comparisons have been made between the observed responses of bridges and those given by advanced methods analysis. It has frequently been found that significant discrepancies existed between the predicted and observed responses, even when the loading was within the linear elastic range of the structure.

As might have been foreseen, the discrepancies between the analytical and measured responses were subsequently found to be due not to inadequacies of the methods of analysis, but rather to the presence of behavioral factors which could not be included in the mathematical modelling because of difficulties in their quantification.

Without for a moment denying their usefulness, especially in the design office, the authors have come to believe that even highly rigorous methods of analysis cannot be relied upon unquestioningly to predict the actual response of a bridge. In support of this somewhat provocative assertion, results from tests on five bridges with steel girders are presented. It is emphasized that the limitation of the examples to bridges with steel girders is due only to considerations of space availability for the paper. Similar examples, underlying the difficulties in realistic analysis, are also available for other structures. It may also be noted that results are discussed herein of only those tests in which the authors had a direct involvement. For this reason, the references cited are only those contributed by the authors.

2. BRIDGE WITH TIMBER DECKING

The first example presented is that of the rolled steel girder Lord's bridge with nail-laminated timber decking in which the wood laminates are laid transversely. As described in [11], the bridge is 6.25 m wide and has a single span that is apparently simply-supported. The girders are 10.2 m long with a bearing length of 0.53 m at each end, and rest directly on timber crib abutments. There are no mechanical devices to transfer interface shear between the girders and the timber decking although there are 100 x 200 mm nailing strips bolted to the top flanges of the girders; the decking is nailed to these strips. The Lord's bridge was tested with a test vehicle under several load levels and different longitudinal and transverse positions. Even up to the highest load level, the girders responded in a linear elastic manner. For two of the load cases, the longitudinal position of the vehicle was the same but the eccentric transverse positions were the mirror images of each other.

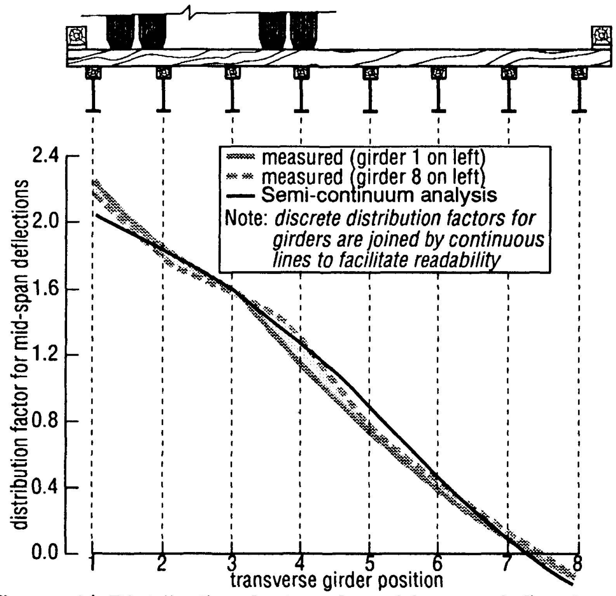

Figure 1/ Distribution factors for mid-span deflections in the Lord's bridge

For these two load cases at the highest test load level, the distribution factors for mid-span deflections are plotted in Fig. 1 by viewing the cross-section of the bridge from two different ends so that the two transverse distribution profiles overlap each other for easy comparison. It is noted that the distribution factor for deflection is the ratio of the actual and average girder deflections at the transverse section under consideration.

If the geometrically-symmetrical bridge was also symmetrical with respect to its structural response, the distribution factors for the two mirror-image load cases, noted above, would have led to transverse distribution profiles that lie exactly on top of each other. As can be seen in Fig. 1, the two profiles are fairly close to each other but not exactly the same, thus pointing out that the two transverse halves of the bridge do not respond in exactly similar manner to corresponding loads. The two sets of distribution factors obtained from measured deflections, are also compared in Fig. 1 with those obtained from deflections given by the semicontinuum method of analysis [4]. It can be seen that the analytical values of the non-dimensionalized deflections are not any more different from the two sets of observed values than the latter are from each other. This confirms that for the bridge under consideration, the semi-continuum method used for analysis is able to predict the pattern of transverse distribution of loads fairly accurately.

The same accuracy of prediction, however, cannot be claimed in the case of the absolute values of girder deflections. This is because of uncertainty in quantifying the parameters discussed below.

As noted earlier, the girders for the Lord's bridge are 10.2 m long with an unusually long bearing length of 0.53 m at each end. It is customary to assume that the nominal point-support for a girder lies midway along the bearing length, in which case the nominal span of each girder would be 9.67 m. It can be demonstrated, however, that for the case under consideration, the vertical pressure under the supported length of a girder, should have its peak away from the midway point and towards the free edge of the abutment. Determination of the exact location of this peak requires detailed knowledge of the modulus of subgrade reaction of the timber crib abutment. Clearly, this factor is not easily quantifiable thus making the task of determining the effective span very difficult. It can be appreciated readily that the clear span of the girder, being 9.14 m, is the lower-bound of the effective span of the girder.

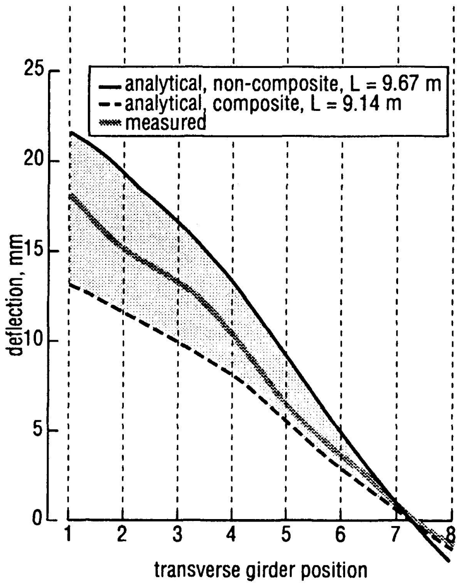

Figure 2/ Girder deflections at mid-span in the Lord's bridge

The transverse modulus of elasticity of wood, which is operative in the longitudinal direction of the bridge, is extremely small compared to the longitudinal modulus. Even if the transverse laminated deck were made composite with the girders, the contribution of the deck to the strength and stiffness of the composite section would usually be expected to be so small as to be negligible. Consequently, no attempt is usually made to provide shear connectors in such bridges. There are some holding down devices, however, to connect the deck to the girders through the nailing strips; these devices, by transferring some interface shear, do make the girders partially composite with the nailing strips and the decking. From measured girder strains, it was discovered that despite the absence of shear connectors, the decking and the nailing strips of the Lord's bridge were partially composite with the girders. The degree of composite action was found to vary from girder to girder, and clearly was not quantifiable.

The Lord's bridge was analyzed using two different sets of idealizations. In one idealization, the girders were assumed to be non-composite and with a simply-supported span of 9.67 m. In the other idealization, full composite action was assumed between the girders and the timber components, being the nailing strips and the decking; the girders were assumed to have the lower-bound span of 9.14 m. As can be seen in Fig. 2, the measured deflections for the same load case for which the distribution factors are plotted in Fig. 1, are bracketed entirely with very large margins by the analytical results corresponding to the two idealizations. It is tempting to believe that the actual condition of the bridge lies somewhere between the two sets of conditions assumed in these idealizations and consequently, errors in analysis are related only to the uncertainties of span length and degree of composite action. However, there is at least one other complicating factor, namely bearing restraint, which was not accounted for in these idealizations and which can have a significant influence on the bridge response; this factor is discussed below.

Observed bottom strains of the girders near the two abutments were generally found to be compressive, indicating the presence of significant bearing restraint forces which varied almost randomly between the girders. It was found that there was no consistent pattern in the bottom flange strains at the mid-span, these being smaller or larger than the corresponding top flange strains. This observation points towards the random, and hence deterministically unquantifiable, nature of both the bearing restraint and the degree of composite action. Because of the presence of these factors and the difficulty in the estimation of the effective span, the analysis for the bridge under consideration cannot be expected to replicate the actual behaviour of the bridge.

3. TWO-GIRDER BRIDGE

The Adair bridge is a single span, single lane structure with a clear span of 12.8 m, as shown in Fig. 3. As is also shown in this figure, the bridge comprises a concrete deck slab supported by two outer steel girders and five inner steel stringers, with the latter spanning between the abutments but also supported within the span by two transverse floor beams that frame into the two girders. A proof test on this bridge is described in [12].

Mid-span strains in the top and bottom flanges of the two girders due to two load cases are plotted in Fig. 4 against the longitudinal position of the test vehicle. It can be seen in this figure that the strains in the top flanges are always much higher than the corresponding strains in the bottom flanges. This observation confirms the presence of fairly large bearing restraint forces. Large compressive strains in the bottom flanges of the girders near their supports also confirmed the presence of significant bearing restraint which again cannot be practically quantified for inclusion in the mathematical model for analysis.

Much larger magnitudes of strains in the top flanges of the girders also point to the lack of composite action between the girders and the deck slab, this bridge not having any mechanical shear connection with the girders. Because of the lack of composite action, the top flanges of the girders getting little relief from bearing restraint at the bottom flanges, govern the load carrying capacity of the girders.

It is interesting to note that, unlike the case in the Lord's bridge and other bridges discussed later, bearing restraint does not provide any significant reserve of strength in the Adair bridge.

The uncertain nature of the composite action in slab-on-girder bridges without mechanical shear connection is underlined by the observation that, in the same Adair bridge, the inner stringers are able to develop full composite action with the deck slab despite the lack of mechanical shear connectors.

Because of the composite action, the stringers had become considerably stiffer thus relieving the non-composite girders of a much greater share of the applied loading than would have been the case if they were also non-composite. It can be appreciated that analysis cannot be very effective without the knowledge of the degree of composite action in the various beams; such knowledge is practically impossible to obtain without a test.

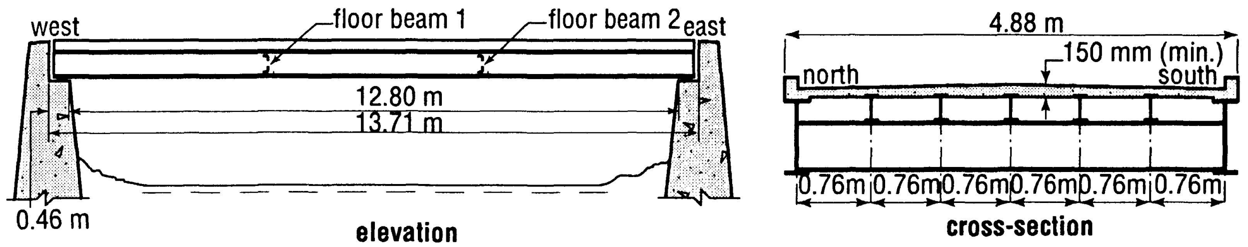

Figure 3/ Details of the Adair bridge

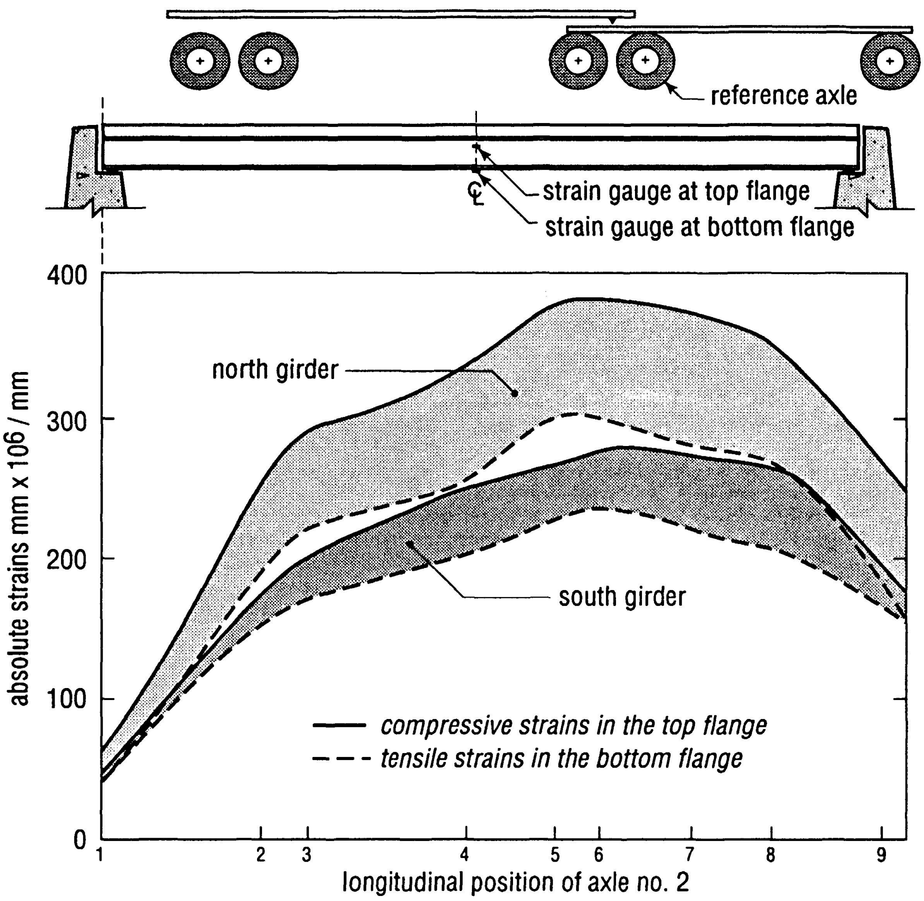

Figure 4/ Girder strains at mid-span in the Adair bridge

4. ULTIMATE LOAD TEST ON A BRIDGE

An ultimate load test on a single span, right, i.e. skewless, slab-on-girder bridge, called the Stoney Creek bridge, is described in [10]. The bridge, which had a clear span of 13.26 m, was loaded to failure in 1978 by loading it with concrete blocks piled in six layers. Girder strains at the mid-span were recorded after each layer of blocks had been placed on the bridge.

To check the validity of the recorded data, the mid-span moments taken by the girders and the associated portions of the deck slab, computed from measured strains, were compared with the total applied moments. It is recalled that in a right, simply-supported bridge, the total moment across any transverse section is obtained by simple beam analysis, and is statically determinate. When it was found that the moments computed from measured strains were up to 30% smaller than the applied moments, the accuracy of the measured data was initially questioned. An example of the comparison of moments thus computed from measured strains and average applied moments is presented in Fig. 5 for load due to one layer of concrete blocks. It is noted that the girder strains under this loading were well within the limit of computed elastic strains.

The initial computations of moments from measured strains were made by assuming that the girders were free from any horizontal restraint at the bearings. The bearing restraint forces were not initially entertained as possible cause for the moment discrepancies mentioned above. This was because bearing restraint forces of the magnitude needed to reduce the applied moments by up to 30%, were believed to be unlikely to develop in practice.

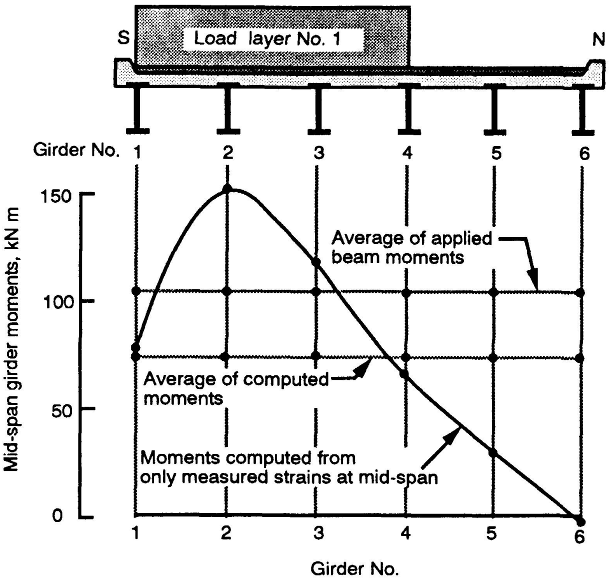

Figure 5/ Girder moments in the Stoney Creek bridge computed from observed data by ignoring bearing restraint

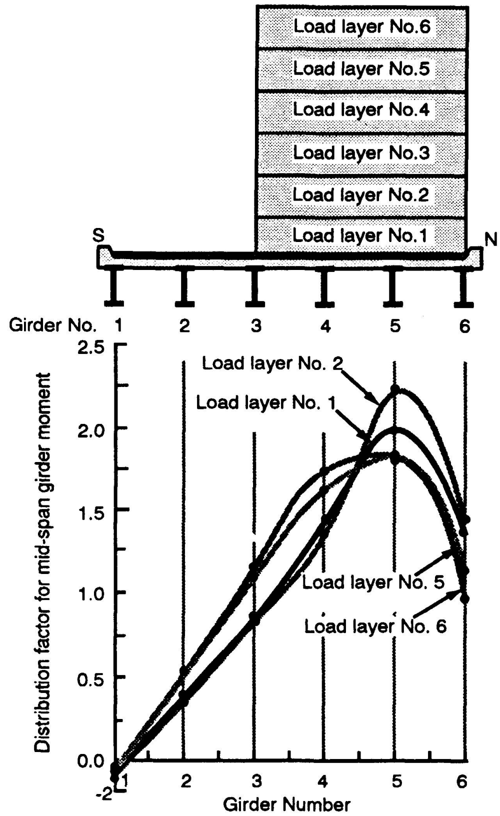

Figure 6/ Girder moments in the Stoney Creek bridge due to load at different levels

Subsequent tests, some of which are discussed herein, confirmed the presence of significant bearing restraint forces in similar slab-on-girder bridges in which girders rest upon steel bearing plates. The presence of these forces invalidates the assumption of simple supports and the compution of moments obtained from measured strains on the basis of no external forces. In light of the knowledge gained from the other tests, the data from the test on the Stoney Creek bridge were re-analyzed about ten years after the test by back-calculating the bearing restraint forces that may have occurred. From these revised computations, it was found that the bearing restraint reduced the applied moment by up to 18%, rather than the 30% range that had been wrongly deduced by previous calculations.

Distribution factors for mid-span moments taken by the girders and the associated portions of the deck slab are plotted in Fig. 6 for loads at different levels. It is interesting to note that the transverse distribution pattern of the bridge does not change very significantly as the load approaches the ultimate sixth layer. As the failure of the bridge approaches, the load gets redistributed only slightly among the most heavily loaded girders. The girder most remote from applied loading, receiving little load at early stages of loading, continues to receive low levels of load even when the total load approaches the failure load of the bridge.

An important outcome of the test was the observation that in the absence of mechanical shear connection, the composite action between a girder and the deck slab, that may exist at low levels of load, breaks down completely as the load approaches the failure load for the girder.

5. A NON-COMPOSITE SLAB-ON-GIRDER BRIDGE

The unquantifiable and random nature of the bearing restraint forces, and of the degree of composite action in the absence of mechanical shear connection, is illustrated by the results obtained from a test on the Belle River bridge [13]. The Belle River bridge is also a slab-on-girder bridge with steel girders and an apparently non-composite concrete deck slab. The nominal span of the bridge is 16.3 m and the width 9.1 m.

As indicated earlier, the transverse load distribution analysis of slab-on-girder bridges without mechanical shear connectors is made difficult, to the point of becoming impossible, by the uncertain degree of the composite action. One is tempted to believe that the actual load distribution pattern of such bridges could be bracketed by two sets of analyses: one corresponding to full composite action and the other to no composite action at all, with the former analysis always leading to safe-side estimates of the maximum load effects in the girders. In reality, a deterministic analysis, no matter how advanced, might fail completely to predict safely such maximum load effects. This assertion is illustrated below with the help of the results from the test on the Belle River bridge.

Transverse profiles of the distribution factors for mid-span girder moments in the bridge under consideration, are plotted in Fig. 7 for a transversely symmetrical load case. One of these profiles corresponds to moments computed from observed girder strains both at the mid-span and near the abutments, with the latter providing information regarding the bearing restraint forces. The other two transverse profiles are obtained from the results of the semicontinuum method of analysis [4] for the two bounds of the composite action. It is noted that no attempt was made to model the bearing restraint in these analyses.

It can be seen in Fig. 7 that the pattern of transverse distribution of actual moments is similar, but only in a general way, to the two analytical patterns. It is also quite random. Unlike the analytical patterns, the actual pattern is far from being symmetrical. In fact, the actual distribution factor for maximum girder moments is about 10% larger than the corresponding analytical factor for the fully composite bridge. It can be appreciated that the occurrence of the very high distribution factor and significant departure from symmetry are probably caused by the middle girder becoming accidentally much stiffer through composite action by bond than the adjoining girders. In light of the results plotted in Fig. 7, there can be little doubt that, for the kind of

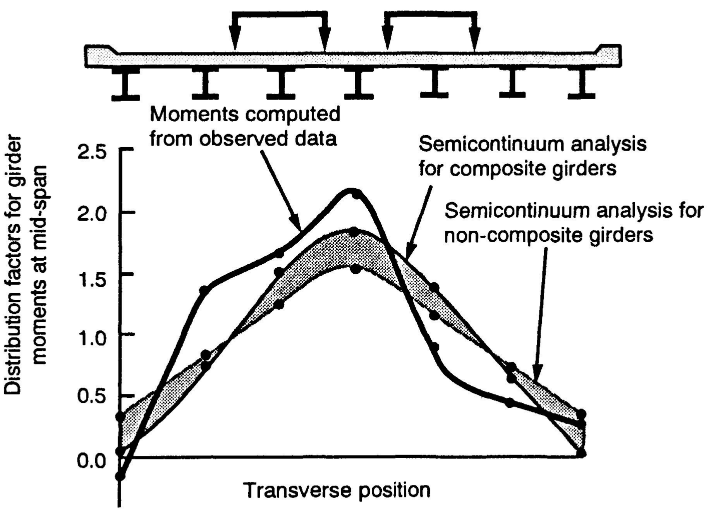

Figure 7/ Distribution factors for midspan moments in the Belle River bridge

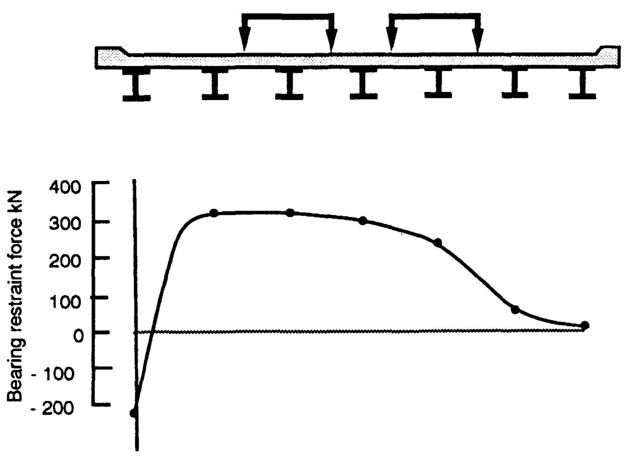

Figure 8/ Bearing restraint forces in the Belle River bridge

bridge under consideration, even the most rigorous deterministic analysis is at best only a fairly close approximation.

Bearing restraint forces in the girders of the Belle River bridge were computed from observed girder strains near the abutments. From these bearing restraint forces and approximately-calculated girder reactions at the supports, it was concluded that the effective coefficient of friction varied between 0.66 and 0.95; the former limit relates to loading by single vehicles and the latter to two side-by-side vehicles. Such effective coefficients of friction may be on the high side but are not uncommon in bridges in which the girders rest directly on highly rusted steel bearing plates.

Bearing restraint forces computed from measured girder strains are plotted in Fig. 8 for the same load case for which the distribution factors for mid-span girder moments are plotted in Fig. 7. The bearing restraint forces are shown as positive when they tend to push the abutment away from the girders.

It can be seen in Fig. 8 that the bearing restraint forces, in all the girders except one, are positive. At the location of the left hand outer girder, the bearing restraint force was found to be not only negative but also fairly large in magnitude. It is postulated that this unusual response is the result of a relatively soft pocket in the backfill behind the abutment in the vicinity of the left hand outer girder.

In light of the uncertainties discussed above, it can be seen that for the kind of bridge under consideration, no deterministic analysis can be expected to predict the actual behaviour of the bridge.

6. A NEW MEDIUM SPAN COMPOSITE BRIDGE

The examples presented so far in the paper are of relatively short span bridges in which there are no mechanical shear connectors and in which girders rest either directly on the abutment or on fairly rusted steel bearing plates. In such bridges, there may be difficulties in assessing the degree of composite action and the magnitude of bearing restraint forces. Further, because of the spans being themselves short, even small errors in the estimation of the effective span can have a relatively large influence on the computed responses of the bridge. Consequently, one might conclude that the difficulties in predicting the realistic response of a bridge are limited to only the kinds of bridge discussed earlier. It is shown in the following that errors in predicting bridge behaviour can also extend to medium span bridges in which mechanical shear connectors ensure virtually full composite action and in which the girders are supported by elastomeric bearings which apparently permit free longitudinal moment of the girders.

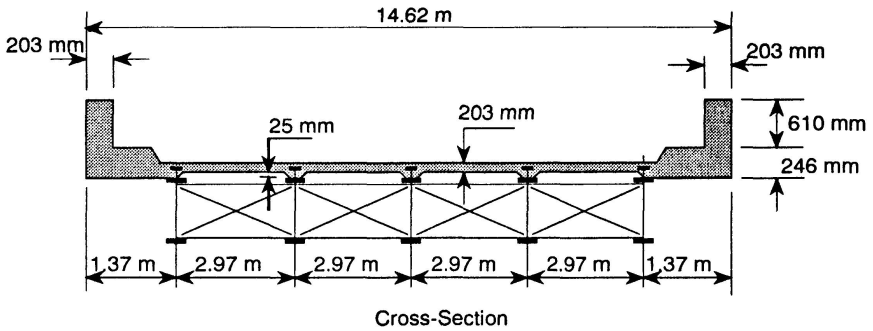

Figure 9/ Cross-section of the North Muskoka River bridge

The cross-section of the single-span North Muskoka River bridge is shown in Fig. 9. This bridge comprises five steel girders and a composite deck slab; its span and width are 45.7 m and 14.6 m, respectively. Both ends of every girder rest on laminated elastomeric bearings each measuring 560 x 335 mm in plan and 64 mm in thickness. The design shear rate for each bearing is about 30 kN/mm.

A dynamic test showed the North Muskoka river bridge to be about 20% stiffer flexurally than could be rationalized by even a very detailed analysis in which all those components of the bridge were taken into account which could conceiva-bly enhance the flexural rigidity of the bridge. To determine the cause for the apparent discrepancy, a diagnostic static test was conducted subsequently. For this latter test, all the girders were instrumented with strain measuring devices to measure longitudinal strains at three transverse sections of the bridge, one section being near the mid-span and each of the other two near each abutment [8].

If the elastomeric bearings had permitted a free longitudinal movement of the girders, then under live loads the strains in the bottom flanges near the bearings would have been tensile and very small. It was found that this was not the case. The test loads induced fairly large compressive strains in the bottom flanges near the elastomeric bearings. Bearing restraint forces computed approximately from observed strains are plotted in Fig. 10 for different load cases. It is interesting to note that under transversely symmetrical loads, the corresponding bearing restraint forces were not exactly the mirror image of each other, as should have been the case for an ideally symmetrical structure. Bearing restraint forces as high as 175 kN, which can be seen in Fig. 10, are considerably larger than a functioning elastomeric bearing would be expected to develop. Nevertheless, such large forces were really present despite the fact that the bearings were apparently in excellent and functioning condition.

A further proof of the presence of large bearing restraint forces in the North Muskoka river bridge was provided by comparisons of applied moments obtained from considerations of simple supports with those computed from girder strains. Figure 11 shows the comparison of mid-span girder moments computed from measured strains with those obtained by the familiar grillage analogy method. The bearing restraint forces were not accounted for in this analysis. It can be readily concluded from this figure that the total moment sustained by all the girders is noticeably less than the corresponding applied moment obtained on the basis of simple supports; this confirms that the applied moments were reduced by the effect of bearing restraint.

It was found that at the time of the test the bearing restraint in the North Muskoka river bridge reduced the mid-span deflections due to test loads by about 12%. This reduction is considerably smaller than the 20% reduction observed in the previous test on the same bridge. The first test was conducted on a relatively cool day in October and the second on a very hot day in June. It is hypothesized that the elastomeric bearings had become stiffer in the cold temperature when the first test was conducted thereby generating higher restraint forces which consequently caused the bridge to become effectively stiffer than it was at the time of the second test.

Results of tests on the North Muskoka river bridge demonstrate the significant influence of the restraining effects of elastomeric bearing which may change with load level and temperature. To be able to analyze bridges with these bearings more accurately, it is essential to include their effective shear stiffness in the mathematical model.

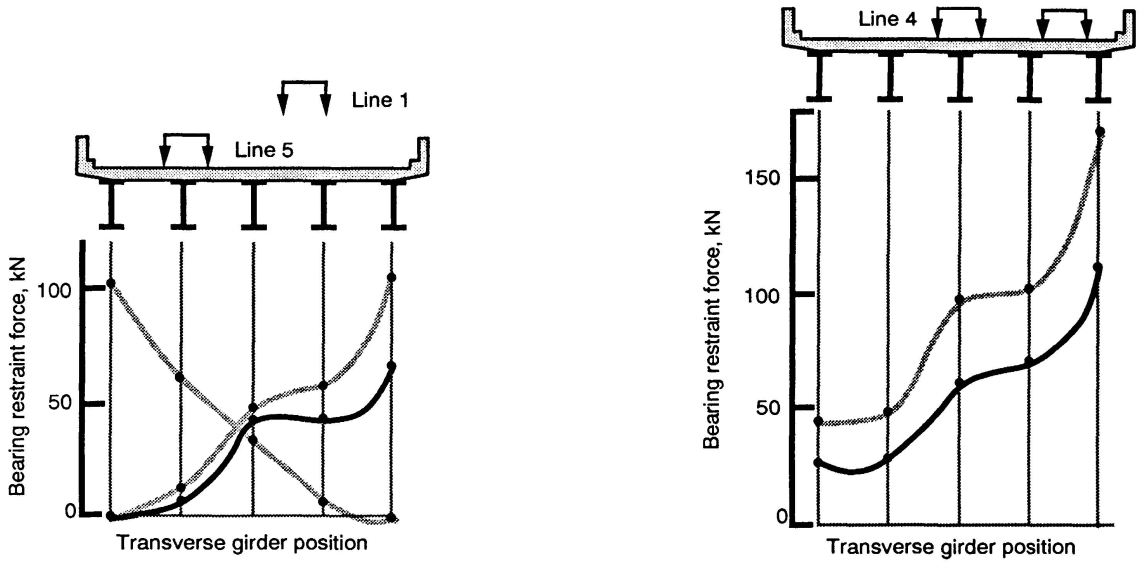

Figure 10/ Bearing restraint forces in the North Muskoka River bridge

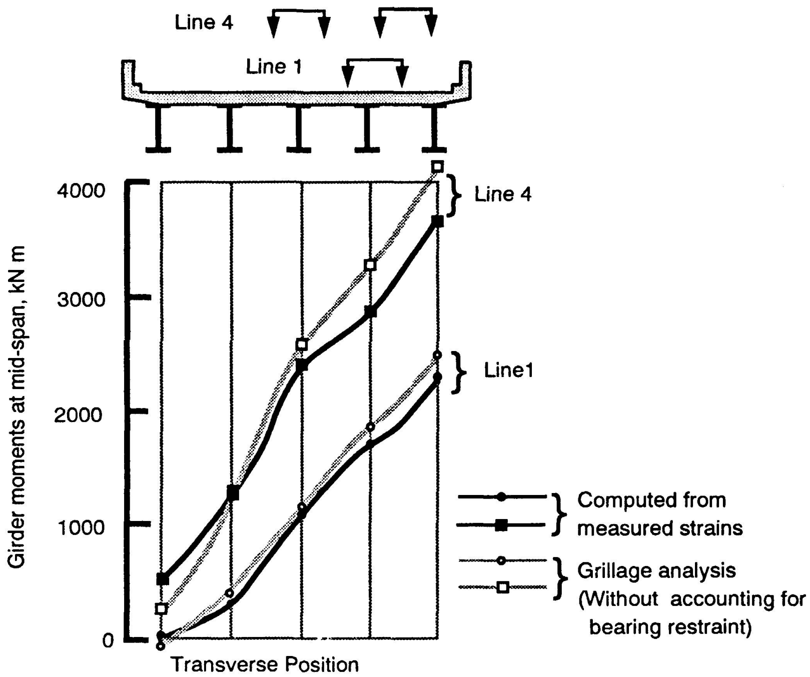

Figure 11/ Mid-span girder moments in the North Muskoka River bridge

7. CONCLUSIONS

The purpose of this paper is not to discredit the rigorous methods of analysis, but to note that there are certain unquantifiable behavioral aspects in bridges which cannot be accounted for realistically in a mathematical model. Because of this difficulty, the predictions of even highly rigorous and very accurate methods of analysis may not reflect reality. This contention has been illustrated with the help of results obtained from tests on five short or medium span bridges with steel girders. It is suggested that in some cases, a realistic evaluation of the load carrying capacity of an existing bridge can be conducted only through a field test.

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SELECTED PAPERS

Reliability-Based Evaluation of Existing Bridges
Appréciation de la fiabilité des ponts existants
Zuverlässigkeitsuntersuchung bestehender Brücken

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Andrzej S. Nowak, born 1945, got his MS and Ph.D. degrees at the Warsaw Technical University, Poland. Since 1979 he is a professor of Civil Engineering at the University of Michigan. His research areas include structural reliability, LRFD bridge design and evaluation criteria and modeling human errors.

SUMMARY
Reliability is considered as a rational measure of the structural performance. The major parameters which require evaluation are random variables, in particular, load components, load distribution factors and resistance. Static and dynamic portions of live load are simulated using the available truck survey data. Resistance is modeled by simulations using the available material data. Reliability indices are calculated for girder bridges. The analysis is performed for steel girders, reinforced concrete T-beams and prestressed concrete girders. Various spans and girder spacings are considered.

RÉSUMÉ
La fiabilité est considérée comme une grandeur rationelle de la qualité structurale. Les paramètres principaux à évaluer comprennent les variables aléatoires, tout particulièrement la nature et la répartition des facteurs de charge, ainsi que la résistance de la structure. Les effets partiels statiques et dynamiques des charges mobiles sont simulés à partir de relevés des données disponibles relatives aux camions, tandis que la résistance est étudiée à partir de ceux des données disponibles relatives aux matériaux. L'auteur en déduit ensuite des indices de fiabilité pour les ponts à poutres, à savoir pour ceux à poutres en acier, en béton armé et en béton précontraint. Dans cette étude, il prend en considération la variation des portées et des entraxes de poutres.

ZUSAMMENFASSUNG
Die Zuverlässigkeit wird als rationales Mass für die Tragwerksgüte angesehen. Ihre auszuwertenden Hauptparameter sind Zufallsvariablen, insbesondere Arten und Verteilung der Einwirkungen sowie der Tragswerkswiderstand. Statische und dynamische Anteile der Verkehrseinwirkungen werden aufgrund verfügbarer Erhebungen von Lastwagendaten simuliert, der Widerstand aufgrund vorhandener Werkstoffdaten. Daraus werden Zuverlässigkeitsindizes für Trägerbrücken ermittelt, und zwar für solche aus Stahl, Stahlbeton und Spannbeton. Unterschiedliche Spannweiten und Trägerabstände werden betrachtet.

1. INTRODUCTION

Bridge evaluation is an increasingly important topic in the effort to deal with the deteriorating infrastructure. There is a need for accurate and inexpensive methods to determine the actual strength of the bridge and the actual load spectrum. The major factors that have contributed to the present situation are: the age, inadequate maintenance, increasing load spectra and environmental contamination [3,7]. The deficient bridges are posted, repaired or replaced. The disposition of bridges involves clear economical and safety implications. To avoid high costs of replacement or repair, the evaluation must accurately reveal the present load carrying capacity of the structure and predict loads.

The major parameters which affect the structural performance are loads and resistance (load carrying capacity). Bridge loads include dead load (own weight of the structural and non structural components), live load (weight of trucks), dynamic load (dynamic effects of moving trucks), environmental loads (wind, earthquake, temperature) and special forces (collisions, emergency braking). Resistance is determined by material properties, dimensions and geometry, and it depends on the method of analysis. Loads and resistance are random in nature. Their variation and uncertainty involved in the analysis can be expressed by statistical parameters. Knowledge of loads, their magnitude and frequency of occurrence, can be gained through surveys, field observations, measurements and statistical analysis. In this study, bridge load and resistance models are reviewed and a procedure is formulated for evaluation existing bridges. Structural performance is measured in terms of the reliability index.

2. BRIDGE LOAD MODEL

The basic load combination for highway bridges is a simultaneous occurrence of dead load, live load and dynamic load. The load models are developed using the available statistical data, surveys and other observations. Load components are treated as random variables. Their variation is described by two parameters: λ = ratio of mean-to-nominal and V = coefficient of variation. Existing bridges are evaluated for various periods of time, e.g. 1, 5 or 75 years. Therefore, the extreme values of loads are extrapolated from the available data base. Nominal values of load components are calculated according to the current AASHTO [1].

Dead load, D, is the gravity load due to the self weight of the structural and non structural elements permanently connected to the bridge. Because of different degrees of variation, it is convenient to consider four components of D, as shown in Table 1.

Table 2-1. Statistical Parameters of Dead Load

Component	λ	V
Factory-made members	1.03	0.08
Cast-in-place members	1.05	0.10
Asphalt (mean thickness)	90mm*	0.25
Miscellaneous	1.03-1.05	0.08-0.10

The need for a reliable truck weight data has been recognized by many bridge authorities. In this study, the load spectra are determined on the basis of truck survey data, truck counts and weigh-in-motion measurements. The data includes truck weights, axle spacings and axle loads. Multiple truck occurrence (more than one truck on the bridge simultaneously) is determined by special truck counts. Bridge performance is affected by moments and shears rather than gross vehicle weights. Therefore, the surveyed trucks were run over the influence lines to determine the moments and shears.

The development of live load model for highway bridges is described by Nowak and Hong [6] and Nowak [5]. The expected maximum live load moments and shears are evaluated for various time periods. Life time is 75 years for newly designed bridges, but 1 to 5 years for evaluation of existing structures. The measured trucks represent a statistical sample of the total number of trucks which cross a bridge in 1, 5 or 75 years. Therefore, calculation of the maximum moment (shear) for longer periods involves extrapolation of the obtained results.

The maximum effect is calculated by simulation of the actual traffic. For multiple occurrence, various truck configurations are considered: in lane and side-by-side. The analysis indicates that a lane load is governed by a single truck up to about 30-36m span. For longer spans two fully correlated trucks govern. For two lanes, the live load is governed by two fully correlated trucks (side-by-side), each being about 85% of the mean maximum 75 year truck. The actual values of the mean maximum moments and shears for various time periods also depend on traffic volume.

The statistical parameters of live load moment for various spans and for time periods 1, 5 and 75 years are presented in Table 2. The nominal value is calculated as a design moment specified by AASHTO [1], as shown in Fig. 1.

Table 2 Statistical Parameters of Live Load Moment

	Time Period					
	1 year		5 years		75 years	
Span (m)	λ	V	λ	V	λ	V
3	1.37	0.15	1.46	0.15	1.65	0.14
12	1.58	0.13	1.64	0.12	1.74	0.11
36	1.90	0.135	1.97	0.12	2.08	0.11
60	1.78	0.14	1.85	0.125	1.96	0.11

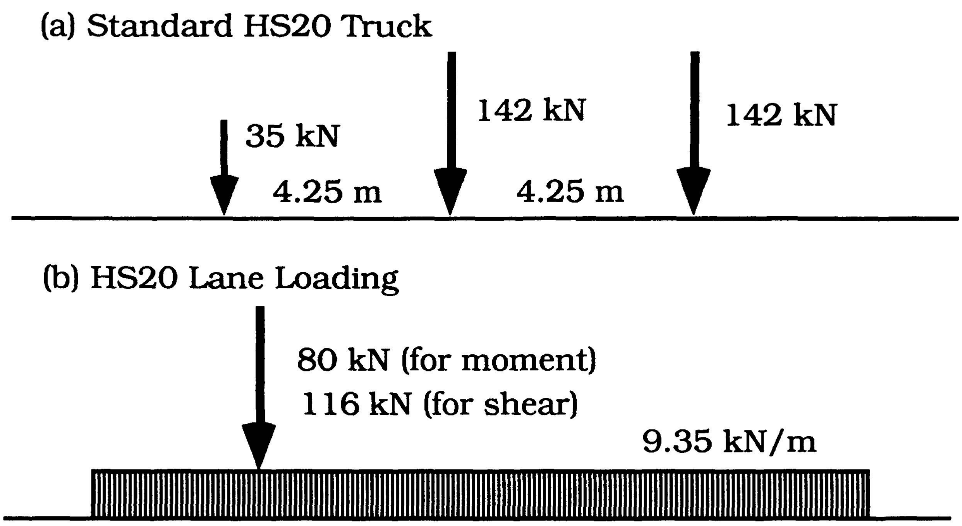

Fig. 1 AASHTO Standard Truck and Lane Loading [1].

Traditionally, the dynamic load is considered as an equivalent static load. Many codes, including AASHTO [1], specify the dynamic load as the function of span length only. However, it has been observed that the dynamic load depends on dynamic properties of the vehicle, dynamic properties of the bridge, and pavement roughness. A procedure was developed by Hwang and Nowak [2] to model the dynamic behavior of girder bridges including the three factors (pavement roughness, bridge and vehicle). The procedure was used for extensive simulations. The major parameters considered include the degree of road roughness, truck type and weight, speed, and structural type of bridge. The results indicate that the absolute value of the dynamic load component is almost constant (measured in terms of deflection). Static deflection increases with truck weight and therefore the dynamic load, as a percent of static live load, decreases for heavier trucks.

Simulations were carried out for the cases of one truck and two trucks side-by-side. The results indicate that, on average, dynamic loads do not exceed 15% of live load for a single truck and 10% for two trucks. The coefficient of variation of dynamic load is about 0.80.

3. RESISTANCE MODELS

The capacity of a bridge depends on the resistance of its components and connections. The component resistance, R, is determined mostly by material strength and dimensions. R is a random variable. The causes of uncertainty can be put into three categories:

- material; strength of material, modulus of elasticity, cracking stress, and chemical composition.
- fabrication; geometry, dimensions, and section modulus.
- analysis; approximate method of analysis, idealized stress and strain distribution model.

The resulting variation of resistance has been modeled by tests, observations of existing structures and by engineering judgment. The information is available for the basic structural materials and components. However, bridge members are often made of several materials (composite members) which require special methods of analysis. Verification of the analytical model may be very expensive because of the large size of bridge members. Therefore, the resistance models are developed using the available material test data and by numerical simulations.

In this study, R is considered as a product of the nominal resistance, R_n and three parameters: strength of material, M, fabrication (dimensions) factor, F, and analysis (professional) factor, P,

$$R = R_n \, M \, F \, P \tag{1}$$

The mean value of R, m_R, is

$$m_R = R_n \, m_M \, m_F \, m_P \tag{2}$$

and the coefficient of variation, V_R, is,

$$V_R = (V_M^2 + V_F^2 + V_P^2)^{1/2} \tag{3}$$

where, m_M, m_F, and m_P are the means of M, F, and P, and V_M, V_F, and V_P are the coefficients of variation of M, F, and P, respectively.

The statistical parameters are developed for steel girders, reinforced concrete T-beams, and prestressed concrete AASHTO-type girders. The results are presented in Table 3.

Table 3 Statistical Parameters of Resistance

Type of Structure	F M		P		R	
	λ	V	λ	V	λ	V
Non-composite steel girders						
Moment	1.095	0.075	1.02	0.06	1.12	0.10
Shear	1.12	0.08	1.02	0.07	1.14	0.105
Composite steel girders						
Moment	1.07	0.08	1.05	0.06	1.12	0.10
Shear	1.12	0.08	1.02	0.07	1.14	0.105
Reinforced concrete						
Moment	1.12	0.12	1.02	0.06	1.14	0.13
Shear	1.13	0.12	1.075	0.10	1.20	0.155
Prestressed concrete						
Moment	1.04	0.045	1.01	0.06	1.05	0.075
Shear	1.07	0.10	1.075	0.10	1.15	0.14

4. RELIABILITY ANALYSIS

The available reliability methods are presented in several publications e.g. [4, 8]. In this study the reliability analysis is performed using Rackwitz and Fiessler procedure. The reliability index, β, is defined as a function of P_F,

$$\beta = -\ \Phi^{-1}(P_F) \tag{4}$$

where Φ^{-1} = inverse standard normal distribution function.

There are various procedures available for calculation of β. These procedures vary with regard to accuracy, required input data and computing costs. The simplest case involves a linear limit state function. If both R and Q are independent (in the statistical sense), normal random variables, then the reliability index is,

$$\beta = (m_R - m_Q)/(\sigma_R^2 + \sigma_Q^2)^{1/2} \tag{5}$$

where m_R = mean of R, m_Q = mean of Q, σ_R = standard deviation of R and σ_Q = standard deviation of Q.

If both R and Q are lognormal random variables, then b can be approximated by

$$\beta = \ln\,(m_R/m_Q)/(V_R^2 + V_Q^2)^{1/2} \tag{6}$$

where V_R = coefficient of variation of R and V_Q = coefficient of variation of Q. A different formula is needed for larger coefficients of variation.

If the parameters R an Q are not both normal or lognormal, then the formulas give only an approximate value of β. In such a case, the reliability index can be calculated using Rackwitz and Fiessler procedure, sampling techniques or by Monte Carlo simulations. Rackwitz and Fiessler [4, 8] developed an iterative procedure based on normal approximations to non-normal distributions at the so called design point. The design point is the point of maximum probability on the failure boundary (limit state function). The procedure has been programmed and calculations are carried out by the computer.

5. RELIABILITY INDICES FOR SELECTED BRIDGES

The calculations are performed for a selected set of girder bridges. The selection was based on material, span, number of girders and girder spacing. For the selected bridges, moments and shears are calculated due to dead load components, live load and dynamic load. Nominal (design) values are calculated using AASHTO [1]. The mean maximum values of live load are obtained using the statistical parameters given in Table 1 and 2. Resistance is calculated in terms of the moment carrying capacity. For each case, two values of the nominal resistance are considered: R_{actual}, the actual as-built load carrying capacity and R_{min}, the minimum required resistance which satisfies the AASHTO [1]. In general, R_{actual} is larger than R_{min}. The basic design requirement is expressed in terms of moments as follows [1],

$$1.3\ D + 2.17\ (L + I)\ <\ \phi R \tag{7}$$

where D, L and I are moments due to dead load, live load and impact, R is the moment carrying capacity, and ϕ is the resistance factor, ϕ = 1.00 for steel and prestressed concrete girders, and 0.90 for reinforced concrete T-beams. The ratio of R_{actual} / R_{min} is an indication of overdesign and it is about 1.5 for steel girders and about 1.1 for prestressed concrete girders.

The selected bridges do not cover a full range of spans and other parameters. Therefore, additional bridges are designed as a part of this study. The analysis is focused on girder bridges with spans from 9 to 60 m. Five girder spacings are considered: 1.2, 1.8, 2.4, 3.0 and 3.6 m. Typical cross sections are assumed. In all considered cases, the actual resistance, R_{actual}, is made equal exactly to R_{min} (the sections are neither overdesigned nor underdesigned).

The reliability indices are calculated for girder bridges described by the representative load components and resistance. The results are presented in Fig. 2-4 for steel girders, reinforced concrete T-beams and prestressed concrete girders.

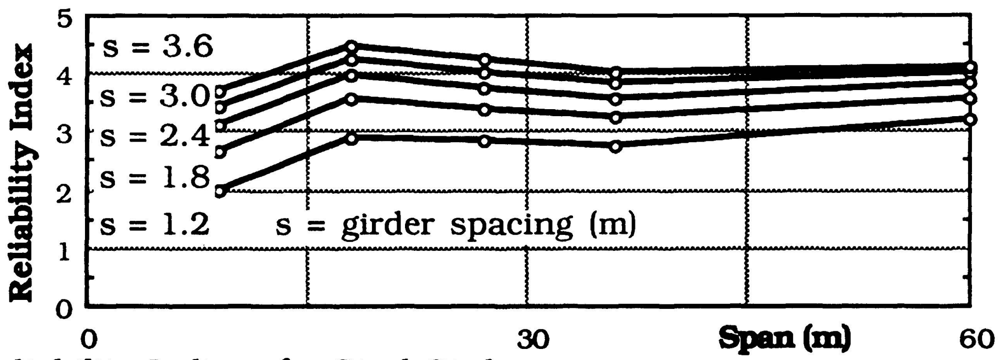

Fig. 2 Reliability Indices for Steel Girders.

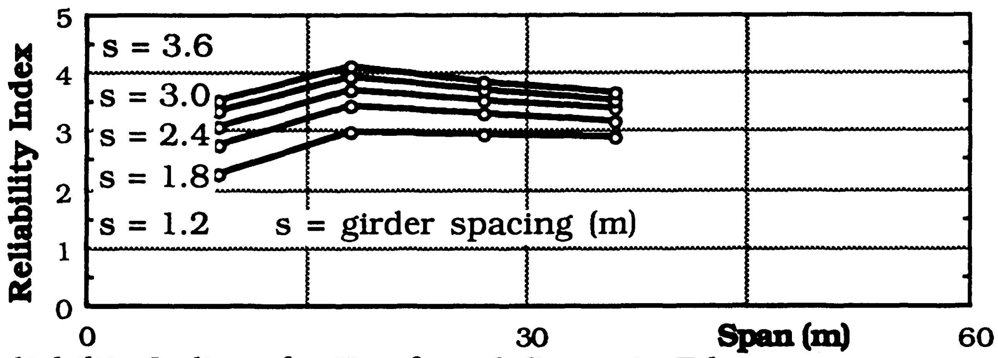

Fig. 3 Reliability Indices for Reinforced Concrete T-beams.

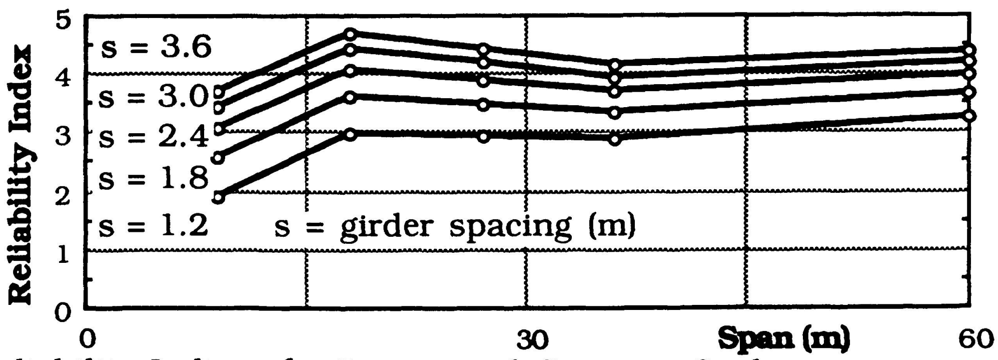

Fig. 4 Reliability Indices for Prestressed Concrete Girders.

6. CONCLUSIONS

The reliability index can be used as an objective measure of structural performance of an existing bridge. The calculation requires the knowledge of site-specific load and resistance parameters. Load parameters can be determined by truck surveys. Resistance depends on the degree of deterioration (e.g. corrosion). The statistical parameters for a general case are presented in Tables 1-3.

Reliability indices are calculated for typical girder bridges, designed using AASHTO [1]. The results show a considerable degree of variation depending on girder spacing, material and span length. This is a clear indication that the current design provides a higher safety reserve for larger girder spacings and spans about 20 m.

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Load Testing and Numerical Modelling of Quebec Bridges
Essais de charge et modéles numériques de ponts au Québec
Belastungsversuche und ihre numerische Modellierung bei Brücken Quebecs

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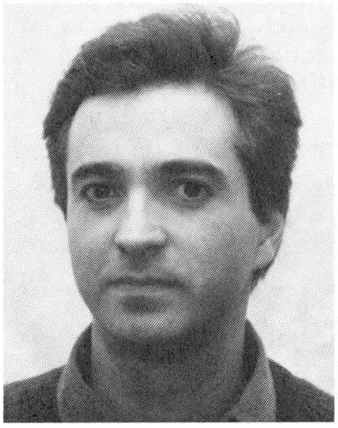

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SUMMARY
This paper presents some of the results concerning the structural evaluation of the Quebec bridges. The approach consists here of two steps. The first one is an experimental step based on the extensive use of a mobile laboratory for static and dynamic measures. The second step is based on the construction and calibration of a three dimensional finite element model. The results which are also compared to standard simplified evaluations show some of the advantages of this kind of more sophisticated evaluations.

RÉSUMÉ
Cet article présente les résultats de l'évaluation de certaines structures de ponts du Québec. L'approche comporte d'abord un volet expérimental effectué à l'aide d'un laboratoire mobile pour les mesures statiques et dynamiques. Puis, un modèle numérique complet en trois dimensions est construit puis calibré sur les résultats expérimentaux. Les résultats obtenus démontrent certains avantages de ce type d'évaluations poussées par rapport aux évaluations standards simplifiées.

ZUSAMMENFASSUNG
Der vorliegende Artikel beschreibt die Bewertungsergebnisse bestimmter Brückentragwerke von Quebec. Dabei handelt es sich zunächst um ein Experiment, das mit Hilfe eines mobilen Labors im Hinblick auf die Erfassung von Statik und Dynamik durchgeführt wurde. Danach wurde ein komplettes dreidimensionales numerisches Modell erstellt, das auf die Ergebnisse des Experiments zugeschnitten ist. Die so erzielten Ergebnisse zeigen deutlich die Vorteile dieser Bewertungsmethode gegenüber einfachen Standardberechnungen.

1. INTRODUCTION

Aging of structures and public budget cuts require more than ever before the exploration of the remaining structural capacity. This is especially true, in many countries, in the field of bridge structures since a major number of these bridges have been build many decades ago. Since then, usual deterioration and fatigue as well as important increase of traffic and loading intensity have been observed. Many aspects concerning the remaining bridge capacity are still unknown. For this reason, the Quebec government has initiated an extensive program of experimental in situ measurements of bridges showing real or potential structural problems.

The paper presents the experimental results as well as the calibration of a related detailed tridimensional finite element model for recent specific bridge cases. The experimental part has been performed here by an efficient mobile laboratory for which strain gauges and accelerometers are available. A special processing of strain data makes it possible to obtain directly the stress resultants (axial force, moments) instead of local stresses. These values are then directly comparable to a simple or more complex finite element model to be calibrated. This calibrated model is then used as the most realistic model for the more complex or more intensive loading cases to be checked according to actual design codes [13]. Both static effects as well as dynamic behavior can be taken into account. interesting results for a first bridge have already shown the scientific as well as the economic benefit of such an approach [1]. New results for two other deck slab steel truss bridges are now available and will be presented in this paper.

The first objective of the paper is to expose the methodology of the experimental approach as well as the calibration of the finite element model for the two bridges. It also shows how much valuable information can be readily available in such a combined approach. Finally, the paper also shows how standard evaluations of existing bridges may sometimes be inadequate in predicting the exact remaining capacity. In fact, in situ measurements and refined modelling can yield in a cost reduction of expected repairs. It also improves the engineer confidence in the making of repair decisions.

2. STANDARD BRIDGE EVALUATION

The standard evaluation practice of old bridges is usually based on methods which are similar to those used for new bridges. Standard linear two dimension displacement models (rigidity method) are normally selected. A distributed factor is used to consider the transverse non symmetrical distribution of the live load. The evaluation is based mainly on the results of an inspection of the bridge which can show structural problems such as rust, cracks and fatigue in members. The assumptions of this analysis are mainly the same as in the original design. One of the main differences lies however in the intensity of the live loads to apply. In fact, truck weights have severely increased for the past decades. That is why, a few years ago, a standard truck specifically developed for the Quebec traffic, the QS660 truck, has been adopted for design or analysis of new and old bridges. This truck has a total weight of 660 kN with a 60 kN, 240 kN, 200 kN, 160 kN distribution from front to end of the truck and with related distances of 4 m, 6 m and 6m respectively and a width of 1.8 m. For instance, for old bridges, like the two bridges presented in this paper, the design live loads were two to three times less than those of the QS660 truck.

For this reason, these live loads are now often critical for the structure. Moreover, the live loads must naturally be increased to keep into account the dynamic effects. This dynamic allowance factor can be related to many factors as shown in the literature [10]. In the Canadian Bridge Code [13], however, the rate of this factor is only related here to the first flexural frequency. A maximum value of 40% increase will be applied for a frequency lying between 2.5 and 4.5 Hz. Otherwise, this value will be at least 20%. For steel truss bridges, this factor is often at its maximum value.

If the evaluation shows an acceptable resistance to the loadings in the Norm, it is considered as being over. If the bridge shows too much weakness, it might be replaced, repaired or limited to lighter vehicles. Sometimes, however, the results of the analysis lies somewhere in a "grey zone". This means that the analyst has a good presumption that the standard evaluation is probably too conservative due to approximations such as the two dimension modelling or as neglecting the rigidity of some components. Two more check steps are then available as described in the next two sections.

3. EXPERIMENTAL EVALUATION

The Ministry of transportation of Quebec acquired, in 1990, a mobile laboratory specifically dedicated to the testing of bridges. Actually, this mobile laboratory has two independent data acquisition systems. One for static tests that includes 60 channels for strain gauges and 20 for other sensors such as LVDT or full-bridge gauges. The dynamic test system embodies 12 channels for accelerometers and 8 channels for strain gauges. As many as 8 pressure tubes can be used to evaluate the total time of the forced vibrations and the averaged speed of the vehicles over the bridge. Up to 50000 samples per second can be recorded from each instruments. The laboratory also includes a micro computer, a color plotter and two printers. A 5000 watts diesel generator is the main power supply of the vehicle. Test trucks used to load and excite the bridge are standard 10 wheel trucks and weight around 260 kN (250 kN for Armagh and 265 kN for Sainte-Marie). Because of the Canadian weather, all tests are performed from late spring to mid fall. It is impossible to give here all the details of the experimental results so that it has been decided to present specific results for each of the two bridges. The Armagh bridge has been selected to present the static approach while the the Sainte-Marie bridge will show the dynamic results (see the corresponding subsections 6.1 and 6.2).

4. FINITE ELEMENT EVALUATION

Once the in-situ tests have been performed and the experimental values postprocessed, a more refined numerical model is build. A commercial program GIFTS (CASA) has been used here on a IBM PC 386. Models are kept moderately large with about 1000 degrees of freedom. The model is kept linear and truss, beam, spring and quadrilateral shell elements are used. A diagonal mass matrix has been selected for computing the eigenmodes via the subspace iteration algorithm. The basic assumption here is that experimental results make the best information available about the structure in spite of normal experimental inaccuracies. Since a first finite element model never fits exactly experimental values, this one has to be calibrated by an iterative process in order to get closer to the experimental data. Once the model has been locally calibrated (local forces in specific member) or more globally calibrated (via dynamic modal values), this one is believed to be adequate in all its tested and non tested members.

There is no standard recipe or standard steps for a successful calibration. The choice of the components to be calibrated depends on many factors like the kind of structures (steel or concrete, for instance) and the results of the in-situ inspection (actual state of some components). For the kind of bridges which are studied in this paper two main factors were selected. The longitudinal restraints of the supports have been modified to take into account the partial restraint of the theoretically free supports. Spring elements have been tested to model this partial restraint. Some artificial beam elements have also been used to link the deck to the trusses in order to modulate the participation of this deck. This appeared to be the most important step. To a lower extent, we also checked the influence of the Young modulus of concrete. A slight dynamic calibration of the mass of the deck (pavement thickness) has also been tested. Finally, once the model has been calibrated, we assume a linear behavior of the structure.Loadings according to the Canadian Code are then performed with strictly localized QS660 trucks and maxima efforts are extracted. These efforts are considered to be the "real" efforts that would appear in the structure in its actual state.

5. DESCRIPTION OF THE TWO BRIDGES

5.1 The Armagh Bridge

The Armagh bridge is a deck slab steel truss bridge which is located on the small river near Saint-Cajetan d'Armagh (Fig. 1). It was build in 1931 and is 51.2m long and 7.3m large. This average size structure has three spans which are 11.9m, 27.4m, 11.9m long with the central one with steel trusses. The thickness of the concrete slab is 0.178m and is 0.065m for the pavement and no specific composite action is present (in theory only). For this structure, steel and concrete Young modulus are 200000 MPa and 250 Mpa. The total number of elements for the standard 2-dimensional truss model is 33. Practically all structural components have been discretized for the 3-dimensional model. These include trusses, stringers, floor beams, the bracings and the concrete slab (Fig. 3). The total number of elements of all kinds is 415 while the total number of degrees of freedom is 976.

5.2 The Sainte-Marie Bridge

The Sainte-Marie bridge is also a deck slab steel truss bridge which is located on the rather large "Chaudière" river in the Sainte-Marie Town (Fig. 2). It is an important bridge in the area. It was build in 1918 and is 130m long and 6.6m large. This structure has two 65m identical spans. Only one steel truss span has been studied. The thickness of the concrete slab is 0.178m and is 0.065m for the pavement and no specific composite action is present in theory. The total number of elements for the standard 2-dimensional truss model is 54. Practically all components have been discretized for the 3-dimensional model that is trusses, stringers, floor beams, the bracings and the slab (Fig. 4). The total number of elements is 523 while the total number of degrees of freedom is 1246.

6. RESULTS

6.1 The Armagh Bridge

As mentioned previously, the Armagh bridge has been specifically selected here to present the static results. Six steel members have been instrumented with a total of 40 strain gauges. These members are those identified with numbers on Fig. 1. Each member had at least 3 or 4 gauges in order to determine all internal efforts developed in the member (axial force N; bending moments M_x and M_y; warping moment B). Considering that the recorded deformation is the sum of the deformation produced by each effort, we have [15]:

$$\varepsilon_i = \frac{N}{EA} + \frac{M_x}{EI_x} y_i + \frac{M_y}{EI_y} x_i + \frac{B}{EI_\omega} \omega_i \tag{1}$$

where E is the Young modulus, A is the cross-section area, I_x and I_y are the moment of inertia with respect to the x and y axis, x_i, y_i and w_i are the x, y and sectorial coordinates of the strain gauge. Having a similar equation for each strain gauge i of a member, this sets up a system of equations easy to resolve for the four unknown N, M_x, M_y and M_ω.

The tests took place on August 1991, during which 8 load paths from A to H (from one sidewalk to the other) have been followed along the bridge. For each path, 12 positions of the 27 tonne 2 axle truck have been occupied (see Fig 1). The truck positions were selected as a function of the second row of wheels being placed directly over each vertical member of the truss. Since a simultaneous calibration of all tested members at any truck position was not possible in practice, the direction of the calibration was dictated by the values of a very few selected members for selected truck positions. Once these values had a satisfactory convergence towards the experimental results, graphics of the axial force in all members as a function of truck positions were plotted. Two examples of these results are shown in Fig. 5 and 6. The Fig. 5 shows the best results while the Fig. 6 shows the worst one which are actually rather good too. The precision of all other results lie between these two cases. Moreover, some dynamic data have been obtained from a few gauges and one accelerometer. The FFT gave us the first flexural frequency which value was 4.10 Hz. As a final check, we computed this value for the calibrated model and obtained 3.83 Hz which is quite satisfactory.

The calibrated model was then ready for its maximum loading according to the Canadian Code. The maximum forces induced by two QS660 trucks (one at 100% and one at 70%) were computed. A significant reduction of 55% to 170% in the load effects was observed compared to the standard analysis for the horizontal top and bottom chords, reducing by almost the half the number of these members to be replaced. Diagonal members improved slightly but appeared to remain weak and will have to be replaced. The vertical members at both extremities of the steel truss appeared to be more loaded in the refined model (by 34%). This shows an interesting example in which the standard approach is here less conservative and then less secure than its more refined counterpart.

6.2 The Sainte-Marie Bridge

The dynamic tests of the Ste-Marie bridge were performed in April 1992. For these tests, 8 low frequency accelerometers have been used, 3 on both sides of the deck for vertical movements measured and 2 for horizontal movements at the abutment and at mid-span, as shown in Fig. 2. One pressure tube was placed at each end of the span. A total of 10 runs have been made at different speeds along the center of the roadway or on the right lane side. The maximum speed was limited to 40 km/h because of the very steep hill at the entry of the bridge.

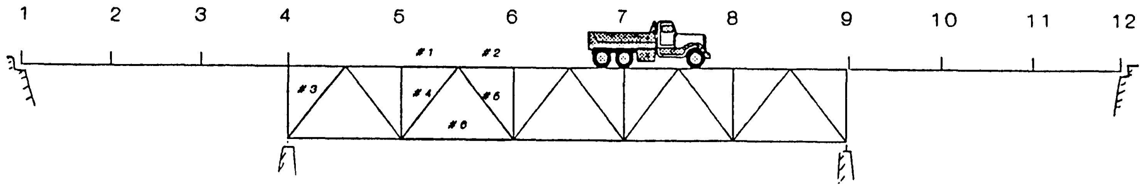

Fig. 1 Armagh Bridge

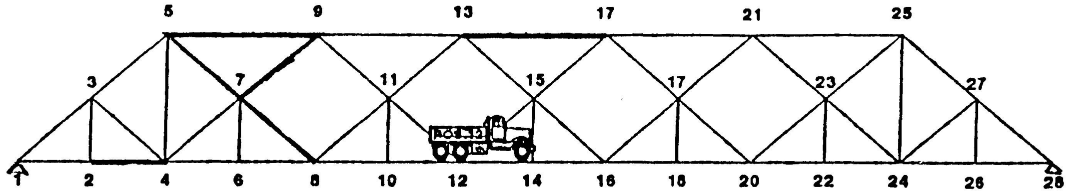

Fig. 2 Sainte-Marie Bridge

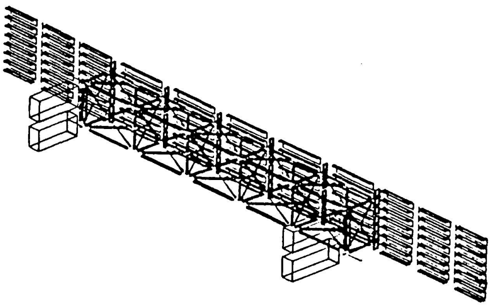

Fig. 3 Numerical Armagh Bridge model

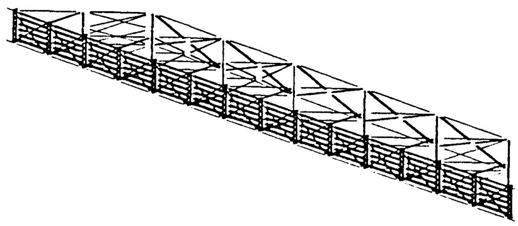

Fig. 4 Numerical Sainte-Marie Bridge model

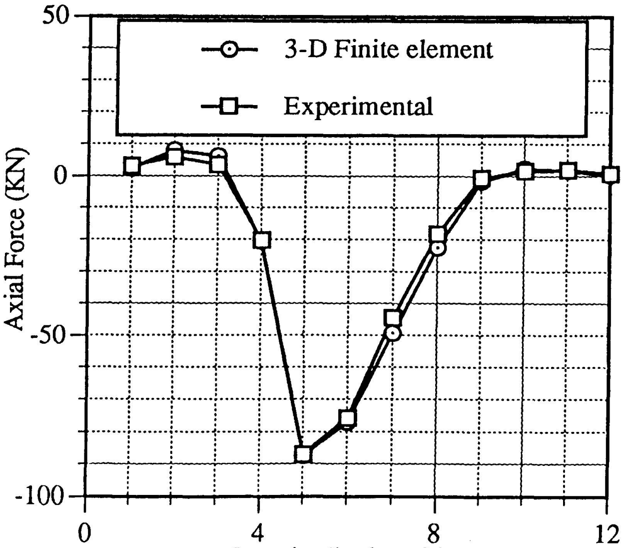

Fig. 5 Axial force in member # 3

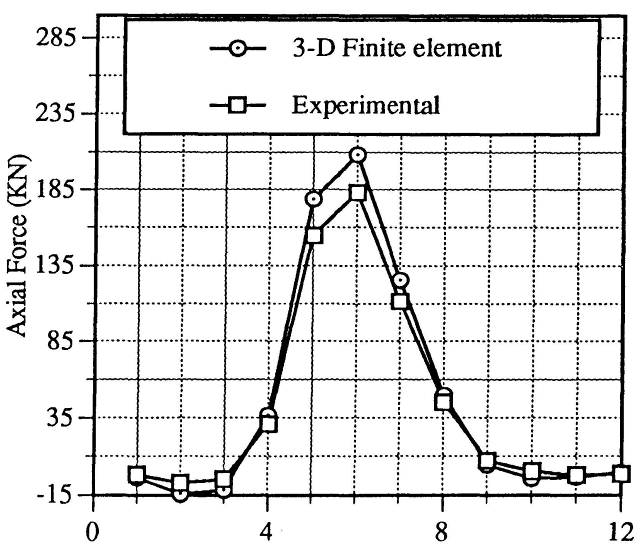

Fig. 6 Axial force in member # 6

In order to evaluate the frequencies of vibration and the corresponding mode shapes included in the response of a bridge under traffic loading, the response of one accelerometer placed away from eventual nodal points is considered as the excitation in the modal analysis. The five other accelerometers measured the outputs. This approach is preferred since the excitation force, which is the variation of the load intensity with time, is not measured.

The computation of the FFT of these 6 signals displays how the energy is distributed in the frequency domain. High peaks usually correspond to the bridge frequencies of vibration. However, during the forced vibration, the frequencies caused by the bridge-vehicle interaction may be included in the spectra [2]. To avoid this potential confusion, the coherence between the excitation signal and the outputs was also computed. The coherence indicates the degree of linearity between the input and the output [10], [6], [7]. Since the amplitude and phase of all points follow a fixed relationship when the bridge vibrates in one of its modes, the coherence should be over 0,9 at mode frequencies of the bridge, and low at other frequencies such as the frequencies of the bridge-vehicle interaction [12]. One may also compute the FFT of each signal for each runs and take the average FFT in the subsequent calculations. This may eliminate noise and some irrelevant peaks. To identify experimental mode shapes, the amplitude of the imaginary part of the transfer function FRF and the phase of the same function give the relative displacements and phases of other accelerometers relative to the reference accelerometer at all frequencies [10]. The procedure described above has been followed for the 8 dynamic runs and the results for accelerometer 2 are presented in Fig. 7. The experimental operating mode shapes are also presented in Fig. 8. One should note a lack of symmetry of three mode shapes, probably due to the short duration of each free vibration records.

The static calibration technique was exactly the same as for the Armagh bridge. Then, a final tuning of model was imposed in order to get closer to the first flexural frequency. A slight modification of the mass of the slab (for pavement) improved the values of the four eigenvalues. These numerical eigenmodes are shown in Fig. 8. A good correlation is observed between the experimental and the numerical values. The model showed however that the experience failed to give some higher flexural modes with lower values than the fourth one but this was due to the small number of accelerometers.

7. CONCLUSIONS

7.1 Summary of the results

In this paper, we presented practical results for two old bridges. The paper showed how experimental results from a mobile laboratory can help to build a well calibrated numerical model. Once calibrated, this model can then give realistic structural values in any part of the structures. The postprocessing of the experimental dynamic data can give the eigenvalues as well as the eigenmodes which can be compared to the numerical model. These values show how standard evaluations are sometimes inadequate. They are often too conservative and, in this case, the combined experimental-numerical approach can save money. They are sometimes (more rarely) not enough conservative and in this case are less secure. Consequently, this combined experimental-numerical approach is certainly going to become a must for the analysis of complex or doubtful aging structures. Final decisions could then be made with a high degree of confidence, allowing engineers to make the proper economic and safety decisions.

7.2 Future considerations

It should be noted that since complete tests for a bridge may request between one to two weeks, the testing program is usually limited to about 10 bridges per season. This kind of program should remain a current practice for the next few years. After this, rapid testing (one or two days per bridge) based only on dynamical excitations might be sufficient for typical or simple bridges. Calibration of the finite element method would then be based only on the dynamic tests. This would avoid the very time consuming strain gauges set up and would partially reduce the weather limitations of these gauges. It would also require the systematic use of the modal synthesis techniques. For the moment, our limited number of accelerometers could still be a problem for more complex structures. For some evaluations based on precise investigations of stress components such as the axial force of certain members, strain gauges will remain an essential tool.

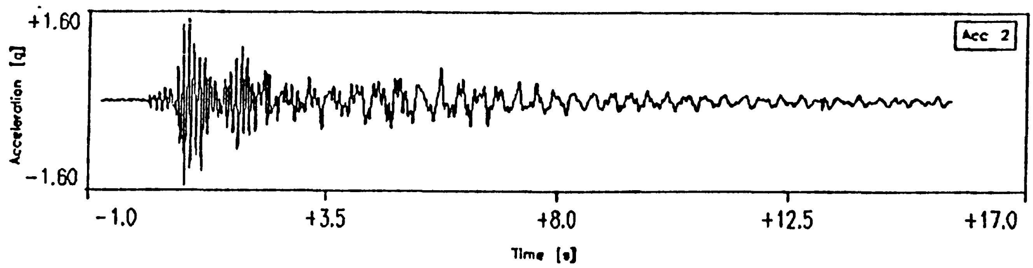

Fig. 7 Accelerations of accelerometer 2

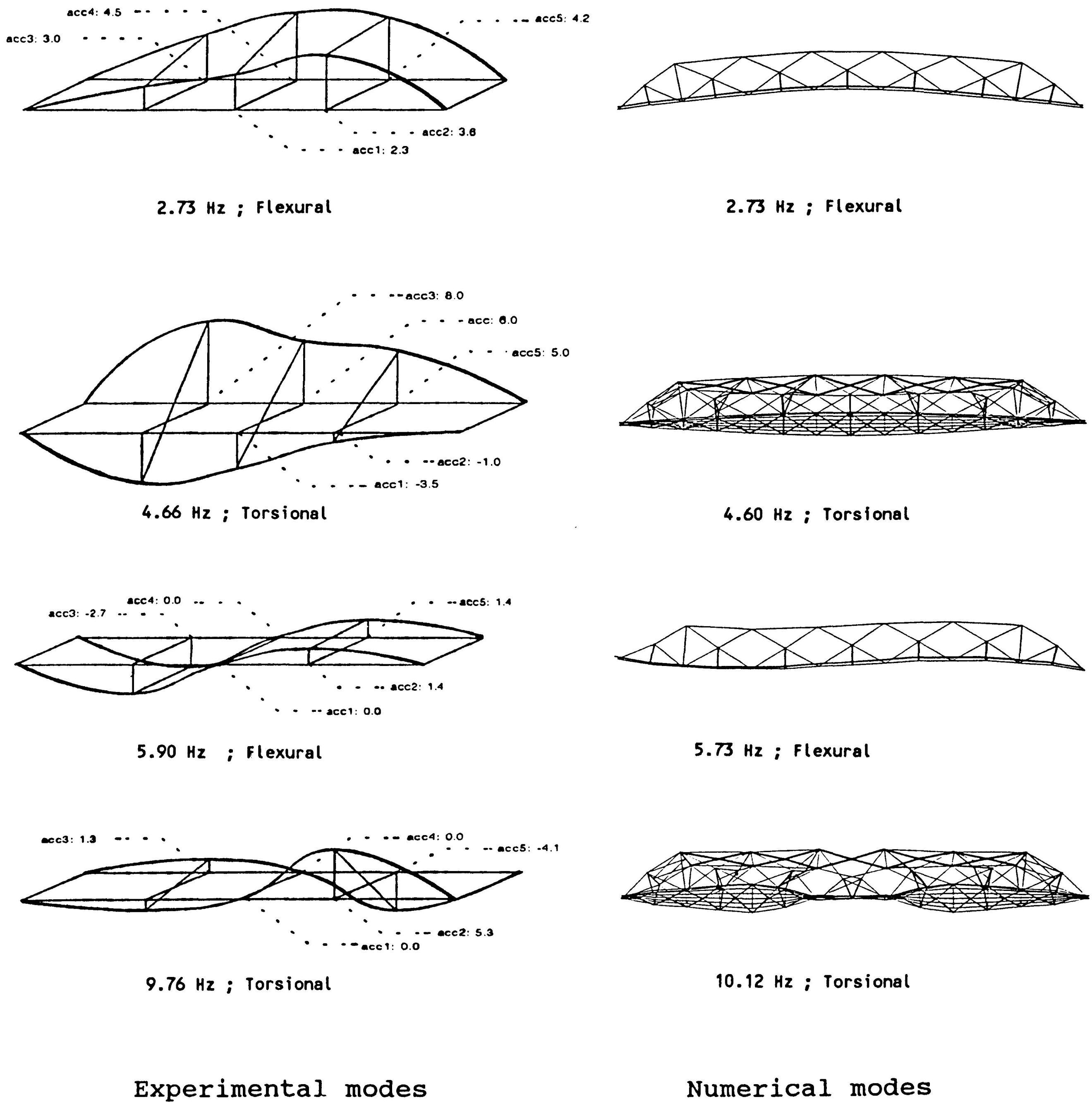

Fig. 8 Experimental and numerical vibration modes

On a more general basis, many aspects concerning old bridge structures are still to be clarified, however. This is why an important scientific collaboration has been settled in order to improve our knowledge of some complex experimental aspects of different kinds of old bridge structures as well as some specific bridge components. This has already led to many interesting results such as the experimental determination of the dynamic amplification factor [6],[7],[10], the ultimate capacity of noncomposite concrete slab on steel girder bridge [8] or the in-situ study of a prestressed bridge [11]. This collaboration has also led, much in the same way, to many improvements and developments of the "domestic" finite element tools to be used soon for bridge modelling. Fundamental algorithmic aspects for dynamic analysis [3] and new thick/thin shell elements for concrete structures [5],[14] have been developed as well as non linear analysis of noncomposite effects in slab [9]. A finite element program written in C (called CLE) is under development for direct bridge-structure interactions [2]. All these developments will be necessary to yield in a better understanding of the behavior of the increasing number of old bridge structures.

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Redundancy and the Reserve and Residual Strength of Frames
Redondance, réserve de résistance et capacité restante de cadres
Redundanz, Reserve- und Restfestigkeit von Rahmen

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Since her academic training at Imperial College, Helen Bolt has been involved with development of the reserve and residual strength technology for offshore jacket structures. She was the engineer on site for the experimental work described in this paper. As Supervising Engineer she now takes overall responsibility for R&D activity.

Keith Ward's expertise is in the analysis of complex problems in offshore engineering. He led the development of the nonlinear programme described in this paper, which is now used to determine the reserve and residual strength of jacket structures. As Associate he leads the company's analysis and software development activities.

SUMMARY
For undamaged structures the conservatism in conventional design practices ensures that a substantial reserve strength exists beyond the design event. Similarly, the redundancy of the structure gives a residual strength enabling it to sustain load even in a damaged condition. Reserve and residual strength, redundancy and collapse mechanisms are important considerations in the design and reassessment of offshore jacket structures. Large scale collapse tests of frames were undertaken and these are described in the paper with a discussion of the findings and analysis predictions.

RÉSUMÉ
Pour les structures intactes, la pratique prudente des méthodes d'étude traditionnelles assure la présence d'une réserve de résistance au-delà de la résistance nominale. De même, la redondance de la structure offre une capacité restante qui lui permet de régister à une charge même en cas de dommages. La réserve de résistance et capacité restante, la redondance et les mécanismes de rupture sont des considérations importantes dans l'étude et la ré-évaluation des structures des chemises des plate-formes en mer. Des essais de rupture de cadres ont été effectués sur une grande échelle; ils sont décrits dans l'exposé, avec une discussion des conclusions et des recommandations.

ZUSAMMENFASSUNG
Bei unbeschädigten Bauwerken wird durch den Konservatismus der konventionalen Entwurfspraktiken eine bedeutende Reservefestigkeit über den Auslegungsfall hinaus gewährleistet. Auf ähnliche Weise wird durch die Redundanz der Konstruktion eine Restfestigkeit erreicht, wodurch gewährleistet wird, dass sie auch in einem beschädigten Zustand die Last tragen kann. Reserve- und Restfestigkeit, Redundanz und Einsturz-mechanismen sind für die Entwurfsgestaltung und Neubewertung von Offshoreplattformen von grosser Bedeutung. An Rahmen wurden Kollapsgrossversuche ausgeführt, die in dem Schriftstück mit einer Diskussion der Ergebnisse und Analysevoraussagen beschrieben werden.

1. INTRODUCTION

Traditionally offshore steel jackets are designed on a component by component basis. Each member and joint is checked against the strengths given in design codes, which themselves have been established from the database of isolated joint and tubular beam-column test results.

In practice a structure's ability to resist loads in excess of the design load (the 'reserve strength') depends not only on the conservatism in the design of individual members and joints, but also on the performance of these components within the frame. Thus the ratios of the collapse load to the design load derived from the critical component, may differ even between structures which have been designed to the same code. Influences of the overall system response on the ultimate capacity of frames, are not generally accounted for in the design of offshore jacket structures since no specific guidance is given within the codes. However, these are being recognised increasingly as important factors in the selection of new platform configurations and in the reassessment of existing installations.

Beyond the ultimate state, be it brought about by accidental damage or overloading, there is an additional requirement for the structure to remain intact, redistributing the loads safely without catastrophic collapse. The ability of a structure to sustain damage in this way, its 'residual strength', is quantified by the ratio of the collapse loads for the damaged and intact structures and depends largely on the structural redundancy within the system. Again, in traditional engineering practice, residual capacity is not considered explicitly in design. However, experiences offshore of changing operational requirements and instances of damage, have emphasised the importance of building redundancy into jacket structures.

If reserve and residual strengths either cannot be quantified justifiably or are considered to be inadequate, the consequences can be costly in terms of repairs or strengthening measures which may in fact be unnecessary or may be carried out at greater risk in inclement weather conditions. With a better understanding of the issues and more prudent designs in the future, it may be that fewer offshore modifications will be required. More detailed description of the sources of reserve and residual strength and early experimental investigations are given in the companion paper presented by the authors at the Inspection, Repair and Maintenance Conference in Aberdeen in 1988 [1].

The ability to understand and quantify the influences on the reserve and residual strength of frames, their redundancy and collapse mechanisms is now an increasing demand of the offshore industry. This is required both for the reassessment of existing structures and for the selection of configurations for new structures. In recognition of both the technical and economic benefits to be gained for the offshore industry, the Joint Industry Tubular Frames Project was established in 1987 to address some of the many questions then arising and to develop a calibrated technique for reserve and residual strength calculations. Sponsored by nine offshore Operators and the UK Department of Energy, the Project marks a significant advance in the application of the reserve and residual strength technology to offshore jacket structures.

Through experiments the project has given unparalleled examples of the ultimate response of frames related to both member and joint failures and has illustrated the important role of redundancy. The frames were the largest ever to have been pushed to collapse in a controlled manner. Coupled with the development of a new nonlinear program, SAFJAC, the Project gave the Participating Organisations a significant advancement in their ability to understand and predict the behaviour of both planned and existing installations.

2. THE JOINT INDUSTRY TUBULAR FRAMES PROJECT

Although early studies had emphasised the role of reserve and residual strength in determining the ultimate capacity of frames, they did not supply sufficient information or a calibrated numerical tool to assess offshore jacket structures of current concern in the North Sea. The Tubular Frames Project was therefore established [2]. The first phase of this joint industry project commenced in November 1987 and was completed in January 1990. The confidentiality period for the Phase I work expires in 1993 and the objectives, scope and findings from the work are described in these sections. In the following section Phase II, which commenced in June 1990, is introduced.

2.1 Objectives of the Frames Project

The overall objectives of the project may be summarised as follows:

- to establish the effects of non-linear joint/member behaviour on frame behaviour and collapse mechanisms.
- to quantify the reserve and residual strength of frames (global safety margins) and to investigate redundancy and load shedding characteristics.
- to investigate the collapse performance of members and joints within frames and to develop procedures for the exploitation of available component data.
- to investigate residual strength and load shedding behaviour of a frame which includes a 'cracked' joint (Phase I).
- to develop a non-linear numerical procedure for the collapse analysis of frames.

2.2 Experimental Scope - Phase I

Four tubular frame tests were conducted to investigate the influence of different modes of failure on reserve and residual strength in Phase I. The general arrangement of the frames and the test set-up are shown in Figure 1.

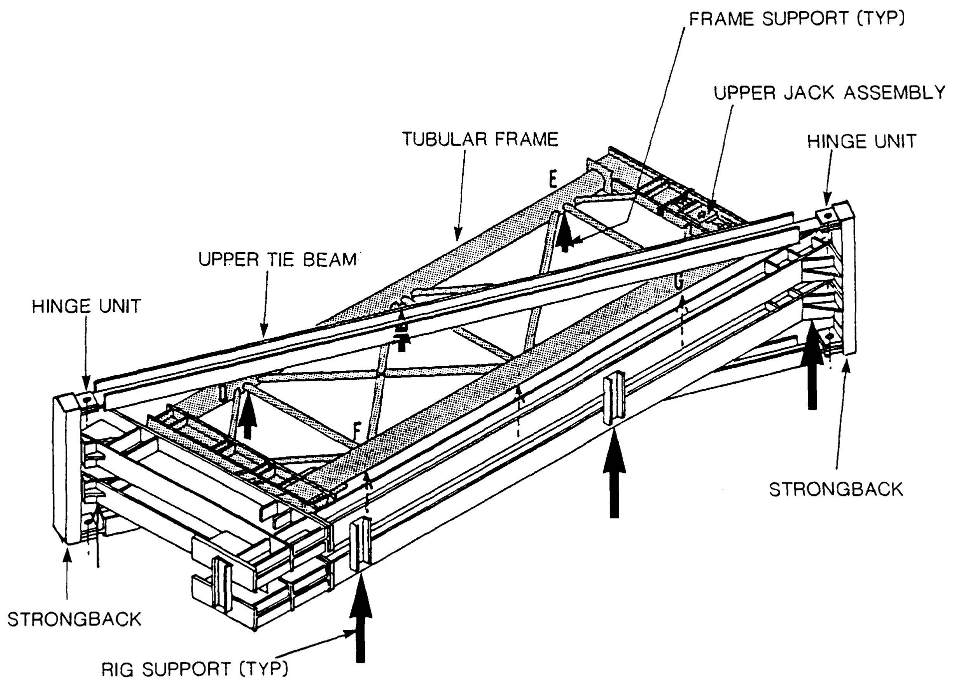

Fig. 1 Frame test general arrangement

The frames constitute the largest specimens ever to be tested to collapse in a controlled manner. General features of the test frames are summarised below:

- the four frames resembled prototypes as closely as possible, representing a scale factor of greater than one-third with respect to typical southern North Sea jacket structures.
- size effects were avoided by adopting minimum tubular diameters and thicknesses of 168mm and 4.5mm, respectively.
- non-dimensional parameters reflected current offshore practice.
- standard offshore fabrication procedures were adopted.
- the influence of boundary conditions was minimised by pinning the frame legs at the base.

The basic configuration of the two-bay, X-braced test frames, 15m high and 6m wide, is shown in Figure 2.

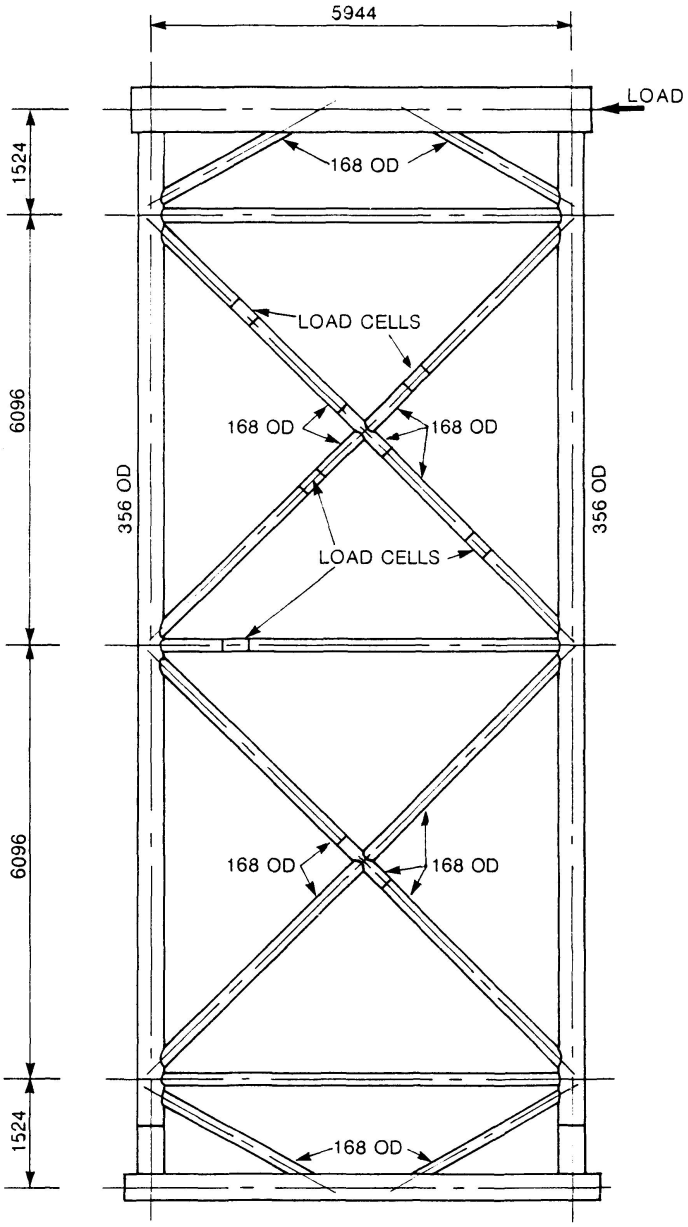

Fig.2 X-braced test frame configuration

Two-bay X-braced frames were selected for the following reasons:-

- X-braced configurations are a popular choice for offshore substructures.
- X-braced frame behaviour provided a stringent test for calibration of the computer program.
- Early generation X-braced jackets usually did not have joint cans, and this situation of practical concern was replicated by one test enabling joint failure and load shedding characteristics within the frame to be studied.
- No large scale X-braced frames of geometries typical of offshore construction had previously been tested.
- Two-bay frame tests were selected to enable load shedding between bay panels, important in the understanding of frame behaviour, and redundancy to be studied. X-braced panels exhibit a significant residual strength after failure of one member, and the presence of a second bay enables this form of behaviour to be fully and realistically captured without interference from end restraints.

The following features of the four frames distinguished their responses:

- Frames I and III were dominated by compression brace instabilities. The frames were nominally identical except that the horizontal brace between the two bays was omitted in Frame III, enabling its contribution to capacity to be assessed.
- Frames I and II were nominally identical, with the exception that the chord can for the top bay DT joint in Frame II was omitted such that joint collapse precipitated failure.
- Frame IV failure was initiated by propagation of a fatigue crack introduced at the critical DT joint of a specimen nominally identical to Frame II.

The frames were fabricated in Aberdeen and tested on site by the authors in a self-reacting rig, seen in the diagram in Figure 1 and the photograph, Figure 3.

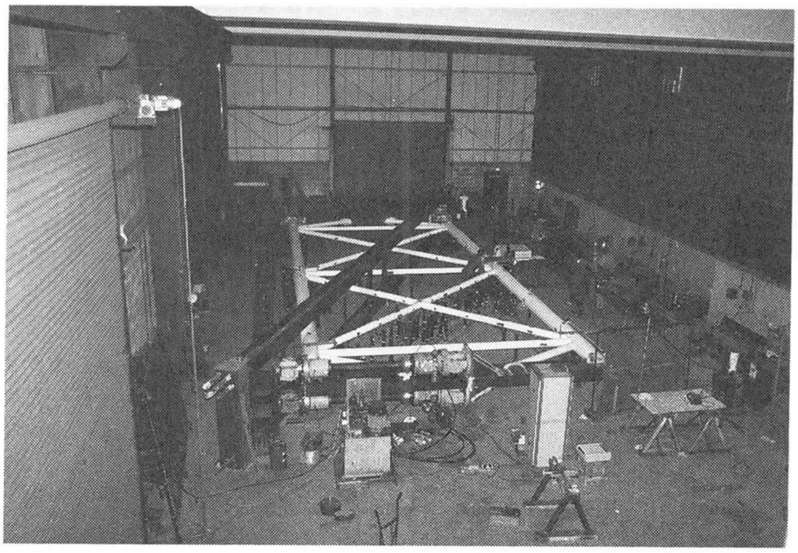

Fig. 3 View of Phase I frame test

The frames were subject to a thorough dimensional and straightness survey before and after testing and were monitored during loading by extensive instrumentation.

Lateral load was applied to the frames under displacement control; in undertaking the test programme, it was recognised that capturing the post ultimate behaviour was essential for the future calibration of the developed software. Previous experimental programmes had addressed the ultimate limit state of a component without due regard to the post-ultimate ductility. In order to ensure that the reserve and residual strength characteristics were fully assessed, all tests in the programme were carried out using equipment and test arrangements capable of handling large displacements in a controlled manner. In addition to the frame tests, tensile coupons, stub columns and tubular joint components were tested. Before discussing the findings from the tests, the scope of the parallel software development activities is reviewed.

2.3 Analytical Scope - Phase I

The purpose of the analytical work was to develop a non-linear numerical program to estimate the reserve and residual strength of tubular frames. The program developed has been called SAFJAC (Structural Analysis of Frames and JACkets) and has the capability to predict large displacement behaviour of plane and space frames including the effects of material plasticity. Development focused on simple structural idealisations and efficient non-linear solutions.

The analytical workscope began with a review of published data and information to define load-deflection characteristics of members and joints against which the existing finite element programs were calibrated. They were then used to develop new parametric relationships for the specification of joint stiffness curves in terms of both peak loads and associated displacements.

The numerical activities within the Project then concentrated on the development of a software package with new finite elements encompassing automatic sub-division to accommodate plasticity and with a non-linear representation of joints [3]. This route was chosen to complement commonly available numerical methods which are finite element based, some of which require several beam elements to monitor the P-delta effects associated with large deflections. Further, the new software contrasts with the phenomenological models available which, whilst computationally efficient, may be considered to require greater expertise from the analyst.

An elastic quartic element was developed to model each member in a frame. This was programmed to sub-divide automatically as plasticity occurred, introducing at its ends either a plastic hinge or a new cubic element devised to monitor the spread of plasticity. The facility to model non-linear joint characteristics was introduced in the program with a piece-wise linear representation. The new elements were calibrated in isolation and against frame test results from both the open literature and the experimental programme. In Phase II the elements and automatic subdivision facilities were extended to give full two and three-dimensional capability.

2.4 Findings - Phase I

The findings from the experimental programme have important ramifications for the repair and maintenance of existing installations as well as for the design of new structures. In addition to the wealth of information gathered from the associated tests, the frames themselves demonstrated substantial reserves of strength.

The findings can be summarised as follows:

- Significantly larger than expected reserve and residual strengths were recorded compared with individual component responses.
- Frames dominated by joint failure exhibited greater reserve strengths than those in which member failure occurred first.
- The relationship obtained in respect of interaction between joint loading-unloading characteristics and overall frame system response was unexpected for both intact and fatigue cracked critical joints.

For the first time, experimental and numerical evidence had been generated which seemed to indicate unusual and unexpected frame action effects for tubular joints. These findings potentially impact on all aspects of tubular joint design practices (both static and fatigue), from isolated joint testing procedures, (which may not be adequately capturing frame effects [4]), to joint data interpretation, failure definition, capacity and joint detailing practices. In recognition of these unexpected and significant findings, a Phase II project was proposed as an extension to Phase I, to address these issues within the context of reserve/residual strength calculations. This is described in the next section.

The role of redundancy in the X bracing was demonstrated by the ultimate frame response. As the compression joint in the top bay began to yield, so the compression load path softened. A greater proportion of the applied load was therefore distributed via the alternative tension diagonal enabling the structure as a whole to sustain increasing loads. This response may be contrasted with a K braced structure where the lack of redundancy through the joint ensures that failure of one component constitutes failure of a panel.

With regard to member failures, the frame tests enabled reserve and residual capacities to be quantified. Furthermore, comparison of results from tests I and III supplied much needed information about the role of horizontal bracing in the ultimate response of X-braced jacket structures. These two frames were tested specifically to illustrate the role of redundancy on the system reliability. Frames I and III were nominally identical but for the absence of the midheight horizontal (see Figure 2) in the latter case. In elastic design this member carries no load and with trends towards lighter, liftable jackets designers are being encouraged to omit these redundant members. The tests showed that although initial failure in both frames was by buckling of the compression brace in the upper bay, the post-peak response was severely compromised in the absence of the horizontal.

In this second case a rapid succession of failures was initiated with a residual frame capacity below the original design load. As the compression brace buckled, so a greater proportion of load was transmitted via the tension diagonal. At the midheight level the only path for the load was the lower bay compression diagonal which soon buckled. The redundancy afforded by the horizontal in the first instance however, had ensured a more even redistribution of load without initiating further component failures. At the midheight level the load from the top bay tension diagonal divided between the horizontal and lower bay compression diagonal.

The midheight horizontal constituted just 2.5% of the structural weight yet the alternative load paths that the redundancy afforded assured a factor of 1.3 on residual capacity. In terms of safety, the redundancy had a significant contribution.

Excellent agreement with the frame test results was achieved by SAFJAC analyses and this has ensured that the program may be used with confidence for the analysis of offshore jacket structures. Indeed, Participating Organisations are already performing 2-D and 3-D pushover analyses using SAFJAC as part of their reassessment of existing installations for re-certification.

In addition to conclusions regarding reassessment procedures, Phase I led to recommendations for the use of the reserve strength technology in the evaluation of new designs. The aim is to ensure that minimum operator-specified reserve strength factors are achieved, thereby enabling structural redundancy to be fully and safely exploited even though this requirement is not yet specifically stipulated in design codes. This approach should lead to efficient and versatile structures for which the need for in-service modifications and/or repairs is much reduced.

Other recommendations focused on the need for future work to examine the issues surrounding joint failure and frame mounted joint capacities, and for K configurations to be addressed. It was also acknowledged that SAFJAC should be developed and extended to enhance its capabilities. The importance of the findings from Phase I of the Frames Project gave impetus to the commencement of Phase II along the lines noted above.

3. FRAMES PROJECT PHASE II

3.1 Overview

A Phase II of the Frames Project was developed by the Participants and BOMEL, with the following objectives:

- To establish further the performance of joints in X-braced and K-braced frames and to investigate the effects of joint failures on the performance of these frames up to and beyond the ultimate limit state.
- To establish levels of reserve strength in X-braced and K-braced frames.

- To examine the effect of load reversals on the ultimate response characteristics of joints.
- To undertake lack-of-fit stress measurements.
- To enhance, calibrate and apply the non-linear numerical procedure SAFJAC for the collapse analysis of frames.

Work commenced in June 1990 and was recently completed. Of the frame tests, two were double-bay X-braced as in Phase I, (Figure 2), and four were of single bay K-braced configurations as shown in Figure 4.

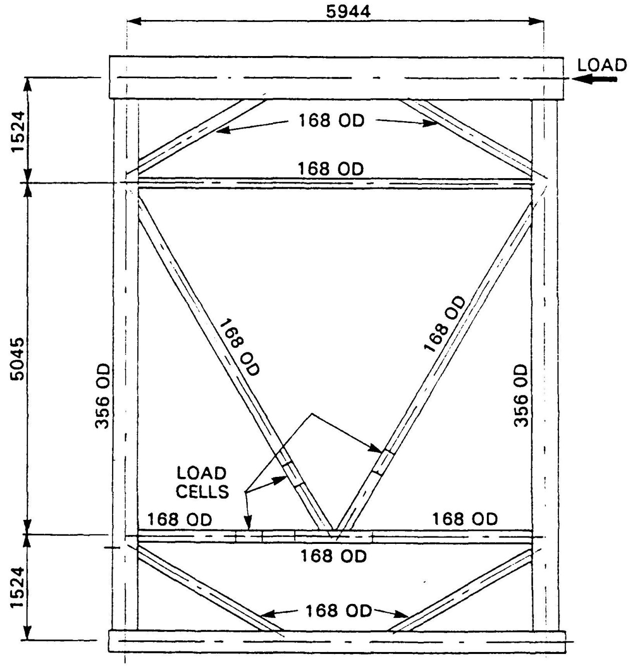

Fig. 4 K-braced test frame configuration

3.2 Experimental Programme - Phase II

Taking account of the findings from Phase I and recent numerical investigations into the ultimate response of frame mounted joints, six frames were tested within Phase II as follows:

- Frame V is nominally identical to the two-bay X-braced Frame II from Phase I and was tested up to the point of joint failure to verify the ultimate capacity of the joint in the frame.
- Frame VI reuses the Frame V structure but with a replacement joint, carrying compression in the through member rather than the braces. The test continued through joint failure up to the maximum displacement available in the rig, thereby investigating post-peak response.
- Frame VII, the first K-braced frame, is detailed for first failure of the simple gap K-joint. Frames VIII and X are variations of Frame VII with different diameter and gap ratios, β and ζ. For all the K-braced frames the tests continued such that both reserve and residual characteristics of the joint and frame were revealed.
- Frame IX, complements the Frame VII test being nominally identical but for the specification of a significant overlap at the K-joint.
- Companion tests of nominally identical isolated joints are an important part of the Phase II programme enabling the role of constraints within the frame to be quantified [4].

These results, coupled with detailed support monitoring and residual 'locked-in' stress measurements, have significantly advanced the understanding of the ultimate response of frames and joints.

4. CONCLUSIONS

From the description of the Joint Industry Tubular Frames Project presented in this paper, it can be seen that the tests are providing new and important information about the reserve and residual strength of structures. A number of significant and unexpected findings have been noted. For the first time, frame tests have been carried out where joint failure precedes member failure, a scenario in line with current design practices which dictate an equal likelihood of joint failure as of member failure. Frame behaviour has been observed which impacts on all aspects of tubular joint and frame design practices. In recognition of these unexpected and unanswered findings, a second phase of work was recently undertaken to develop the technology further. The parallel development of a calibrated numerical tool, SAFJAC, ensures that the findings can be directly applied to both planned and existing offshore jacket structures.

An increasing awareness of the need to quantify reserve and residual strength is being raised for a variety of reasons:

- Re-assessment of existing installation is being required more often due to:-
  - more onerous loading, as environmental conditions are reviewed,
  - additional topside loading to extend facilities or meet new safety criteria,
  - requirements to extend the platform life beyond its design value to exploit remaining hydrocarbon reserves,
  - deterioration of capacity through damage or corrosion.

  The intention of re-analysis is to assess the fitness for purpose of the structure under the modified load/resistance regime and to ascertain whether strengthening measures are essential for safe operation. In many instances, if the combined reserve strength resources could be demonstrated, the need for costly strengthening measure may be removed.
- Cost is also driving the trend towards lightweight structures with few primary members and a commensurate reduction in reserve and residual strength. The implications for such structures subjected to design storm loading or accidental loading can only realistically be assessed through pushover analyses.
- The advent of limit state codes requires a thorough understanding of the ultimate strength of structures in both the intact and damaged conditions. Hard evidence is being generated by the frames tests.

The data and information resulting from the Frames Project and the availability of a calibrated and substantiated non-linear numerical procedure will allow the safe and economic application of steel jacket structures. It will enable the proper exploitation of redundancy inherent in jacket structures whilst maintaining desired safety levels. In addition, it will provide an important tool in the reassessment of intact or damaged existing structures, leading to the identification of global safety levels and the development of inspection, maintenance and repair (or strengthening) procedures in a more rational, cost-effective and complete manner than has been possible in the past.

Analytical phases of the work are continuing and a 3D structural collapse test is planned to investigate load redistribution between structural planes and the role of redundancy.

5. ACKNOWLEDGEMENT

Phase I of the project was carried out by the authors under the auspices of the Steel Construction Institute. The software (SAFJAC), which was developed by personnel from Imperial College working under the direction of the authors, is confidential to the Participants.

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Stress Histogramme and Fatigue Life Evaluation of Highway Bridges

Historgrammes des contraintes et évaluation de la durée de vie à la fatigue des ponts-routes

Spannungshistogramme und Ermüdungslebensdauer von Autobahnbrücken

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SUMMARY

In the case of field measurement of stress histogramme acting on steel bridge members under traffic, a measuring device to measure stress ranges and their frequencies , which is called "Histogramme recorder", has been used. This paper presents the outline of the histogramme recorder and its applications to evaluating the condition of steel highway bridges in service.

RÉSUMÉ

Un dispositif appelé enregistreur d'histogrammes a été développé pour saisir l'évolution des contraintes, tant en amplitude qu'en fréquence, se produisant dans les éléments porteurs de ponts métalliques sous charge mobile. L'auteur présente cet appareil ainsi que son application pour l'évaluation de l'état des ponts-routes métalliques.

ZUSAMMENFASSUNG

Zur Aufnahme der Spannungsgeschichte an Bauteilen von Stahlbrücken unter Verkehr wurde ein sogenannter Histogramm-Rekorder entwickelt, der die Spannungsamplituden und ihre Auftretenshäufigkeit registriert. Der Beitrag stellt das Gerät und seine Anwendung bei der Zustandsbewertung von Autobahnbrücken vor.

1. INTRODUCTION

Total number of highway bridges defined that their lengths are 2 m or over is more than 650,000 in Japan and the number is increasing still more. Maintenance technology of existing bridges including inspection, diagnosis, repair and strengthening methods is most essential to keep them in service for long period. Safety of bridges is generally evaluated based on various informations such as damage conditions, traffic conditions, structural characteristics obtained through inspection or more detailed survey. Stress histogram measurement is one of direct and effective means to evaluate structural behavior of bridges or their individual components. In the past, it had been a time-consuming work for preparing and analysing obtained data when using former measurement devices. Now we have used a much simpler measurement device, which is called "Histogram recorder", in order to measure stress ranges and their frequencies acting on bridge members under traffic for about 10 years. It can obtain stress histogram from measured stress automatically in field on time.

This paper presents the outline of histogram recorder and its applications for checking load carrying capacity and durability for fatigue of steel highway bridges in service.

2. OUTLINE OF FIELD STRESS HISTOGRAM MEASUREMENT BY HISTOGRAM RECORDER

2.1 Outline of histogram recorder

The histogram recorder used is an device which consists of strain meter, amplifier, A / D converter, processor and memories to analyze stress ranges and their frequencies of histogram by digital process of analog data obtained from strain gages or other sensors. The histogram recorder we use has 8 channels to input and 8 data memories. Appearance of devices used for stress measurement is shown in Fig. 1 and Photo 1. The histogram recorder is small in size (200mm(W)X75mm(H)X120mm(D)) and light in weight(about 2kg). Histogram analyser in the figure is used only for initial setting and operation of histogram recorder, collecting of obtained data. It contains CRT, printer, and micro floppy disk drive. Obtained data can be checked in field on time. System disks and data disks are used for analytical

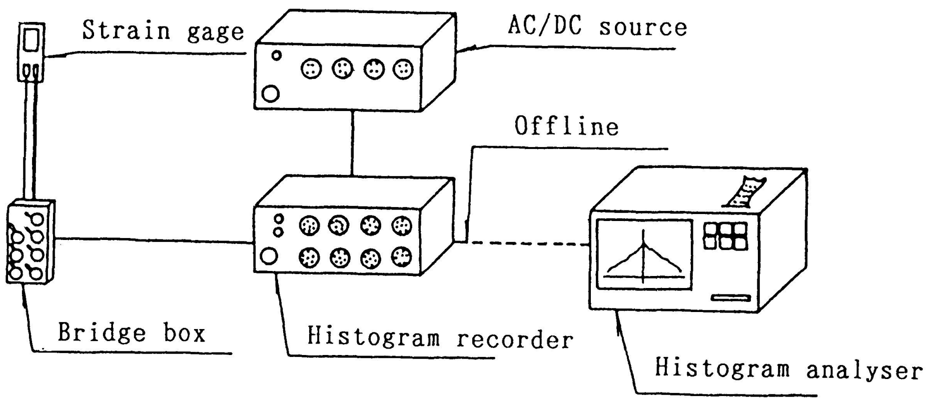


Fig. 1 System of measuring devices

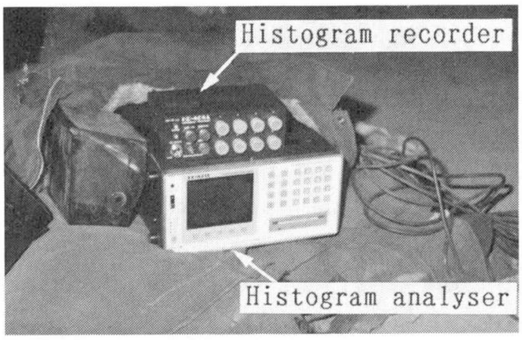
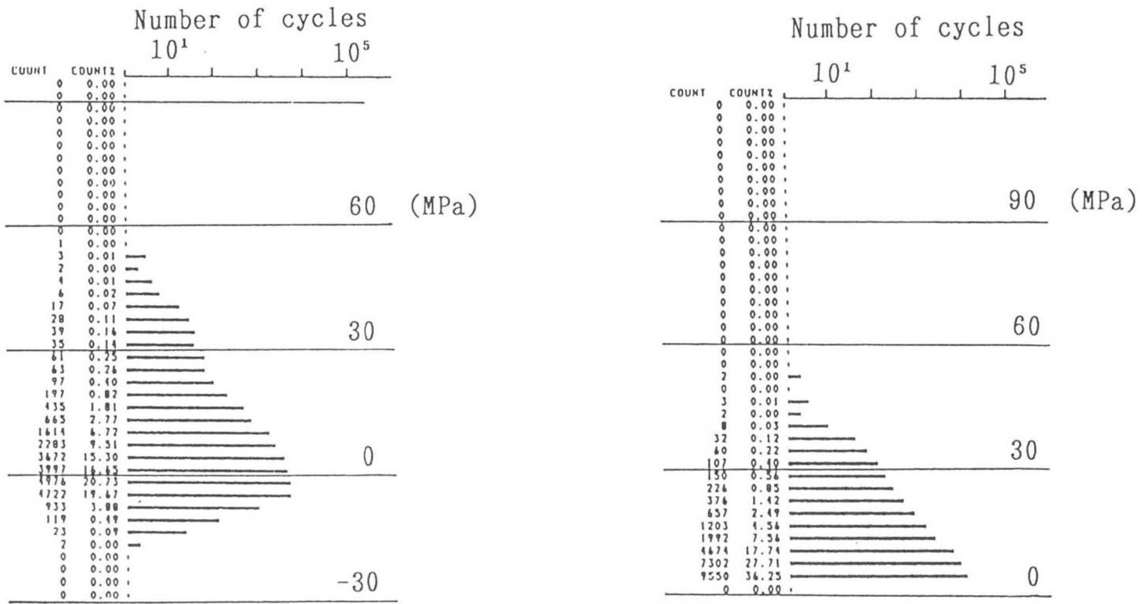


Photo 1 Appearance of measuring devices

Photo 2 Initial setting of histogram recorder in field



(a)Histogram by the Peak-valley method (b)Histogram by Rain-flow Method

Fig. 2 Stress range histograms

program setting, recording and storing the obtained data. Photo 2 shows initial setting of histogram recorder installed on a bottom flange of a main girder end in an I-shaped plate girder bridge using histogram analyser. The histogram recorder is small enough to be installed on the bottom flange.

2.2 Stress frequency histogram data and its utilization

Stress histogram measurement can be carried out automatically without disturbing traffic passing through bridge. All the operation needed to do in field are setting and data collecting before and after measuring period respectively. Obtained histogram data means the characteristics of structural response due to various effects such as traffic condition and impact by the irregurality of road surface and truck loading, etc.. These data provide us much information with respect to load carrying capacity or degree of fatigue damage of bridges in use.

Fig. 2 shows typical stress range histograms measured on a bottom flange of span center of a plate girder bridge during 24-hour period. (a) and (b) are obtained applying Peak-valley method and Rain-flow method to process measured stress respectively. In Peak-valley method, number of peak and valley appeared is recorded. The maximum stress obtained by Peak-valley method can be utilized to evaluate the load carrying capacity. Rain-flow method processing produces stress range histogram. Stress range histogram directly represents the accumulated fatigue damage of a bridge member during the measuring period. Remaining life for the fatigue damage can be estimated by applying the modified Miner's cumulative damage rule for such stress range histogram.

3. APPLICATION OF STRESS HISTOGRAM MEASUREMENT

3.1 Evaluation of load carrying capacity of an existing bridge

Almost half of highway bridges in service defined that their lengths are over 15m or over in Japan are designed by the design live load(DLL) of 14 ton truck or lighter because Japanese DLL was smaller before 1956. In trunk highway, most of such bridges have already been replaced or strengthened. However, a great number of bridges are still in use. The load carrying capacities of these bridges are generally evaluated for the present DLL. If the present DLL is simply applied to evaluate these bridges, the calculated stress naturally exceed the allowable stress. But with respect to most of these bridges, it can be considered a rare case that heavy trucks comparable to the present DLL come one after another on a bridge and make lines on it. Therefore, live load for evaluation of these bridges in use can be reduced less than the present DLL load according to the actual traffic conditions on the bridges. To clarify the live load, research based on computer simulation of traffic passing through bridge had been conducted in various research institutions in Japan. This paper proposes a more simple and practical evaluation method for administrators based on measured stress under traffic. As mentioned before, measured stress contains the various effects with respect to the structral characteristics and the traffic condition. Therfore, it would enable more rational evaluation of the load carrying capacity of these bridges.

Table 1 shows comparison between the measured maximum stresses during 24 hours and caluculated stresses at the span center of main girders of a plate girder bridge by present DLL. Fig. 3 shows the cross section of the bridge, which was designed by

Table 1 Comparison between measured maximum stress and caluculated stress

Main girder	Measured maximum stress σ_{max}(MPa)	Calculated stress by DLL σ_L(MPa)	$\sigma_a - \sigma_D$ (MPa)	$\frac{\sigma_a - \sigma_D}{\sigma_L}$	$\frac{\sigma_a - \sigma_D}{\sigma_{max}}$
G2	45.0	77.4	77.2	1.0	1.7
G3	57.0	92.3	65.6	0.7	1.2

12 ton truck. As one of limit states, we consider condition that residual deformation does not remain in bridge components, i.e. , in case of main girder, it does not yield. For example, with respect to a main girder, assuming that the acting stress mostly consists of stress components due to dead and live load, the value of $(\sigma_a - \sigma_D)$ is required to be larger than measured maximum stress due to actual live load as follows.

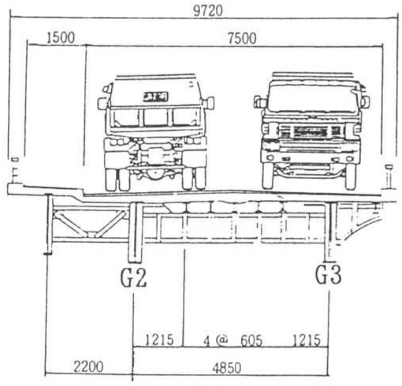


Fig. 3 Cross section of investigated bridge

$$(\sigma_a - \sigma_D)/\sigma_{max} > 1.0$$

where, σ_a = allowable stress for the steel grade of main girder

σ_D = calculated stress for the design dead load, if possible, actual dead load is better

σ_{max} = the measured maximum stress of main girder

As shown in table 1, the values of $(\sigma_a - \sigma_D)/\sigma_L$, where σ_L =calculated stress for the present design live load, are approximately 1.0 for girder G2 and 0.7 for girder G3, which mean the safety factors of main girders of this bridge. On the other hand, the values of $(\sigma_a - \sigma_D)/\sigma_{max}$ of girder G2 and G3 are 1.7 and 1.2 respectively, which are both larger than 1.0. The measured maximum stresses σ_{max} are less than calculated stresses σ_L for the the present DLL. It is confirmed that this bridge can be kept in use to present traffic unless the condition changes drastically. With extension of measuring period, more reliable evaluation would be possible.

3.2 Evaluation of durability for fatigue damage

3.2.1 Evaluation of the effect of repair and strengthening method

It had been considered that there is little chance for steel highway bridges in Japan being subjected to a load condition corresponding to present design live load, and therefore, fatigue have not been considered to be a serious problem. However, in this decade, various types of fatigue cracks caused by repetitive loading of heavy traffic, especially overloaded vehicles, have been reported mainly in secondary members subjected to stress concentration.

Photo 3 shows fatigue cracks which occurred at a connection between a main girder and a cross beam of an I-shaped plate girder bridge. Crack ① occurred top end of a vertical stiffener welded to top flange. Crack ② occurred on web at fillet-weld toe connecting web and top flange of main girder. Generally, crack ② occurred after crack penetration of top end welding of the vertical stiffener. These cracks, which are the most frequent of all cracks reported in Japan, do not

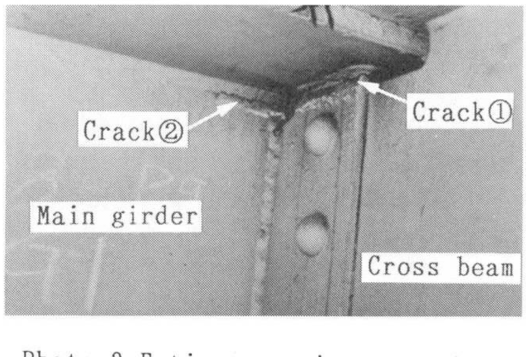


Photo 3 Fatigue cracks occurring at cross-beam connection

lead to a serious problem quickly. However, it can results in loss of durability of the bridge itself. The causes of damage are not clear enough so far. But at least, it can be said that these cracks were caused by secondary stresses due to relative deflection of main girders and deflection of RC floor slabs by wheel load of heavy traffic. The characteristics of damaged bridges and the effectiveness of repair and strengthening method have been investigated by directly monitoring stress histogram at the top ends of vertical stiffners where cracks possibly occur.

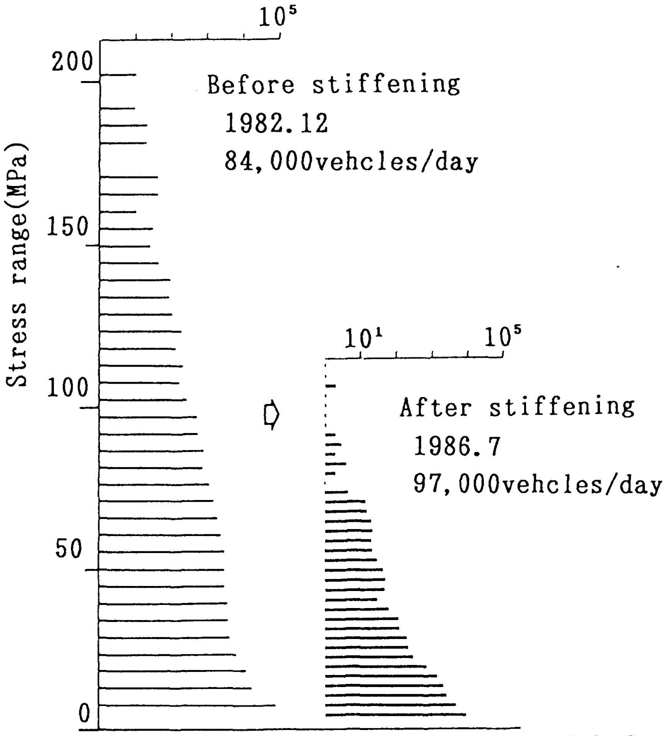


Fig.4 Stress range histograms measured before and after the stiffening

Fig. 4 shows the results of stress range histogram measured during 24 hours without traffic disturbance before and after stiffening of RC floor slab with additional stringers between main girders. The stiffening is one of the strengthening methods for RC floor slab. From this figure, the distribution of stress histogram can be seen shifted to much lower stress range after the stiffening. The fatigue life after the stiffening is estimated about 80 times as long as before. It is confirmed that the stiffening is effective for not only stiffening the RC floor slab but also reducing the local stresses at the cross-beam connection. This measurement could be carried out without disturbing traffic, therfore it is effective as one of the direct and easy approach for the investigation of fatigue damaged bridges.

3.2.2 Investigation of the characteristics of fatigue damaged bridges

With respect to the fatigue cracks mentioned in paragraph 3.2.1, to clarify structural chracteristics of damaged bridges, the relationship between structural parameter and measured local stress at cross-beam connection was investigated for each different bridge.

The equivalent stress range σeq(the root-mean-cube stress range), which will cause the same fatigue damage as the measured stress histogram for equal number of the cycles, derived from the modified Miner's cumulative damage rule is defined by the following equation:

$$\sigma eq = (\Sigma \sigma i^3 \cdot ni / N)^{1/3}$$

where, σi = stress range
ni = number of stress range σi
N = number of heavy traffic passing through the bridge during the

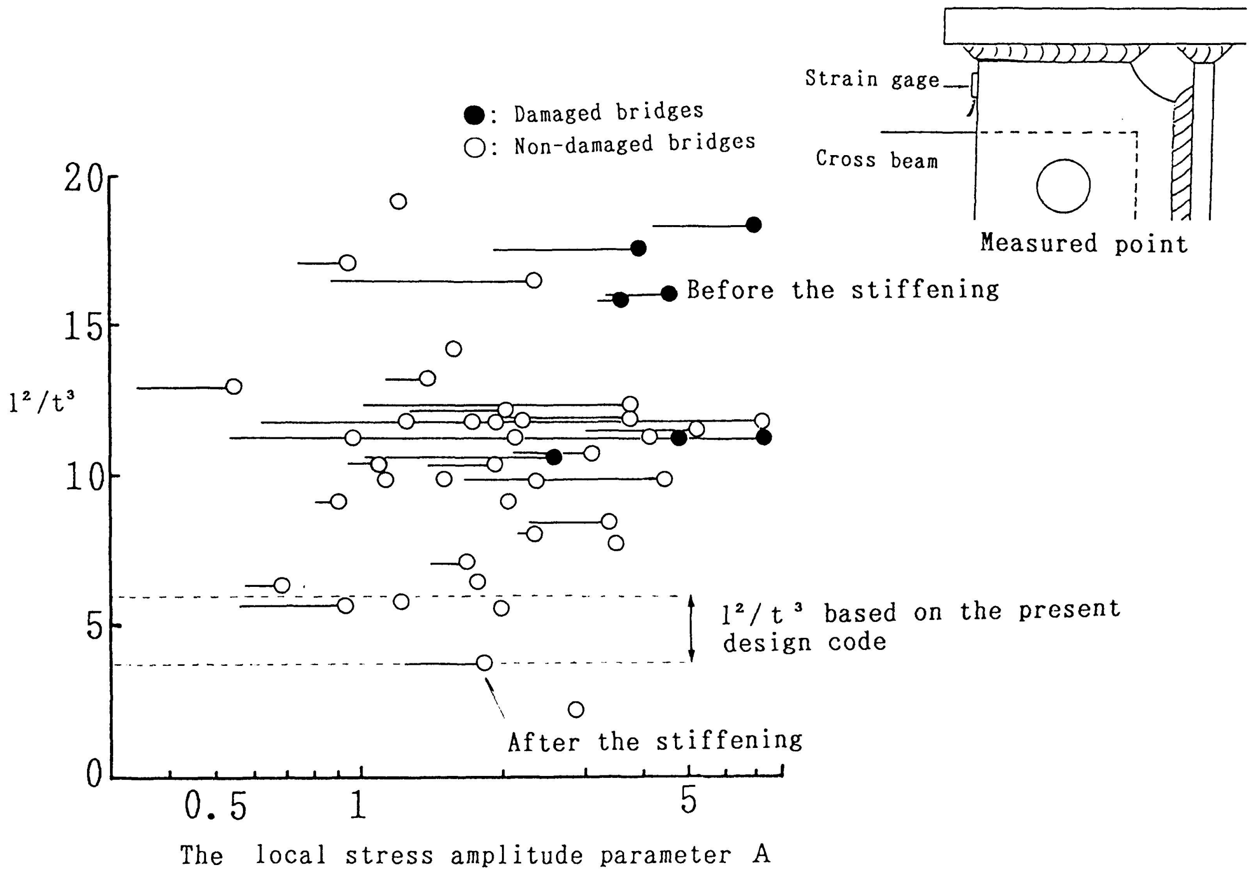


Fig. 5 Relationship between l^2/t^3 and local stress amplitude parameter A for each bridge.

measuring period$(=\Sigma ni)$

Suffix i is counted from maximum stress range side. As one of structural parameters of damaged bridges, a local stress amplitude parameter A is defined as follows:

$$A=\sigma eq/\sigma eq'$$

where, σeq and $\sigma eq'$ are equivalent stress ranges obtained from measured stresses at the cross-beam connection and bottom flange at the span center of main girder respectively. σeq is divided by $\sigma eq'$ to reduce influence of the traffic condition of each bridge.

Fig. 5 shows relationship between l^2/t^3 and the local stress amplitude parameter A for each bridge. l^2/t^3 is the same order as a rotational angle of RC floor slab on main girder web where l=spacing of main girders, t =thickness of RC floor slab. The local stress amplitude parameter A in each bridge is shown by solid line for the scatter range of A at different cross-beam connections. Black dots means fatigue damaged bridges at cross-beam connection. Also indicated in this figure, is the comparison before and after the stiffening of RC floor slab with addtional stringers mentioned in paragraph 3.2.1. By such direct approach, it is confirmed that occurrence of fatigue crack is influenced by the structural parameter l^2/t^3 to a considerable degree.

The Japanese design code of RC floor slab had been revised several times for the purpose of increase of its stiffness from the durability point of view. As shown in the figure, The values of l^2/t^3 for the RC floor slab designed by the present design code are approximately 3.8 to 5.8. These values are less than those of the fatigue damaged bridges. This indicates that the fatigue cracks are unlikly to occur at bridges designed by the present design code.

4. CONCLUSIONS

This paper presents the outline of histogram recorder and its applications. We are also trying to apply the histogram recorder for long time period measurement in order to monitor condition of bridge members, especially for the change of traffic condition and propagation of fatigue cracks. Field stress measurement had been conducted on more than 60 plate girders from fiscal 1986 as a 3-year-project in PWRI. Obtained data will contribute significantly not only to preparing both durability and load-carrying-capacity evaluation metohds of existing bridges but also to introducing fatigue design to future bridge design code.

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Field Testing and Safety Evaluation of Concrete Bridges
Essai sur site et évaluation de la sécurité des ponts en béton
Feldtest und Sicherheitsbegutachtung von Betonbrücken

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SUMMARY

This paper describes a method for safety evaluation of concrete bridges and its verification based upon field tests performed either under static loading by test trucks for application of the system identification method or under dynamic loading by falling mass for application to the modal analysis. The safety factors for flexural and shear failures were evaluated from these test results. Then, they were verified through the ultimate load test carried out in situ on the reinforced concrete main girders isolated by cutting off from the bridge system. Finally, the remaining life of the bridge was predicted by applying the fuzzy set theory which deals with the subjective information of bridge engineers.

RÉSUMÉ

Ce document décrit une méthode d'évaluation de la sécurité des ponts en béton et sa vérification d'après des essais sur place réalisés soit sous charge statique avec camions pour application de la méthode d'identification de système, soit sous charge dynamique avec chutes de masses pour application de l'analyse modale. Les facteurs de sécurité pour la flexion et la torsion ont été évalués à partir des résultats de ces essais. Ensuite, ils ont été vérifiés au cours d'un essai de charge limite mené sur le terrain sur les poutres principales isolées de l'ensemble du pont. Et à la fin, la durée de vie restante du pont a été prévue en appliquant la théorie de l'ensemble flou qui tient compte des informations subjectives des ingénieurs des ponts.

ZUSAMMENFASSUNG

Diese Arbeit beschreibt eine Methode zur Sicherheitsbewertung von Betonbrücken und eine dazugehörende Verifikation, basierend auf Feldtests, die entweder unter statischer Belastung durch Testlastwagen zur Anwendung der Systemidentifikationsmethode oder unter dynamischer Belastung durch fallende Massen für Anwendung der Modellanalyse durchgeführt wird. Die Sicherheitsfaktoren für Biege- und Bruchschäden werden auf der Grundlage dieser Testergebnissen bewertet. Danach werden sie durch einen endgültigen Belastungstest vor Ort auf Hauptträgern aus Stahlbeton, die durch Abschneiden vom Brückensystem isoliert sind, verifiziert. Schliesslich wird die Restlebensdauer der Brücke durch Anwendung der Fuzzy Set Theorie vorausgesagt, die subjektive Information von Brückeningenieuren verwerndet.

1. INTRODUCTION

It is an important problem for maintenance and rehabilitation of existing bridges to develop a method of safety evaluation such as remaining life and load carrying capacity of bridges. This paper describes a method of safety evaluation of concrete bridges in service and the verification of the evaluated results by field tests. The field tests to evaluate structural safety were performed as static loading test by test trucks for application to the system identification(SI) method, and also as dynamic loading test under forced vibration caused by falling mass for application to the modal analysis.

Both of the safety factors for flexural and shear failures and the change of dynamic behavior were evaluated from the field tests. These results were verified through the ultimate load test carried out in field on the reinforced concrete main girders isolated by cutting off from the bridge system. Finally, the remaining life of the bridge was predicted by application of fuzzy set theory which deal with the subjective information of bridge engineers. The fuzzy mapping which was determined based on questionnaire results performed on more than 20 experts was introduced for remaining life prediction of the existing bridges. A few concrete bridges on which field data have been collected are analyzed to demonstrate the applicability of this method. Through the application to the cracked reinforced concrete bridge girders, reasonable results were obtained by field tests.

2. FIELD TEST FOR STRUCTURAL SAFETY EVALUATION

2.1 Flow of Structural Safety Evaluation

Fig. 1 shows a general flow of safety evaluation and its verification for concrete bridges based on field tests. As shown in the figure, there are two types of field tests, one is non-destructive test to identify the system parameters which consist of the geometrical moment of inertia of both main girder and cross beam and Young's modulus of concrete, while the other is destructive tests such as ultimate load test and material tests to verify the evaluated results[1]. The non-destructive tests in situ are performed as static and dynamic loading tests, and deflection and acceleration are measured as mechanical behaviors. The stiffness(flexural rigidity) of each girder is identified by applying the SI method to the mechanical behaviors[2, 3], and then the section forces such as bending moment and shear force for the modeling of load variables are evaluated by structural analysis for the design load using the identified system parameters for each girder. Furthermore, the material tests are carried out for modeling of resistance variables linked with the statistical data of results of ultimate load tests which have been previously carried out on other bridges. Then, the structural safety for bending and shear failure are evaluated by calculating the safety factor, γ, safety index, β and the probability of failure, P_f. Finally, the ultimate load test is conducted in order to verify these evaluated results with the material tests.

2.2 Bridge Description

The bridges for which tests were performed are six national highway bridges as shown in Table 1. These bridges consist of five RC-T simply supported beam bridges and a RC-T continuous beam bridge, and all of the bridges are located in Hyogo prefecture, Japan. It is noted that the five simply supported bridges have almost the same bridge characteristics and history of service conditions except

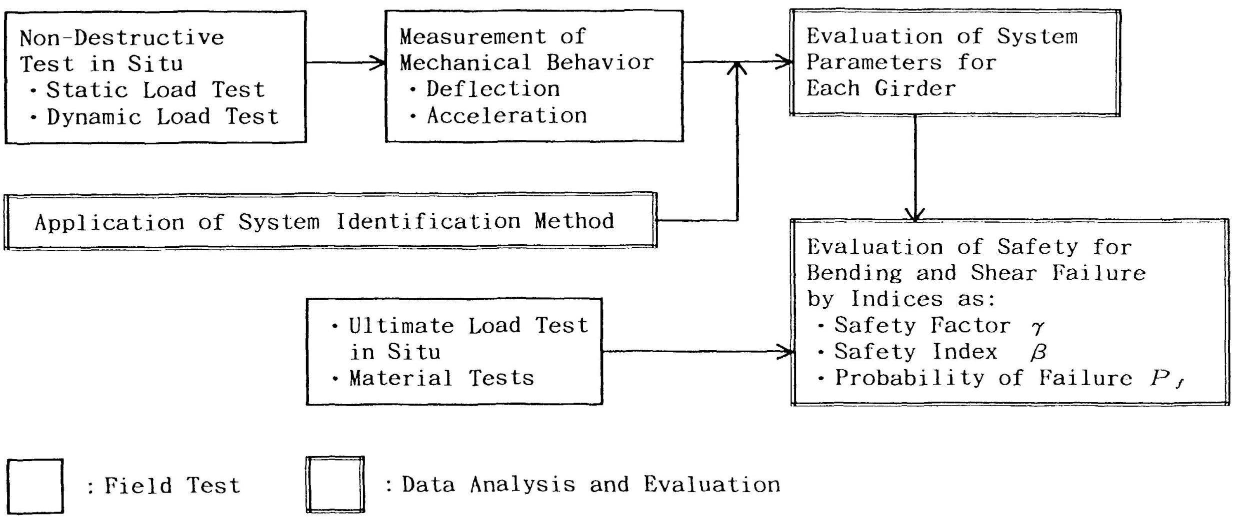


Fig. 1 Flow of safety evaluation and verification for concrete bridges

for age, number of girders, existence of cross members and type of handrail. The age of the bridges is in the region of about 40 to 60 years and the number of girders is three to five. Span length of all bridges is about 10m. Widening using an additional girder was performed on only Sakurabashi bridge in 1968. Fig. 2 illustrates the general view of Maenobashi bridge as example of the tested bridges.

2.3 Details of Field Tests

2.3.1 Non-Destructive Tests

Static loading was applied to the deck by a three-axled truck, loaded with crushed stones, with the total weight(about 20tf) being accurately measured prior to arrival on site. The truck was positioned in such a way as to cause a realistic severe loading condition on each girder as shown in Fig. 3. Positions of forced vibration by falling mass(300kgf) were arranged to obtain the various modes of vibration as shown also in Fig. 3. Typical loading procedure for the falling mass test is shown in Fig. 4. Mass dropping was carried out from about 70cm height, for ten times at the same loading point to cancel the white noise and to obtain a stable average value. Deflection and acceleration response of each girder for static and dynamic loading tests were measured with electronic deflection meters which have 1/1000mm accuracy and 10 mm capacity and ultra-small high capacity acceleration sensors which have constant frequency response of up to 700Hz and 20G capacity, respectively. Typical positions of the deflection meters and the acceleration sensors are shown also in Fig. 3. Modal analysis was applied on acceleration data to identify the modal parameters such as frequency, mode shape, damping, etc.

2.3.2 Ultimate Load Test and Material Tests

Test apparatus for the ultimate load test was specially designed to apply both of the static and dynamic load to the reinforced concrete main girders which were isolated by cutting them away from the bridge system. Fig. 5 shows the details of the loading system for the ultimate load test[1]. Static load was applied at the midpoint of the span via a 100tf capacity hydraulic jack reacting against a steel beam

Table 1 Outline of tested bridges

Bridge Name	Sakurabashi Bridge	Maenobashi Bridge	Taitabashi Bridge
Total Length	21.84m	45.80m	49.00m
Span Length	2@10.9m	5@9.16m	5@9.8m
Width	6.75m	5.50m	5.50m
Construction	1933 (Repaired in 1968)	1931	1950
Applied Spec.	1926 Edition(2nd Class)	1926 Edition(2nd Class)	1939 Edition(2nd Class)
Bridge Type	5 RC-T Simple Beam	4 RC-T Simple Beam	3 RC-T Simple Beam

Bridge Name	Nakaibashi Bridge	Oyasubashi Bridge	Aokibashi Bridge
Total Length	108.00m	45.90m	15.60m
Span Length	10@10.8m	3@14.7m	2.8+10+2.8m
Width	5.00m	7.30m	6.85m
Construction	1928	1962	1950(Repaired in 1969)
Applied Spec.	1926 Edition(2nd Class)	1956 Edition(1st Class)	1939 Edition(2nd Class)
Bridge Type	3 RC-T Simple Beam	4 RC-T Simple Beam	3 RC-T Continuous Beam

Fig. 2 General view of tested bridge (Maenobashi bridge)

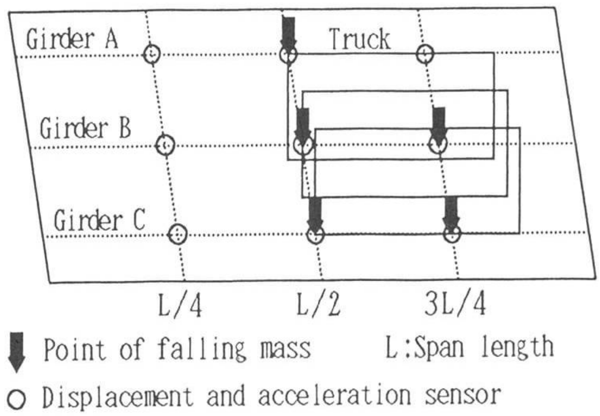
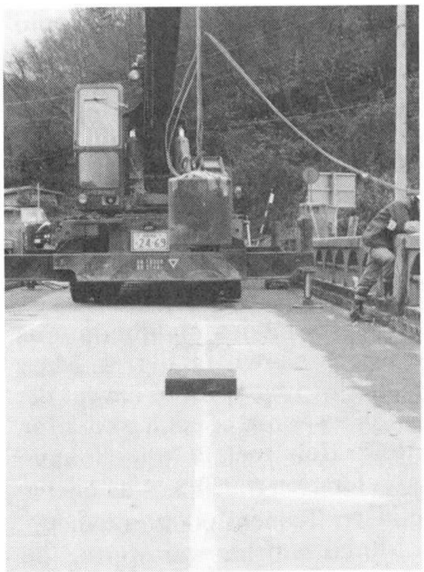


Fig. 3 Example of loading points and arrangement of sensors for static and dynamic tests



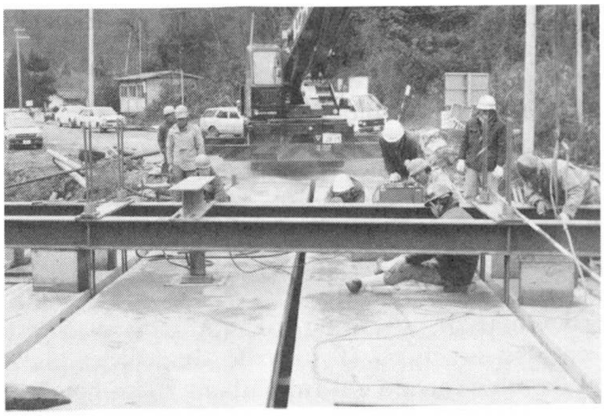


Fig. 5 Details of ultimate load test

Fig. 4 Details of falling mass (dynamic)test

embedded in the loading system. The loading scheme consisted of a series of loading-unloading cycles up to failure. The relationship between applied load and deflection at midspan was monitored using an X-Y recorder. On the other hand, a dynamic load test by falling mass at the midpoint of the span was carried out on each main girder after unloading in the static cyclic loading test, to evaluate the change in dynamic behavior up to failure. The acceleration responses were measured at a few points in the main girder. On the material tests, specimens of concrete cores and reinforcing bars extracted from all girders were tested to evaluate mechanical properties such as strength and modulus of elasticity and the depth of carbonization.

3. SAFETY EVALUATION AND ITS VERIFICATION

3.1 SI Method using Sensitivity Analysis

The SI method is one type of back analysis method, which can be used to identify system parameters such as flexural rigidity, corresponding to the degree of damage in the problem, by minimizing the error between the mechanical behavior as obtained from test and analysis. For the modeling of the target bridge in the SI method, a lumped mass(for modal analysis) model of gridform using finite beam elements possessing flexural, shearing and torsional rigidity, was applied for rationality in iterative calculation for this problem. In addition, this model has spring elements for friction restraint of rotation at the supports, corresponding to the progress of damage in the support region[4].

In the procedure of the SI method in this study, a sensitivity analysis of damage to mechanical behavior and the sequential linear programming(SLP) method were applied[4]. In this procedure, the objective function was defined as minimizing the total squared error between the mechanical behavior obtained from field tests and analysis. For the dynamic problem, the normalized objective function was defined as follows:

$$F = W_1\left(\frac{\mu_p}{\mu_p^m} - 1\right)^2 + W_2\sum_{k=1}^{n}\left(\frac{Z_{pk}}{Z_{pk}^m} - 1\right)^2 \rightarrow min \qquad (1)$$

where, p is the order of normal vibration, n is the number of measuring points, μ, μ^m are the eigen values obtained from analysis and field tests, respectively, Z, Z^m are the normalized modes of vibration, and W_1, W_2 are weights for the eigenvalue and vibration mode. Here, it is assumed that $W_1 = 1.00$, $W_2 = 1/n$.

For static loading, the objective function can be expressed by:

$$F = \sum_{k=1}^{n}\left(\frac{\gamma_k}{\gamma_k^m} - 1\right)^2 \rightarrow min \qquad (2)$$

where, γ, γ^m are deflections obtained from analysis and measurement, respectively.

Following this, identification of design variables can be performed by applying the SLP method using the objective function and its derivative for the design variables. Fig. 6 shows the flow of the SLP method for the dynamic problem. In the first step, the initial values of design variables such as flexural rigidity of the main girder and the spring coefficient of rotation of the support, are assumed, and modal parameters such as eigen values and vibration mode are evaluated by analysis. Next, linearization of the objective function is carried out within the region of movement limits for design variables, using Eq. (1), and a search for the minimum point of the objective value is tried using the simplex method. In the event that the change of design variables exceeds the movement limits, reanalysis of modal parameters and restart of the search for the minimum values from updated initial values are executed in the same procedure iteratively up to the stage in which the objective function is within the region of allowable limit.

3.2 Evaluation of Load Carrying Capacity

For material tests, the specimens of concrete core and reinforcing bar extracted from the main girders of the six existing bridges were tested to evaluate the deterioration of material. It was found that the compressive strength and modulus of elasticity of concrete were evaluated to be very low compared with the design value, and the degree of carbonization of concrete was very great. However, the effective section area of reinforcing bars providing tensile strength maintained their initial value, although surface corrosion was detected. Since the carbonization of concrete is considered to be the index of deterioration which includes the effects of quality of material and construction standards, traffic loading condition, permeability of slab and web concrete, acid environment, and so on, the relationship between the depth of carbonization and the compressive strength of concrete, as shown in Fig. 7, is suitable for the evaluation of the load carrying capacity considering the effects of these factors.

Ultimate load tests of main girders were carried out on five existing bridges, and results of these tests are summarized in Table 2, compared with calculated values based on the evaluation equations defined in the specification, and using compressive strength obtained from material tests. In this case, the load carrying capacity was defined to be bending moment at yield point of reinforcing bar for bending failure and shearing force at yield point of stirrup for shear failure. The results show that the calculated values give good agreement with the results of field tests, except for the value of bending failure on Taitabashi bridge which was affected by the bond failure between the reinforcing bars and concrete, and the value of shear failure on Sakurabashi Bridge, including the effect of scatter in the section area of the compression zone.

3.3 Modeling of Resistance and Load Variables

In the safety evaluation method, the probability model for load effect and load carrying capacity should be constructed considering the scatter of data and error in evaluation. The probability model for load carrying capacity can be evaluated by considering the correction coefficient corresponding to the ratio of the value obtained from the ultimate load test to that from analysis. The correction coefficient a is assumed to be a random variable characterized by the normal distribution, $N(\mu_a, \sigma_a)$ and then, the load carrying capacity for bending and shear failure can be expressed by[4]:

$$N(\mu_{Mut}, \sigma_{Mut}) = N(\mu_a M_{ucal}, \sigma_a M_{ucal}), \; N(\mu_{Sut}, \sigma_{Sut}) = N(\mu_a S_{ucal}, \sigma_a S_{ucal}) \quad (3)$$

where, M_{ucal} and S_{ucal} are calculation results of the load carrying capacity for bending failure and shear failure, respectively, and M_{ut} and S_{ut} are results of ultimate load tests for bending failure and shear failure, respectively. The coefficient, $N(\mu_a, \sigma_a)$ can be determined through comparison of estimated and measured values for load carrying capacity, as shown in Table 2 and Table 3.

On the other hand, for the evaluation of the probability model for load effect, the mean value, μ_S and standard deviation, σ_S of the section force, S for total weight of the three-axled truck, $W = N(\mu_W, \sigma_W)$ can be expressed by the following equation using the influence surface[4]:

$$\left.\begin{aligned} \mu_S &= \mu_W \left(0.1 \sum (\eta_L + \eta_R)_F + 0.4 \sum (\eta_L + \eta_R)_R\right) \\ \sigma_S &= \sigma_W \left(0.1 \sum (\eta_L + \eta_R)_F + 0.4 \sum (\eta_L + \eta_R)_R\right) \end{aligned}\right\} \quad (4)$$

Here, the distribution of the total weight of three-axled truck was assumed to be as given in the results of research by Hanshin Expressway Public Corporation [5].

In this way, the safety factor, γ, safety index, β and probability of failure, P_f can be evaluated by the

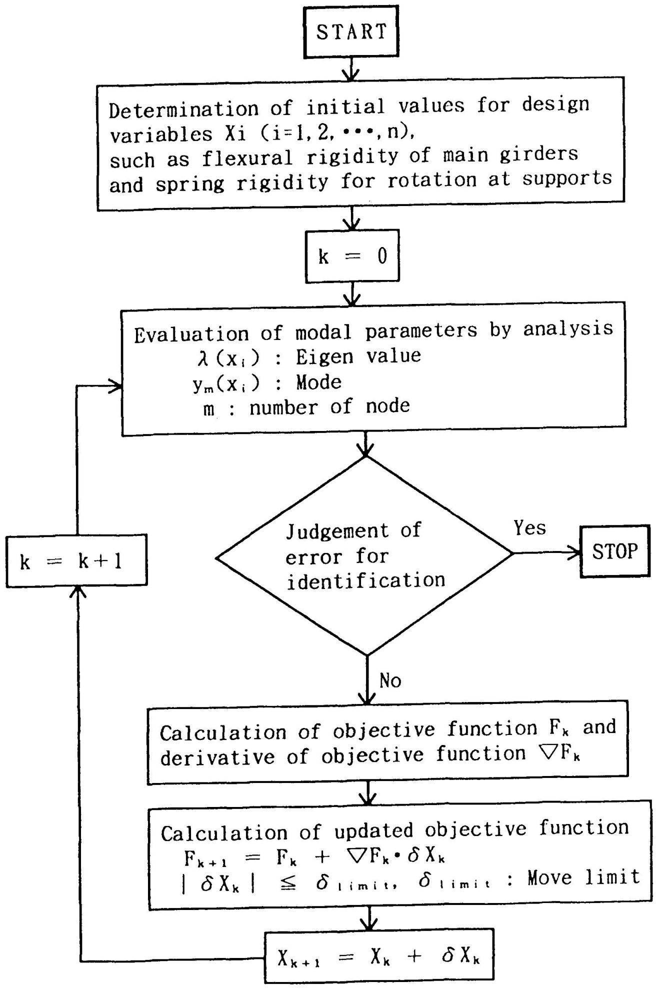


Fig. 6 Flow of sequential linear programming (for dynamic problem)

Table 2 Comparison of estimated and measured values for load carrying capacity

(a) Case of bending failure (tf•m)

Bridge Name	Span No.	Girder	M_{ucal}	M_{ut}
Maenobashi	1	C	73.28	78.22
	2	B	73.28	74.15
	2	C	73.28	69.82
Taitabashi	1	B	78.51	34.46
	2	B	79.24	41.30
	3	B	79.77	56.23
Oyasubashi	3	B	318.4	388.78

(b) Case of shear failure (tf)

Bridge Name	Span No.	Girder	S_{ucal}	S_{ut}
Sakurabashi	1	C	16.43	12.49
	1	D	16.43	9.92
Nakaibashi	1	B	18.20	22.29
	2	B	25.85	25.59
	3	B	26.63	26.59
Oyasubashi	1	B	57.91	62.49
	1	C	56.20	64.09
	2	B	53.61	75.24
	2	C	54.88	62.10

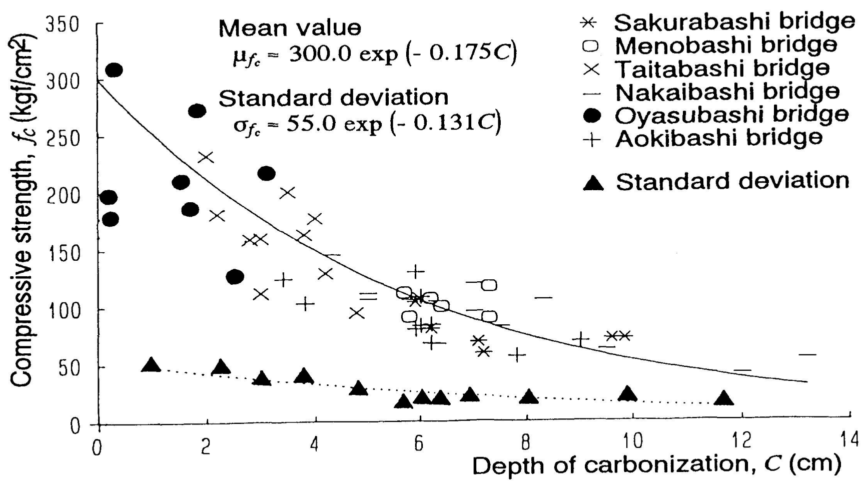


Fig. 7 Relationship between compressive strength and depth of carbonization for concrete

Table 3 Ratio of measured value to estimated value for load carrying capacity

	M_{ut}/M_{ucal}	M_{ut}/M_{ucal} (ex. Taitabashi)	S_{ut}/S_{ucal}
Mean Value	0.845	1.063	1.034
Standard Deviation	0.294	0.115	0.239

following equations:

$$\gamma = (\mu_{Fu} - 1.1 \cdot \mu_{Fd}) / \mu_{Fl}, \quad \beta = (\mu_R - \mu_S) / \sqrt{\sigma_R^2 + \sigma_S^2}, \quad P_f = P((R-S) < 0) \qquad (5)$$

3.4 Evaluation Results and Verification

The proposed method for structural safety evaluation based on field tests was applied to the existing bridges as shown in Table 1. The flexural rigidity identified by applying the SI method to results of field tests, and the safety evaluation based on safety factor, γ, safety index, β and probability of failure, P_f are summarized in Table 4. The reduction of the flexural rigidity of the oldest bridge, Nakaibashi bridge, was found to be remarkable compared with the design value, that is, the detected degree of damage for this bridge is very large. From the comparison between the safety evaluation for bending failure and shear failure, it can be judged that the failure mode for Aokibashi bridge would be of the bending type and that for Sakurabashi and Nakaibashi bridges would be of the shear type. In particular, since the probability of failure for Nakaibashi bridge is extremely large compared with the standard for the initial condition, of which the range is about 10^{-3} to 10^{-5}, the safety condition of this bridge seems to be critical from the bridge maintenance viewpoint.

The comparative study between the above mentioned evaluation and the results of the ultimate load test or material tests, can be considered to be available for the verification of suitability and utility of the proposed method. Firstly, for the girder *B* of Nakaibashi bridge, of which the degree of damage was estimated to be the largest of all tested bridges, both results of Table 4 and the ultimate load test can be seen to show that the failure mode of this girder would be of the shear type. Similarly, for the girder of Taitabashi bridge which has the second highest grade of damage of all tested bridges succeeding Nakaibashi bridge, both results of Table 4 and the ultimate load test suggest that the failure mode would be of the bending type. Since remarkable damage was not detected in the evaluation for inside girder of Oyasubashi bridge, the possibility and verification of bending failure or shear failure cannot be distinguished exactly. Accordingly, the prediction of the failure mode at the stage of heavy damage can be considered to be appropriate. In addition, the results of the SI method and safety evaluation shown in Table 4 can be found to be adjusted by the results of material tests, where compressive strength and modulus of elasticity of concrete for Nakaibashi bridge are lower than half the design values. For Aokibashi bridge, compressive strength and modulus of elasticity of concrete were about half the design values and the order of the resulting values of the three girders *A*, *B* and *C* was *A* > *B* > *C*. Therefore, the results of the SI method shown in Table 4 can be seen to be reasonable.

Table 4 Summary of structural safety evaluation results for existing concrete bridges

Bridge Name	Girder	Flexural Rigidity ($\times 10^4$ tf·m)		Safety Evaluation					
				Bending Failure			Shear Failure		
		Design Value	Estimated Value	γ_M	β_M	P_{fM}	γ_S	β_S	P_{fS}
Sakurabashi	A	7.66	6.14	3.88	2.173	1.483×10^{-2}	2.53	1.934	2.631×10^{-3}
	B	7.01	2.88	8.22	2.789	2.626×10^{-3}	4.31	2.543	5.451×10^{-3}
	C	7.01	1.68	16.12	3.174	7.502×10^{-4}	8.94	3.318	4.494×10^{-4}
	D	7.66	4.73	5.64	2.422	7.685×10^{-3}	4.39	2.525	5.734×10^{-3}
	E	13.22	7.80	3.75	2.097	1.793×10^{-2}	2.68	2.280	9.872×10^{-3}
Taitabashi	A	8.38	5.00	4.25	2.424	7.665×10^{-3}	3.64	2.594	4.740×10^{-3}
	B	10.03	6.23	5.34	2.491	6.363×10^{-3}	5.05	2.806	2.508×10^{-3}
	C	13.25	6.63	3.68	2.259	1.194×10^{-2}	3.37	2.483	6.506×10^{-3}
Nakaibashi	A	25.90	7.91	4.80	2.380	8.664×10^{-3}	1.80	1.326	9.238×10^{-2}
	B	17.21	9.11	5.49	2.414	7.886×10^{-3}	1.97	1.383	8.332×10^{-2}
	C	25.90	8.93	4.31	2.265	1.176×10^{-2}	1.67	1.185	1.181×10^{-1}
Oyasubashi	A	101.04	110.25	7.68	2.567	5.103×10^{-3}	7.02	2.899	1.846×10^{-3}
	B	88.32	92.27	16.32	2.877	1.998×10^{-3}	11.55	3.013	8.355×10^{-4}
	C	88.32	86.41	16.02	2.880	1.980×10^{-3}	14.00	3.050	1.128×10^{-3}
	D	101.04	89.68	8.41	2.638	4.158×10^{-3}	7.96	2.895	1.272×10^{-3}
Aokibashi	A	9.98	5.74	4.23	2.367	8.916×10^{-3}	4.58	2.797	2.561×10^{-3}
	B	10.87	4.48	5.86	2.560	5.225×10^{-3}	6.12	2.866	2.058×10^{-3}
	C	18.50	3.21	5.29	2.578	4.973×10^{-3}	5.91	3.005	1.342×10^{-3}

4. REMAINING LIFE PREDICTION

Based upon the above mentioned evaluated results, it is of interest to predict the remaining life of the bridges. However, the practical procedure for remaining life prediction involves an uncertainty scheme such as engineering judgement based on technical knowledge with professional experience. Therefore, the fuzzy set theory was introduced in a process of remaining life prediction for dealing with the subjective uncertainty.

The relationship between the integrity(soundness), S and the remaining life, t is assumed based on questionnaire results for the serviceability of actual bridges(Maenobashi bridge, Taitabashi bridge and Nakaibashi bridge). The questionnaires were performed on more than 20 experts for each bridge. The remaining life, t can be assumed as in the following equation[6]:

$$t = 3.03 \times 10^{-1}\, S^{0.98} \tag{6}$$

Since the deviation of the data from the equation is very large, the treatment of the subjective uncertainties included in this evaluation should be considered. In order to represent these uncertainty, the "kernel" of log-normal distribution is introduced for membership function of fuzzy set theory[6].

$$\mu_R(s, t) = \frac{t_m}{t} \exp\left[-\frac{1}{2}\left(\frac{\ln[t] - \lambda_S}{\phi}\right)^2\right] \quad \text{where,} \quad \lambda_S = \ln[t_m] = \ln\left[\left(3.03\times10^{-1}\right) S^{0.98}\right] \tag{7}$$

Next, the membership function for the remaining life, t is related to the membership function for the integrity, S by Fuzzy synthesis with aid of fuzzy relation, R[6]. Relative prediction of remaining life for the bridges is performed by integrating the forecast of respective remaining life, T_i based upon the corresponding membership function obtained from evaluated results for each judgement factor.

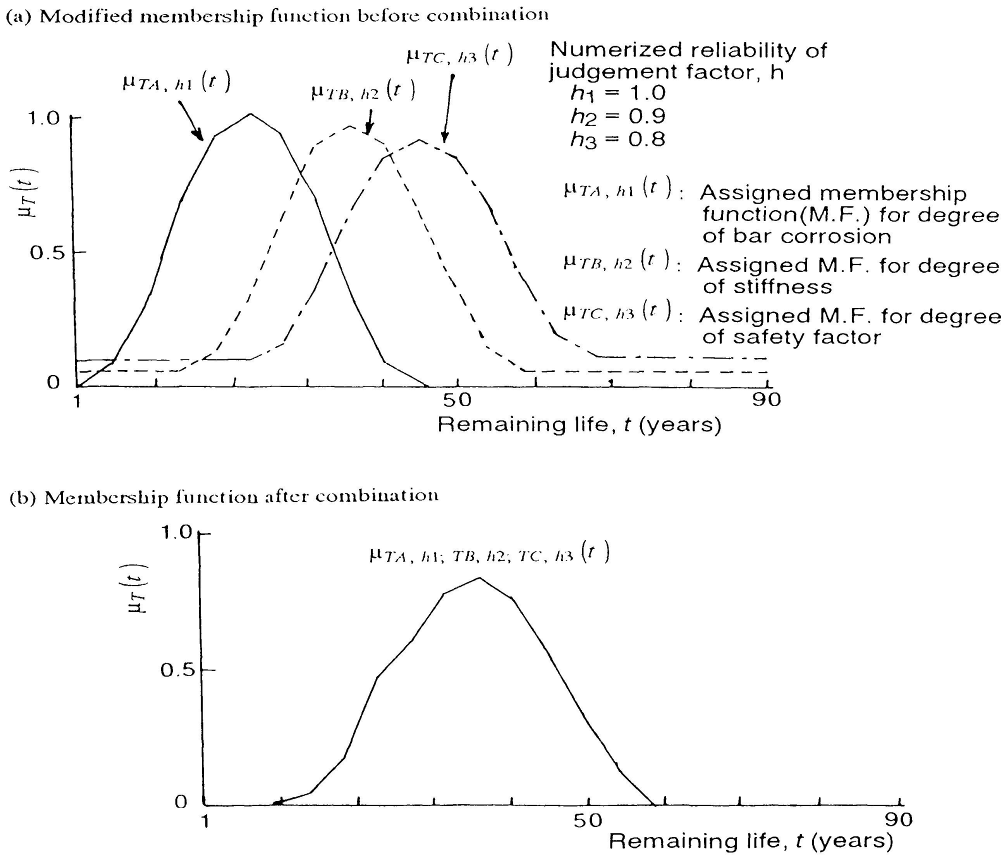


Fig. 8 Prediction of remaining life

As an example, Fig. 8 shows the result of calculating the remaining life for Taitabashi bridge using the "degree of bar corrosion", "stiffness of girder" and "safety factor" evaluated by field inspections and tests as a measure of the integrity. From this figure, the remaining life of the bridge is predicted to be about 35 years corresponding to the peak value of the membership function after combination. Furthermore, the predicted values for Sakurabashi bridge, Maenobashi bridge and Nakaibashi bridge are roughly about 15 years, 18 years and 18 years, respectively.

5. CONCLUSIONS

The major part of bridge diagnosis which is the kernel of the systematization for bridge maintenance is to develop a method of safety evaluation on items such as remaining life and load carrying capacity. The main conclusions obtained in this study can be summarized as follows:
(1) A few examples of the field testing and the measuring procedures, the evaluation methods of both the safety factors, including the safety index and the probability of failure, for flexural and shear failure and the verification concept of the evaluated results by field tests were performed on six reinforced concrete bridges.

(2) The prediction of remaining life of concrete bridges in service, which is a major problem faced in the process of bridge maintenance, can be quantitatively evaluated based on the evaluated results for the judgement factors. The relation among the factors are obtained through the fuzzy relations in the questionnaire results performed on experts.

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Interpretation of 200 Load Tests of Swiss Bridges

Interprétation de 200 essais de charge de ponts en Suisse

Auswertung von 200 Belastungsversuchen an Schweizer Brücken

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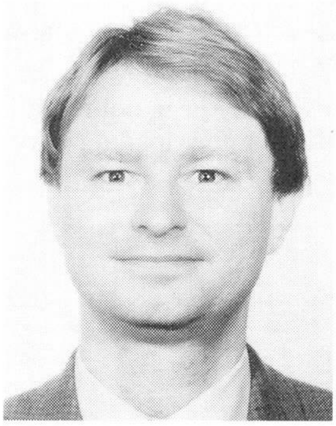
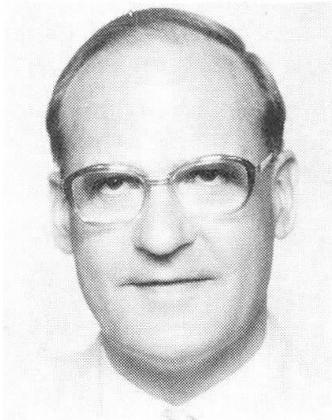




SUMMARY

The Institute of Reinforced and Prestressed Concrete (IBAP) of the Swiss Federal Institute of Technology has organized and interpreted over 200 load tests over the past 20 years. A full-scale load test is a real opportunity to observe and understand the behaviour of the bridge. A database containing the characteristics of the bridges along with their statistic and dynamic behaviour during the load test was established. The results of the statistical analysis of the database are presented. Some answers are proposed of the effective modulus of elasticity and the contribution to the rigidity of secondary elements, such as parapets or asphalt layer. The problem of estimating the structural capacity based on the results of load tests is discussed.

RÉSUMÉ

L'institut de béton armé et précontraint (IBAP) de l'Ecole Polytechnique de Lausanne a organisé et interprété plus de 200 essais de charge ces 20 dernières années. Un essai de charge en vraie grandeur est une réelle chance pour observer et comprendre le comportement d'un pont. Une base de données, contenant les caractéristiques des ponts avec leur comportement statique et dynamique observé lors de l'essai de charge, a été établie. Les résultats de l'analyse statistique de cette base de données sont présentés. Quelques réponses sont proposées pour le module d'élasticité effectif et pour la contribution à la rigidité des éléments secondaires, comme les parapets ou le revêtement. La problématique de l'estimation de la capacité structurale basée sur les résultats de l'essai de charge est abordée.

ZUSAMMENFASSUNG

Das Massivbauinstitut der Eidg. Technischen Hochschule Lausanne hat während der letzten 20 Jahre über 200 Belastungsproben durchgeführt. Ein Grossversuch im Masstab 1:1 ist eine Gelegenheit, um das Verhalten eines Bauwerkes zu beobachten und zu verstehen. Eine Datenbank wurde aufgestellt mit dem Gegebenheiten der Brücken und dem Verhalten während des statischen und dynamischen Belastungsversuches. Es werden Resultate einer statischen Analyse gezeigt. Es wird auf die Problematik des E-Moduls eingegangen sowie auf die Mitwirkung von Bordüre und Belag. Eine direkte Vorhersage der Tragsicherheit lässt sich allerdings aus einem Belastungsversuch nicht herleiten.

1. INTRODUCTION

For the past 20 years, the Institute of Reinforced and Prestressed Concrete (IBAP) has been involved with full-scale load testing of bridges (Fig. 1). This has resulted in over 200 bridges being tested. The Swiss Codes recommend a load test for any new bridge with spans exceeding 20 m [1-2].

The objective of a load test is to determine and quantify the global behaviour of a bridge. The majority of load tests are acceptance tests, aimed at examining the serviceability of a new bridge, and in consequence put it into service or not. The decision of acceptance is generally based on the concordance between the measured and calculated deflections, on residual deformations, on cracking and on the affinity between measured and calculated deflected shapes. Experience shows a strong correlation between an unsatisfactory behaviour during a load test and an abnormal long-term behaviour of the bridge, characterized by a non-stabilization of cracking and sagging. Therefore an abnormal behaviour of the bridge under a load test is an alarm signal, generally leading to more frequent inspections and early maintenance work.

The main results of these two hundred tests have been collected in a computerized database. General conclusions on the real behaviour of a bridge under loading have been collected and are presented in this article.

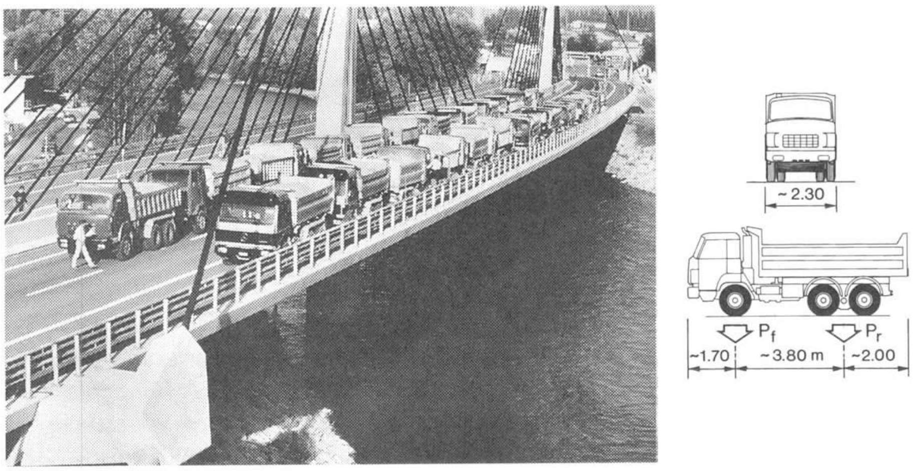


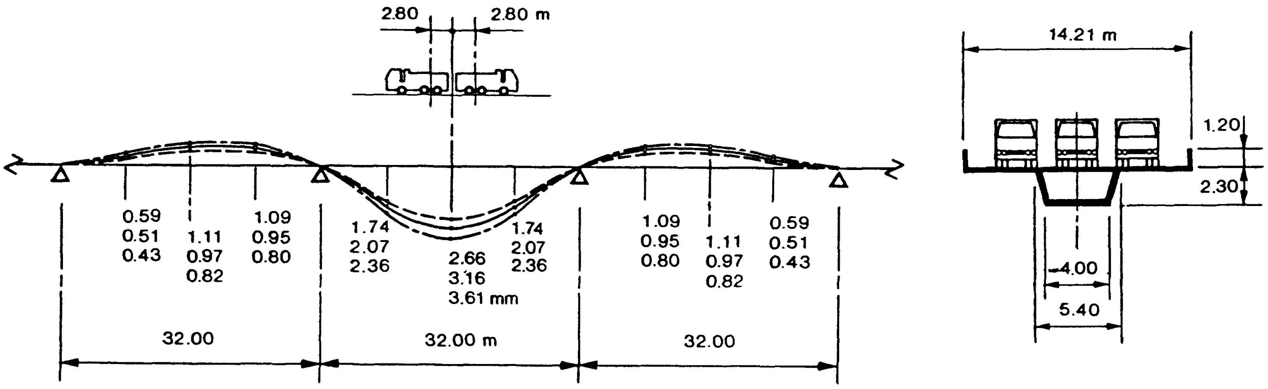
Fig. 1: Static load test of the Chandoline Bridge. The nominal dimensions of the lorries used for the load testing of bridges are also shown: $P_f = 60$ kN, $P_r = 190$ kN, $P_{total} = 250$ kN

2. INTERPRETATION OF LOAD TESTS

The main criterion to evaluate the behaviour of a bridge subjected to a load test is the concordance between measured and calculated deflections. On the measuring side, qualified operators, high precision instruments, and the repetition of each load case at least three times lead to highly accurate measurements. Temperature effects are eliminated by frequent "zero readings". On the computing side, more uncertainties are present, because of several parameters which are only imperfectly known.

2.1 Modulus of elasticity of concrete

The most important unknown parameter for the calculation of deflections is the effective modulus of elasticity of the concrete. Code formulas for estimating the modulus of elasticity of concrete based on concrete strength only are notoriously inaccurate. The modulus of elasticity is strongly influenced by local parameters such as the aggregates, the composition of the cement paste and the cure of the concrete. This complexity is increased by the fact that the actual modulus of elasticity is time- and strain-dependent. One possibility to determine directly this parameter is by testing cores taken from the bridge. Unfortunately, a limited number of cores is not necessarily representative of the entire structure; micro-cracking of the core may have occurred during its extraction and influence the results. Another possibility for determining the modulus of elasticity is to use non-destructive methods such as ultrasonic measurements. It is in all cases desirable to have results from samples taken during construction. The influence on the calculated deflections of the method used for determining the modulus of elasticity is shown in Fig. 2, taking the example of the viaduct of Coudray.



------ Ec=38 GPa from elastic modulus-compressive strength relationship (SIA),
——— Ec=32 GPa from cores, —·—·— Ec=28 GPa from ultrasonic pulse velocity

Fig. 2: Calculated deflections for viaduct of Coudray based on three methods for determining the modulus of elasticity

2.2 Effective moment of inertia

At first sight, it would seem that the moment of inertia of the cross section is easily determined as it depends only on geometry. However, the effective inertia depends on several parameters such as reinforcement, parapets, asphalt deck surface, cracks and micro-cracks due to construction or other causes.

In the past, these parameters have been roughly taken into account in the evaluation of the results of load tests by adjusting the modulus of elasticity of the concrete. To cover the participation of such secondary elements, the value of the modulus of elasticity was taken as 40 GPa in most cases. A better solution is to base the calculated deflections on the real modulus of elasticity of the concrete and on a better estimate of the effective inertia of the cross section.

3. DATABASE

The differences between calculated and measured deflections during load testing led IBAP to establish a computerized database. The database contains the main characteristics of each bridge along with the results of the static and dynamic load tests. The objective of this database is a statistical study of bridge behaviour as observed during load tests. The influence on the behaviour of the type of structure, cross-section, parapet and other factors can be discovered and quantified by sorting and filtering the results.

Table 1 shows the structural system and type of cross-section of all bridges contained in the database. One hundred and sixty five of the tested bridges are post-tensioned concrete structures.

Structural system		Cross-section	
Type	Number	Type	Number
beam	168	box-girder of constant depth	55
rigid portal frame	21	box-girder of variable depth	25
arch	5	slab beam	28
cable stayed	3	open (beams connected by concrete deck)	89

Table 1: Structural system and type of cross-section of all bridges contained in the database

3.1 Selection of bridge type and statistical parameters

In order to obtain a homogeneous set of results, the analysis focused on intermediate spans of post-tensioned bridges with the structural system being a continuous beam. Because it was anticipated that the influence of massive R.C. parapets and of an asphalt layer would be important, bridges for which no information was available on the presence of these items at the time of testing were also rejected. Finally, bridges for which the ratio of measured to calculated deflections were inferior to 0.65 and superior to 1.30 were rejected because of doubts on the reliability of the engineer's calculations.

Sixty six bridges satisfied these criteria. In this article, "R_m" is used for the average ratio of measured to calculated deflections. At mid-span of the loaded span, $R_m = 0.94$ and $\sigma = 0.16$ (standard deviation) for the sample of 66 bridges; taking $E_c = 40$ GPa for the calculated deflections. The effect of the R.C. parapets and the asphalt layer is neglected for the calculated deflections. While the difference of the average ratio to one can be explained by the participation to the inertia of secondary elements, and by the value of the modulus of elasticity, the dispersion is large. In order to understand and reduce this dispersion, the following three sub-samples were examined:

- bridges with RC parapets (22 bridges), $R_m = 0.82$, $\sigma = 0.095$;
- bridges with a box-girder of variable depth (10 bridges), $R_m = 1.03$, $\sigma = 0.11$;
- bridges with an asphalt layer (33 bridges), $R_m = 0.93$, $\sigma = 0.15$.

The smaller dispersions of these three sub-samples shows the influence of certain characteristics of the bridge on its behaviour, as it clearly appears in Fig. 3.

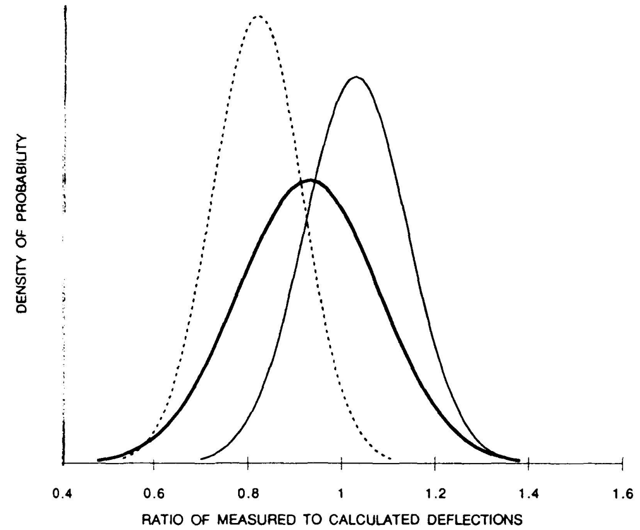


Fig. 3: Normal probability distribution curve for the ratio of measured and calculated deflections for: bridges tested with an asphalt layer; the box-girder bridges of variable depth; all the bridges with massive R.C. parapets (E_c = 40 GPa)

3.2 Analysis of data

The results of the load tests in the sample of 66 bridges were successively corrected using the reciprocal of the R_m determined for each factor contributing to the moment of inertia. The increase in stiffness due to the presence of normal and prestressed reinforcement was estimated to 6%.

After these corrections, the results of the calculations led to an average ratio of measured to calculated deflections of 1.19, with a modulus of elasticity of concrete equal to 40 GPa. In order to obtain R_m = 1.0 for this sample, the modulus of elasticity of the concrete should be taken to 33.5 GPa, which is much closer to actually measured values. Fig. 4 shows the histogram of the ratio of measured to calculated deflections and the Gauss distribution after correction of the influence of the parapets, of the asphalt layer, of the reinforcement and of the cross-section. Table 2 summarizes the participation to inertia of asphalt layer, R.C. parapets and reinforcement.

The asphalt surface increases the effective inertia of the superstructure. This participation depends strongly on the surrounding temperature and on the cross-section of the bridge. Fig. 5 shows the normal probability distribution for the ratio of measured to calculated deflections for bridges tested with and without asphalt.

Element	Asphalt layer	R.C. parapets	Reinforcement
Participation to inertia	6%	24%	6% (estimated)

Table 2: Participation to inertia of asphalt layer, R.C. parapets and reinforcement

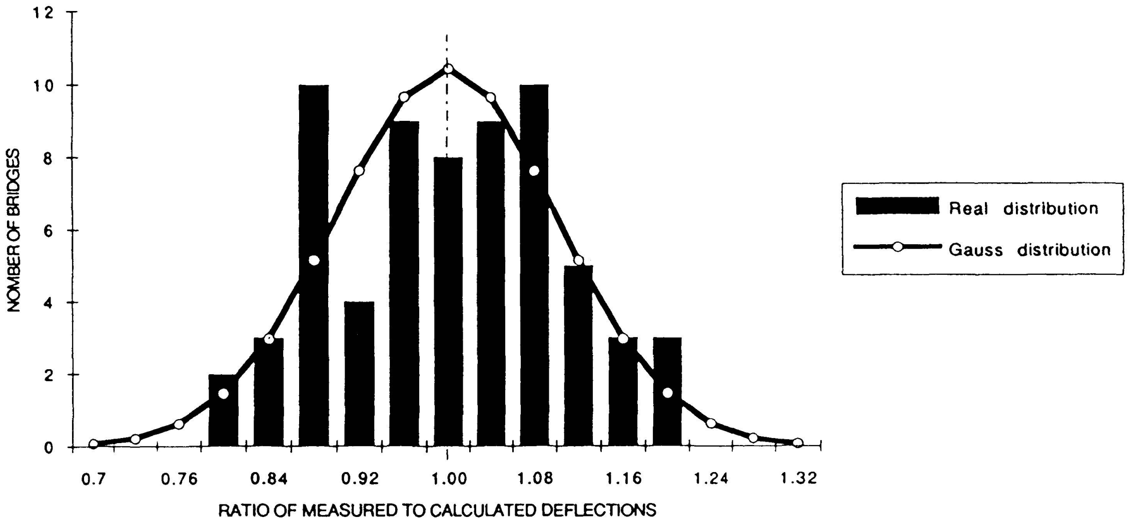


Fig. 4: Histogram of distribution of ratio of measured to calculated deflections for 66 bridges after correction (E_c = 33.5 GPa)

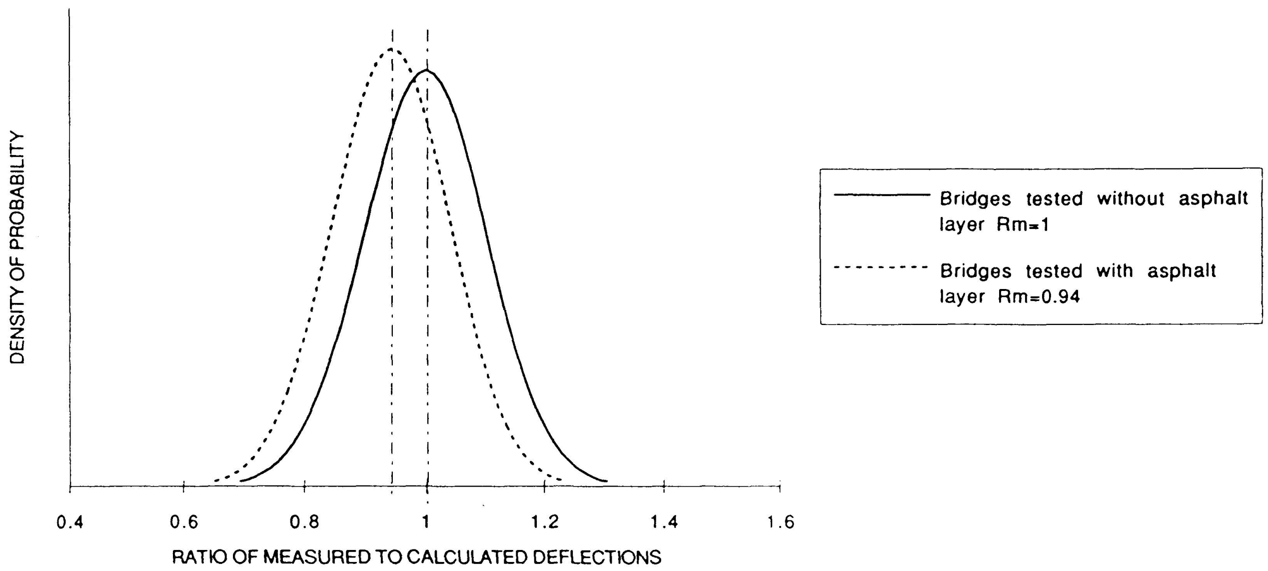


Fig. 5: Normal probability distribution curve for the ratio of measured to calculated deflections for bridges tested with and without asphalt (E_c = 33.5 GPa)

The parapets increase the effective inertia of the superstructure. The amount of this participation depends on the type of parapets, on the connection between the parapets and the superstructure and on the cross-section of the bridge. Fig. 6 shows the normal probability distribution for the ratio of measured to calculated deflections for bridges tested with and without reinforced concrete parapets. It should be noted that the effect of parapets is neglected for the calculated deflections.

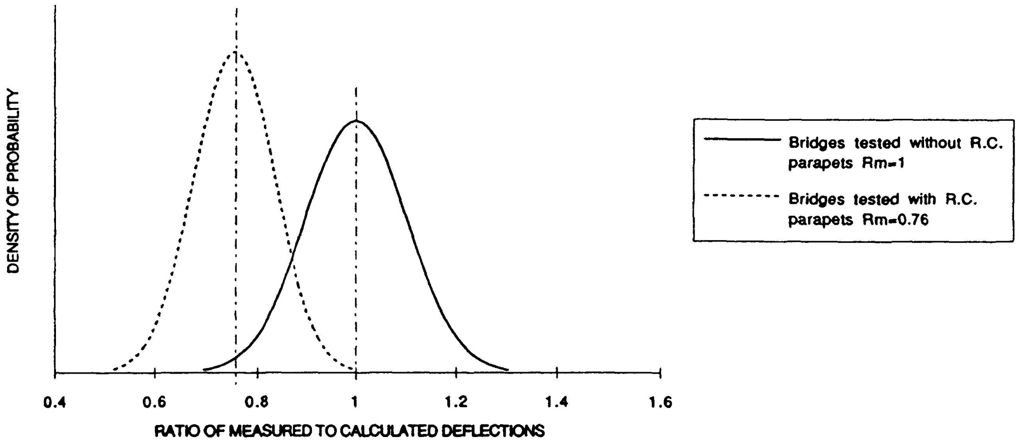


Fig. 6: Normal probability distribution curve for the ratio of measured to calculated deflections for bridges tested with and without R.C. parapets (E_c = 33.5 GPa)

Box-girder bridges systematically exhibit an effective modulus of elasticity lower than average, especially box-girders of variable depth. On the other hand, slab bridges and bridges of an open cross-section systematically show an effective modulus of elasticity greater than the average modulus. Table 3 shows the effective modulus of elasticity for the bridges with various cross-sections.

Type of cross-section	Effective modulus of elasticity [GPa]
Box-girder of constant depth	32
Box-girder of variable depth	31
Slab beam	37
Open (beams connected by concrete deck)	35

Table 3: Effective modulus of elasticity for the bridges of different types of cross section

A possible explanation for these variations lies in the fact that the method of construction is strongly dependent on the type of cross-section. Fig.7 shows the normal probability distribution for the ratio of measured to calculated deflections for box-girder bridges with a constant depth constructed on fixed scaffolding, for box-girder bridges with a variable depth constructed by the balanced cantilever method and for box-girder bridges with a constant depth constructed by incremental launching. In the same manner slab beams and open cross-section bridges are shown in the same figure.

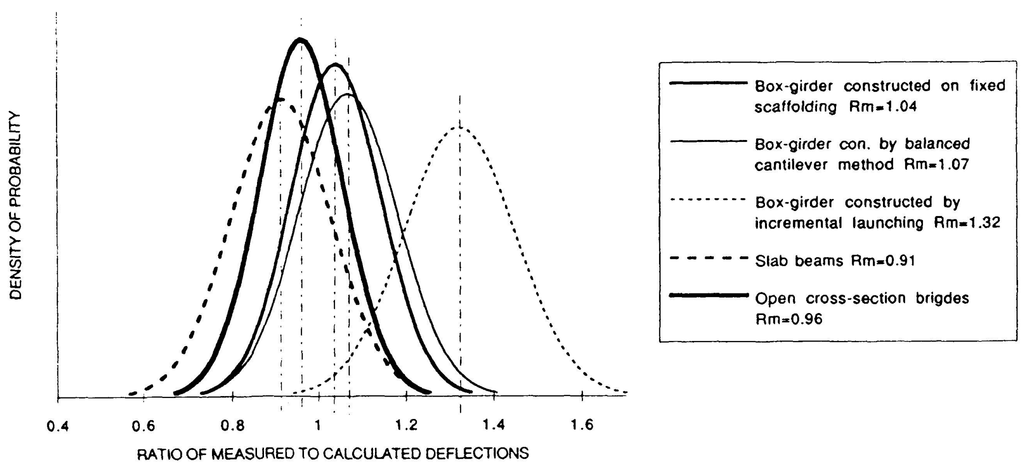


Fig. 7: Normal probability distribution curve for the ratio of measured to calculated deflections for bridges of different type of cross-sections and different construction methods (E_c = 33.5 GPa)

3.3 Affinity between the measured and calculated deflections

The ratio between the measured and calculated deflections in the loaded spans is systematically larger than those in the adjacent spans. If we make the calculated deflections correspond to those measured in the loaded spans at mid-span ($R_m = 1$), R_m becomes 0.84 in the adjacent spans (see Fig. 8). This systematic difference in stiffness between adjacent spans could be explained by cracking of the loaded span and/or a lack of continuity on supports. Another factor can be a reduction of the modulus of elasticity in the loaded span due to the level of stress in the concrete.

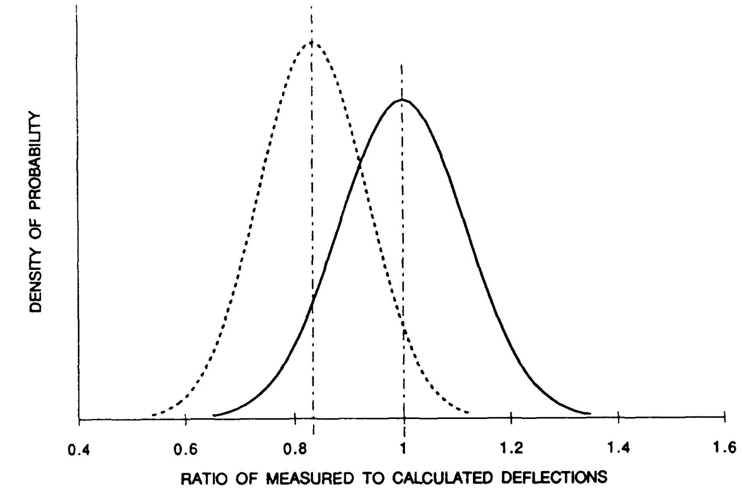


Fig. 8: Normal probability distribution curve for the ratio of measured to calculated deflections for the loaded central span and for the adjacent span

4. ULTIMATE LOAD FROM LOAD TEST RESULTS

The load-deflection curve for a prestressed concrete bridge can be approached by a tri-linear relationship as shown in figure 9, in which we have assumed that 80% of the permanent load is balanced by the effect of prestressing, a permanent load of 20 kN/m^2 and a live load of 5 kN/m^2. Considering these different values and assuming a global safety factor of 1.7, the ultimate design load is 1.7 (20+5) = 42.5 kN/m^2. Because the load test is an acceptance test the upper limit of loading is generally about 5 kN/m^2, which normally should not lead to cracking of the bridge. It seems that the extrapolation of the ultimate load based on the results of a single load test is senseless. However, a good correlation between measured and calculated deflections indicates a satisfactory structural behaviour, provided the deflections are reversibles and the requirements of ductility are fulfilled.

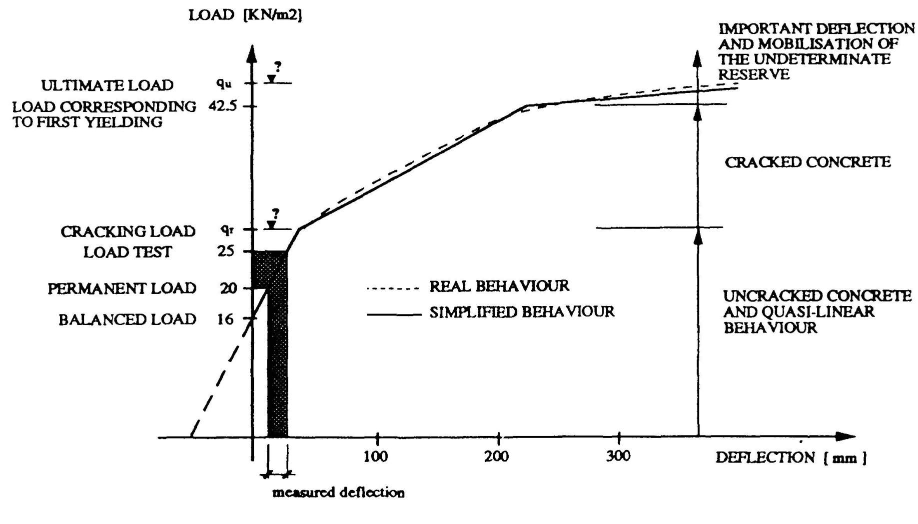


Fig. 9: Load-deflection curve of a prestressed concrete bridge (order of magnitude)

5. CONCLUSIONS

The load test is a real chance and a challenge to observe and interpret the actual behaviour of a structure under load on a 1:1 scale. A frequent correlation between an unsatisfactory behaviour during a load test and the long-term behaviour of a bridge regarding a non-stabilization of cracking and of sagging has been observed.

The contribution to the rigidity of the parapets, the asphalt layer and the reinforcement has to be taken into account for calculated deflections. The modulus of elasticity has to be experimentally determined and not only estimated according to the compressive strength of the concrete.

Although our research concerning 200 load tests is not completely finished, some important preliminary conclusions can be drawn:

Box-girder bridges exhibit an effective modulus of elasticity lower than the average. On the other hand slab bridges and bridges of open cross-section show an effective modulus of elasticity greater than the average modulus.

The method of construction influences the stiffness of the bridge; box-girder bridges with a constant depth constructed by incremental launching are greatly less rigid than those constructed on fixed scaffolding. This can be explained by micro-cracking during the execution.

Even if the calculated deflections are corrected to correspond to those measured at mid-span of the loaded span, the affinity between the measured and calculated deflected shapes is not perfect. A systematic difference in stiffness between loaded spans and adjacent spans is observed. This can be explained by cracking of the loaded span and/or a lack of continuity over the supports.

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Remaining Carrying Capacity of Temporary Steel Bridges
Capacité restante des ponts provisoires en acier
Resttragfähigkeit provisorischer Stahlbrücken

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SUMMARY
The paper discusses the problems of fatigue life-time and failure of special steel elements adopted as structural components of temporary steel bridges. Attention is paid to properties and variations of loading effects occuring in actual conditions of bridge structures studied. The main results of extensive fatigue tests of steel elements studied are described and discussed. The presentation of a new cumulative conception for the calculation of the fatigue life-time is submitted.

RÉSUMÉ
Cet article analyse des problèmes de fatigue et de la rupture des éléments d'acier spéciaux qui sont utilisés dans les ponts provisoires en acier. L'attention est portée aux qualités et aux changements des charges se trouvant sur les ponts réels. Dans cet article on décrit et analyse des résultats principaux des vastes essais de fatigue. On y présente une nouvelle conception cumulative du calcul de la durabilité des constructions provisoires en acier.

ZUSAMMENFASSUNG
Der Beitrag diskutiert die Probleme der Ermüdungslebensdauer und des Kollapses von speziellen Stahlelementen, die als Komponenten provisorischer Stahlbrücken verwendet werden. Besondere Aufmerksamkeit wird den Eigenschaften und Variationen der Belastungseinflüsse geschenkt, die unter tatsächlichen Bedingungen vorhanden sind. Die Hauptresultate von ausführlichen Ermüdungsversuchen der Stahlelemente werden dargelegt. Ein neues kumulatives Konzept für die Berechnung der Ermüdungslebensdauer der provisorischen Stahlbrücken wird präsentiert.

1. INTRODUCTION

Nearly in every country we can meet special steel elements which are intended to be used as components of temporary bridge structures during the reconstruction of existing bridges or for special transport purposes. After some short time of service they are disassembled and stored for another utilization. After certain time periods it is necessary to decide what is their remaining carrying capacity and whether they can be used again with sufficient degree of reliability. Similar problems can arise in the case of such steel elements which were produced with some degree of defects as notches, small cracks, geometrical imperfections, etc. In our paper we deal with the results obtained from the tests and analyses of assembled elements of czecho-slovak temporary steel bridges. Two asembly bridge systems were tested. In the first system one from the tested elements was previously used in the temporary bridge during the reconstruction of the tram bridge for the period of two years. Others of them did not fully answer to conditions of safety due to initial production defects. The second system was later developed with an intention to replace the first system. Due to results of our tests its details and a production technology were modified and improved.

2. PROPERTIES OF LOADING IN ACTUAL AND LABORATORY CONDITIONS

Dynamic fatigue loading of bridges is caused in large degree by variable combined static-dynamic actions from different vehicles moving through the bridge. Some dynamic loading can be caused by natural effects as are strong winds and earthquakes. Then the dynamic loading process of bridges is random stationary or non-stationary. As a base for the analysis of dynamic loading properties we can use time records of accelerations, velocities, deflections, strains, etc. Dynamic measurements on structures and in laboratory need a wide assortment of different devices such as pickups, amplifiers, recorders, computers with A/D and D/A convertors, appropriate softwares, etc.

A random stochastic process as a continuous random function of a non-random variable $t \in (-\infty, \infty)$ may be written as a set $X(t)$, where $X(t) = x_1(t), x_2(t), \ldots x_i(t), \ldots$, in which $x_i(t)$ $(i=1,2, \ldots)$ are realizations of random process. Having realizations $x_i(t)$ we can calculate for each of them all necessary characteristics as distribution functions, probability densities, correlation functions, power spectral densities, transfer functions, coherence functions, etc., due to the purpose of analysis [5], [6]. Frequency distribution in power spectral density is influenced by properties of the source loading, e.g. from the motion of vehicles, and by dynamic properties of the bridge structure alone. In Fig. 1 we can see acceleration records in the midspan of tram bridge points 10, $\overline{10}$ and chosen power spectral densities. When passing from acceleration to velocity or deflection spectrum, the peaks in upper frequency region are falling down as can be seen in Fig. 1-A. This record was obtained from the motion of the sole heavy truck through the bridge. The fluent motion of heavy vehicles can cause the vibrations which are nearly harmonic with narrow frequency band [7]. When using piesoelectric measuring system with primary acceleration measurements it is convinient to apply modified integration according to algorithms described in [6]. The obtained frequency distribution is important for the fatigue analysis when following the expected number

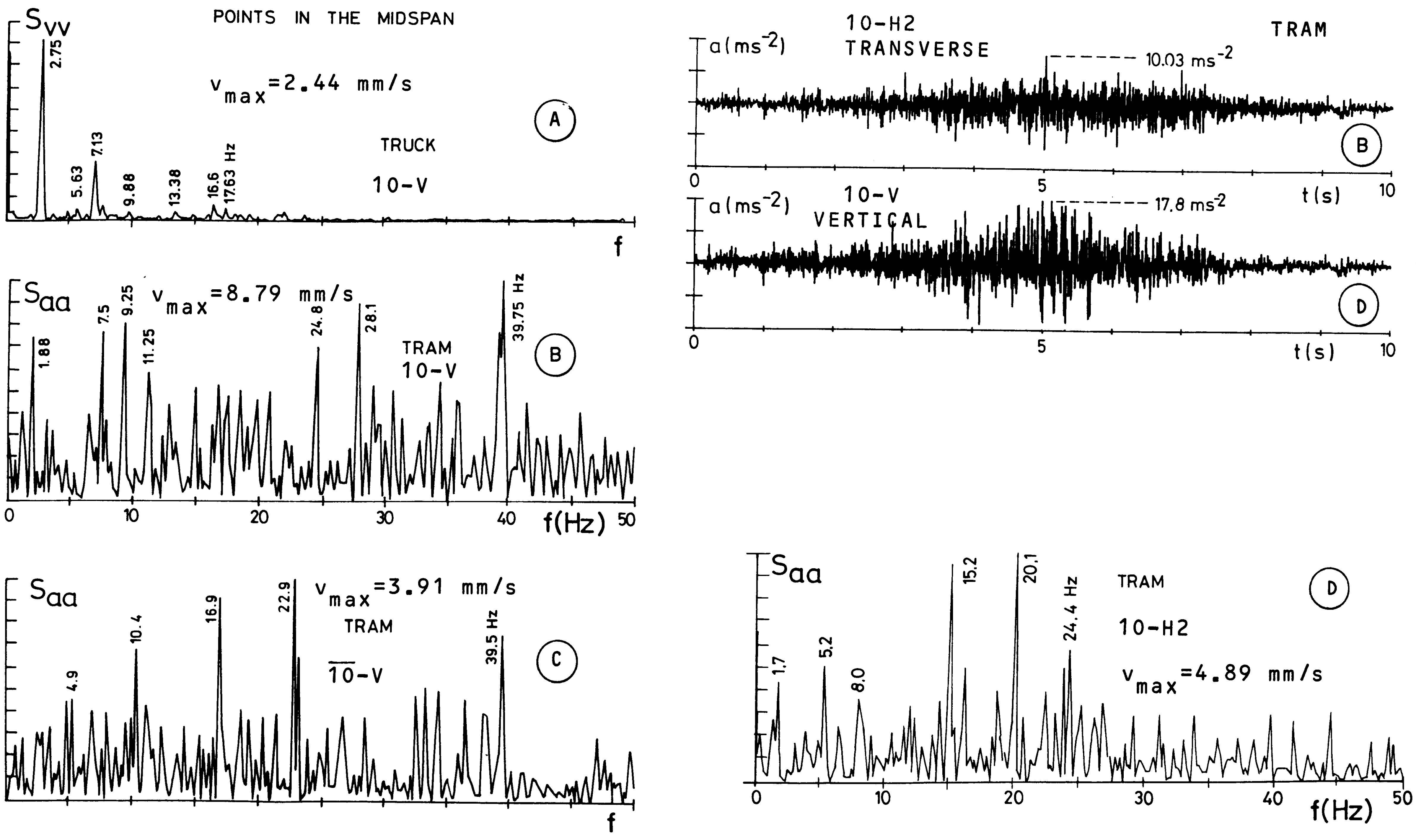

Fig. 1. Vibration of a tram bridge. Time records and power spectral densities (S_{aa} determined from acceleration records, S_{vv} from velocity records)

of cycles and frequency components with larger amplitudes. Dynamic measurements give usually informations only about dynamic component of deformation. Then for laboratory fatigue tests we use the combination of measured and calculated values. In laboratory conditions we can apply actual loading processes directly through the simulation of a chosen loading, or we can apply one level or multi-level harmonic loading with the prescribed spectral distribution [1], [2], [8], [9], [11].

3. LABORATORY TESTS AND THEIR RESULTS

Experimental verification of the fatigue strength and the life-time of steel bridges is not usually available on full-scale structures. Technical and time factors are decisive. The technical and spatial facilities are also limiting factors for the fatigue tests of full-scale bridge structures in special laboratories. Therefore, the fatigue strength and life-time of steel bridges is often investigated on their models. The transfer of conclusions and results from model tests into actual structures can be problematic. This is, however, very problematic in the cases of determination of total and remaining fatigue life-time of already exploited bridge structures with certain constructional influences and production-technological defects or cracks.

In Czecho-Slovakia the special assembly bridge systems for temporary repeated exploitation have been developed and produced. Due to originality of technical solution, valid standards and former scientific knowledge have not enabled to determine their total and remainig fatigue life-time. The connecting bolts and areas of individual assembly elements have been especially problematic from the view of fatigue strength and failure. Therefore, the experimental verification of these phenomena was realized at our Institute [1],[2],[10].

Our laboratory enables to realize static-dynamic tests of structures and their elements with constructional length up to 18 m. It was not enough for the test of the whole bridge system, therefore, only individual assembly elements were tested in special configuration (Fig. 2). The aim of this was to simulate the design loading of investigated connecting bolts and areas near connections.

In the first stage the static succesive and repeated loading of system was realized by one or two forces until $P = P_u$, where P_u is a theoretical ultimate static load, the effect of which corresponds to the effect of design loading on full scale bridge structures. Deflections v and strains ε were measured in chosen points of flanges, webs and stiffeners. Then fatigue tests were realized. with loading $P(t) = P_0+P_1 \sin(2\pi ft)$, $P_0=(P_{max}+P_{min})/2$, $f = 5$ Hz, $P_1=(P_{max}-P_{min})/2$.

The fatigue tests continued in stages with interuption after every 50 000 or 100 000 cycles, when the systems were controlled both visually and through the computer. Controlled values were deflections and strains for load levels P=0; P_{min} and P_{max}. The differences in strains indicated beginning of cracks and their position. After appearance of cracks their succesive increasing was recorded with investigation of their influence on the system behaviour. The tests continued until the failure of systems.

From system 1 there were tested five assembly elements. Every element was tested twice in cantilever composition. It means following the behaviour of both its ends in position a or b in Table 1. The element, previously used in the tram bridge, had already visible fatigue cracks in the place of connections. The others were chosen from stocks. From fatigue tests six different types of cracks were ascertained - A up to F, as can be seen in Fig. 2. The decisive cracks and total numbers of cycles N for individual elements and tests are in Table 1.

Table 1. The results of fatigue tests of system 1.

Indication			MS0	MS1	MS2	MS3	MS4
Position	a	cracks	C	A,E	D	F	B
		N	1 150 000	1 160 000	1 170 000	585 000	1 010 000
	b	cracks	A	D	F	F	F
		N	680 000	1 820 000	750 000	260 000	565 000

For system 2 previously four elements were tested with interchanging of their ends. All these tests finished with fatigue cracks in places of tension connections of types A and B from Fig. 3. On the base of obtained results the modification of technology was recommended. Then next four tests with another four elements were realized. Here arose new types of fatigue cracks C, D and E, which were decisive for the failure of the whole system (Fig. 3). Total numbers of cycles N and decisive cracks are in Table 2.

Table 2. The results of fatigue tests of system 2.

Indication			MS1	MS2	MS3	MS4
Element	I	cracks	-	-	C	C
		N	-	-	2 223 000	1 850 000
	II	cracks	C	D	-	-
		N	3 380 000	3 690 000	-	-

The knowledge and results of presented laboratory tests were used for the determination of the total and remaining fatigue life-time of verified temporary steel bridge systems. Owing to the small number of tests and their character, the obtained results could not be, however, sufficient and competent theoretical accesses. It means to apply them together with the standards data in the fatigue analysis.

4. FATIGUE STRENGTH AND LIFE-TIME

The antecedent standards for the judgement of fatigue strength and life-time of bridge structures have considered a harmonic loading with constant amplitude. Also experimental fatigue verification has been mostly realized with this type of loading. But, as we have mentioned previously, actual steel bridges are subjected to variable-amplitude loading.

Much activity has already been concentrated on the development of a fatigue design method for service variable-amplitude loading. The result of it is a definition of the effective constant-amplitude stress, which allows to use the constant-amplitude relationships for the variable-amplitude load conditions. Palmgren-Mi-

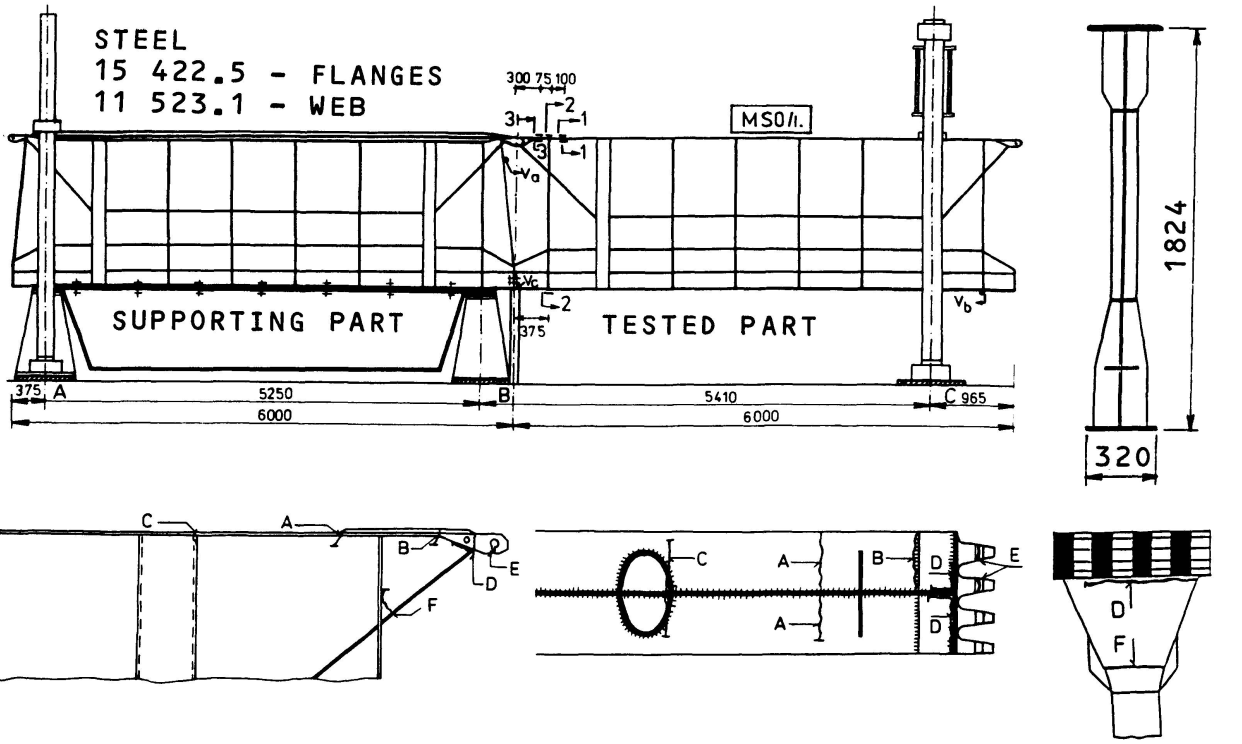

Fig. 2. Static and geometric scheme of test system 1. Positions of fatigue cracks.

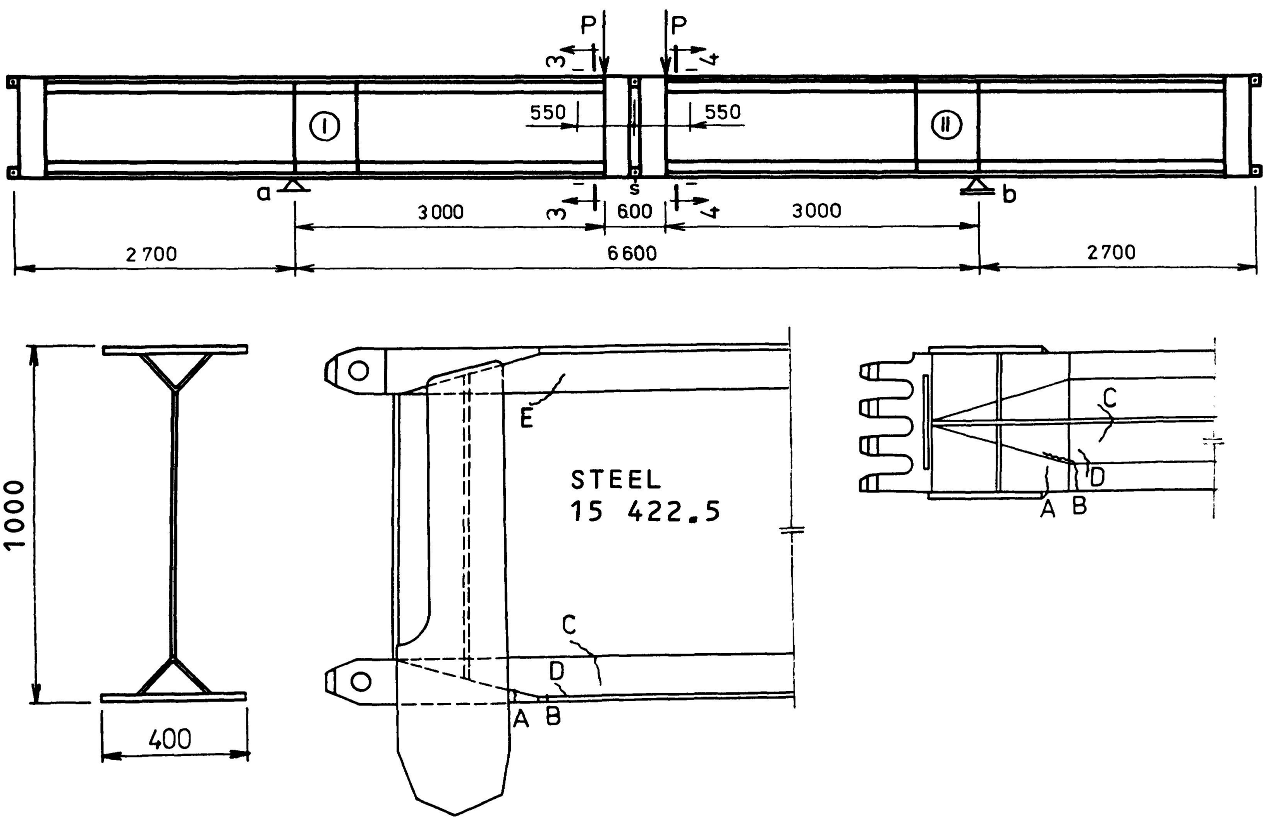

Fig. 3. Static and geometric scheme of test system 2. Positions of fatigue cracks.

ner's linear sumation is usually used for accounting the cumulation of fatigue damage caused by variable-amplitude loading. It is generally known that Palmgren-Miner's cumulative rule is not sufficiently accurate. In our Institute the extensive experimental program has been realized for verification of some cumulative assumptions and fatigue design methods [3],[4],[8],[9]. The result of this program is a new conception for the determination of the fatigue life-time.

In the case of constant harmonic loading the fatigue life-time can be determined from experimental or standard fatigue curve concerning the most exposed place of structure. The stochastic stationary loading can be expressed in simplified way using loading spectra with different double stress amplitude $\Delta\sigma_i$ and answering number of cycles n_i. Considering constant harmonic loading with $\Delta\sigma_{max}$ we can state from responsible fatigue curve the minimal number of cycles N_{min}. It means that according to the prevailing influence of variable amplitude loading the total number of cycles N should be higher than N_{min}.

We have used our obtained results like a basis for the new determination of fatigue life-time using N_{min}, $\Delta\sigma_{max}$ and aggressivity of loading $\bar{r}$:

$$\bar{r} = \sum_{i=1}^{s} \frac{\Delta\sigma_i}{\Delta\sigma_{max}} \frac{n_{i(1)}}{n_{(1)}} \alpha_i \leqq 1 \quad , \tag{1}$$

where σ_{max} is maximum double stress amplitude, $n_{i(1)}$ is the number of cycles for individual loading levels and $n_{(1)}$ is total number of cycles in one block loading.

For constant harmonic loading $\bar{r} = 1$. Coefficient $\alpha_i = 1$ for $\Delta\sigma_i > \Delta\sigma_0$; and $\alpha_i < 1$ for $\Delta\sigma_i < \Delta\sigma_0$. $\Delta\sigma_0$ is defined as the limit fatigue stress. Due to obtained test results when $\Delta\sigma_i < \Delta\sigma_0$ it answers very well $\alpha_i = 0.8$. Therefore we recommend for the determination of total number of cycles N:

$$N = N_A = N_{min}/\bar{r}^k \quad , \tag{2}$$

where

$$k = (1 + \bar{r}^2)\,(3.25 - \bar{r}) \tag{3}$$

and the total life-time T_A

$$T_A = N_A/n_{(1)} \quad . \tag{4}$$

When calculating the life-time of the lower chord of the railway steel bridge [2] we can compare the results of different fatigue conceptions [8], [9], where

- authors: T_A = 10.506 years,
- Palmgren-Miner: T_{PM} = 9.721 years,
- SVUM: T_{SM} = 4.347 years,
- ČSN 73 6205: $T_{ČSN}$ = 7.130 years.

It can be seen that authors' fatigue life-time is larger than that which is supposed due to valid standard demands. Moreover, from the analysis and comparison of results it follows that the theoretical life-time, described by number of cycles N_A is more close to experimental results N than the theoretical ones N_{PM}

or N_{SM} determined according hypothesis Palmgren-Miner and SVUM.

5. CONCLUSION

The total and remaining fatigue life-time is very relevant for the reliable exploitation of assembled temporary steel bridges. The fatigue of these structures depends on many factors as are in particular actual service loading process, material characteristics, geometrical parameters, constructional solution, production-technological effects and defects. All these factors are, of course, random variables. Therefore, the exact judgement of fatigue strength and the determination of the life-time for such structures is problematic also nowadays. In cases of original pretentious steel bridges, as those presented in the paper, the experimental verification is inevitable. The appropriate simulation of actual conditions and characteristics is, however, very important for the reliable results. Our proposed approach to the total and remaining fatigue life-time determination concerns variable multi--level loading. For general cases it can be determined using the value of minimum life-time in the case of maximum loading together with the application of aggression effect of the whole loading spectrum. The fatigue life-time is then appropriately increased. The new conception is accounting for the influence of general loading process. It is more exacting than the application of usual linear cumulation hypotheses.

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Load Tests on Acute-Angled Motorway Bridge
Essai de charge sur un pont d'autoroute biais
Belastungsversuch an einer spitzwinkeligen Autobahnbrücke

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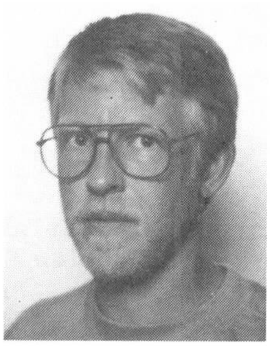

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SUMMARY
A full-scale load test in the serviceability limit state has been carried out on a 6-year-old motorway bridge. The test was made with the purpose of examining the structural behaviour of the bridge under heavy load. The load on the bridge came from two heavy vehicles, and strain and deflection measurements were made on the bridge deck and the edge beam. The test results have been compared with the results from a FEM analysis.

RÉSUMÉ
Un essai de contrainte, grandeur nature, à la limite de charge, a été réalisé sur un pont d'autoroute construit depuis 6 ans. Cet essai a été effectué dans le but d'examiner le comportement structurel du pont sous l'effet d'une lourde charge. Les charges exercées par deux véhicules lourds ont permis de mesurer les augmentations relatives de longueur ainsi que les fléchissements vers le bas du tablier du pont et de la travée latérale. Les résultats des essais ont été comparés avec ceux provenant d'un modèle d'éléments finis.

ZUSAMMENFASSUNG
An einer sechs Jahre alten Autobahnbrücke wurden Belastungsversuche im Bereich der Gebrauchslast einschl. Sicherheitsbelastung durchgeführt. Damit sollte ermittelt werden, wie die Brücke strukturell auf schwere Belastungen reagiert. Nach Plazierung zweier Schwerlastfahrzeuge wurden die Dehnung und die Durchbiegung jeweils an der Deckenplatte und am Randbalken gemessen. Die Ergebnisse des Versuches wurden mit denen aus einem FEM-Modell verglichen.

1. INTRODUCTION

The paper describes a load test performed on a six year old motorway bridge. Originally the bridge was constructed as a part of a large ferry terminal project at the Great Belt which separates the eastern and western part of Denmark. The terminal was a part of a project in which both railway and roadtraffic were to be shipped from the same terminal instead of from two separate terminals several kilometers apart, as was the case until then. However, as a decision was made to establish a fixed link across the Great Belt, the terminal plans were given up during the construction of the bridge, and the bridge therefore never served its real purpose.

As it turned out impossible to re-use the bridge as a part of the on-shore constructions for the Great Belt crossing, the bridge was demolished just after the load testing had ended.

The bridge was constructed using high quality materials, and thus, being practically undeteriorated, it offered a unique opportunity to perform a variety of tests. These could e.g. through a demonstration of the real life behaviour indicate possible needs for alterations in the current design practice and the basic assumptions concerning the concrete properties.

Only tests in the serviceability limit state with live loads were considered, due to limitations on time and budgets.

2. DESCRIPTION OF THE BRIDGE

The bridge consisted of a 2-span post-tensioned plate, simply supported on retaining walls and columns. The angle of intersection was only about 18 degrees, resulting in free edges spanning up to 40 metres. Prior to the load testing, the concrete properties were measured by testing drilled-out cores. The average strength was determined to be 40 MPa and the modulus of elasticity approx. 39.500 MPa.

The bridge is shown in fig 1.

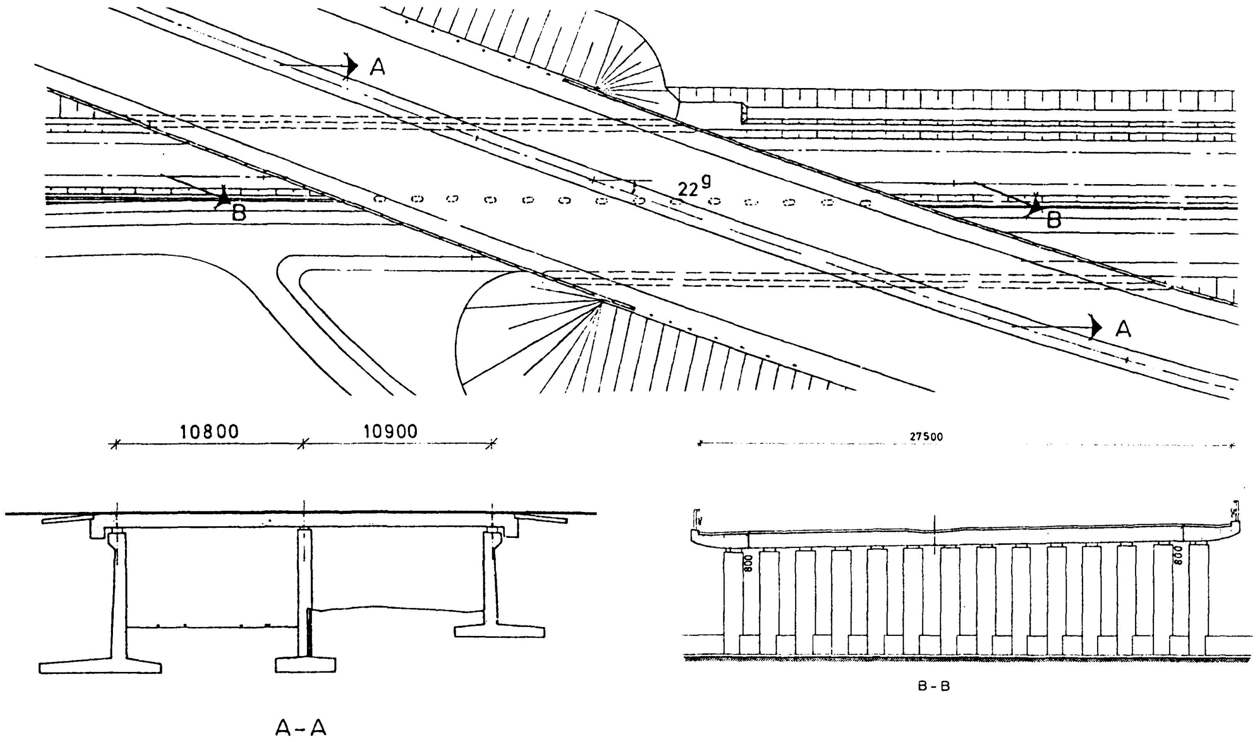


Fig 1 The bridge

3. DESCRIPTION OF THE TESTS

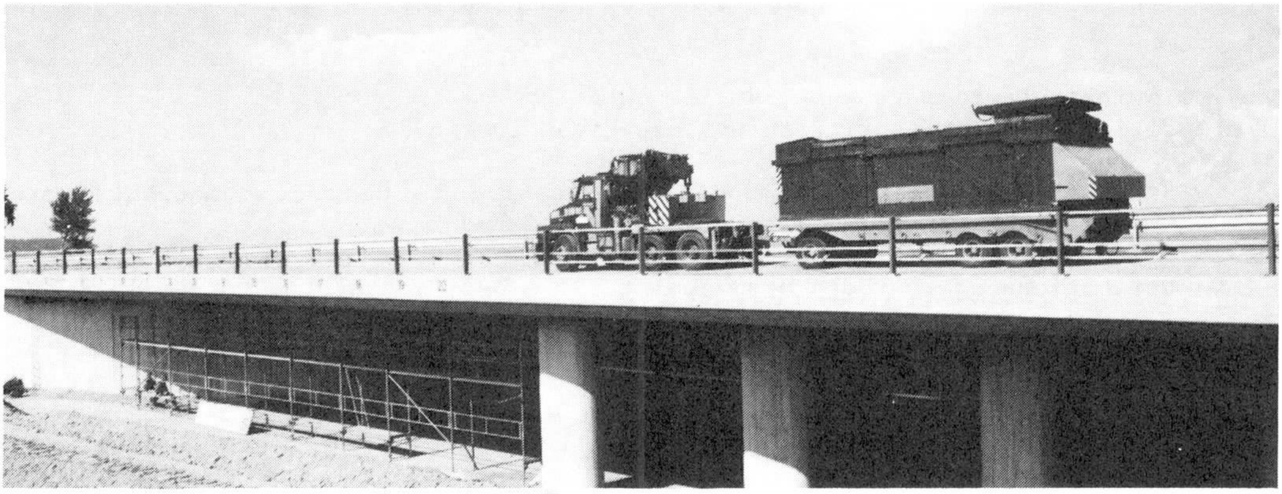
The tests were carried out over two full days at the end of July 1991. The weather was dry and sunny, with temperatures of 20-23°C. The load was provided by two heavy vehicles from the Danish Road Directorate (DRD) and the Royal Danish Army, respectively. The load characteristics of the vehicles are given below, in table 1.

	front-axle-load tons	Rear-axle-load tons	Total load tons
The Danish Road Directorate Loaded trailer	22.0	2 x 22.05	66.1
The Danish Road Directorate Volvo truck	6.0	2 x 8.85	23.7
The Royal Danish Army Tank on tank-transporter	-	2 x 31.4 (2x23.7)*	62.8 (47.4)*
The Royal Danish Army MAN Truck	6.6 (7.7)*	2 x 6.0 (2x13.1)*	18.6 (34.0)*

*) With tank in normal driving position

Table 1 Load characteristics of the heavy vehicles.

The vehicle from the Danish Road Directorate, was a heavy Volvo truck with a trailer loaded with steel plates. The vehicle from the Royal Danish Army was a Centurion tank on a Scrammel tank transporter pulled by a MAN truck. The vehicles are shown in fig 2.



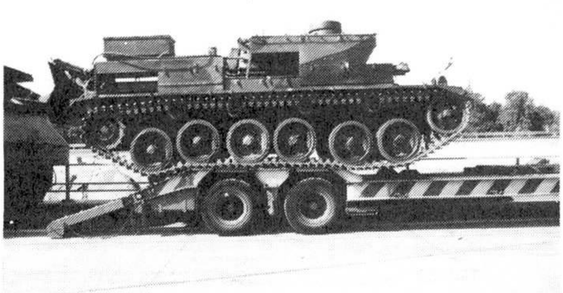


Fig 2 The heavy vehicles. The truck with steel loaded trailer from the Danish Road Directorate (above) and the tank from the Royal Danish Army in position above the rear axles of the transporter (left).

By placing the tank directly above the rear axles of the tank transporter, a load on these two axles of about 63 tons was obtained. Further, a relatively concentrated load of more than 100 tons could be obtained by placing the two vehicles in a back-to-back in-line configuration.

The axle loads were measured with an accuracy of ± 50 kg and the vehicles could be placed on the deck within an accuracy of ± 0.5 m.

13 tests were carried out with different combinations of the loads: 5 tests with the DRD-truck alone, 6 tests with both vehicles positioned in-line next to the edge beam and 2 tests in a side-by-side configuration. Furthermore, 6 reference tests were performed without loading. These reference tests were performed morning, noon and evening, and were performed for calibration purposes, in order to be able to compensate for temperature effects.

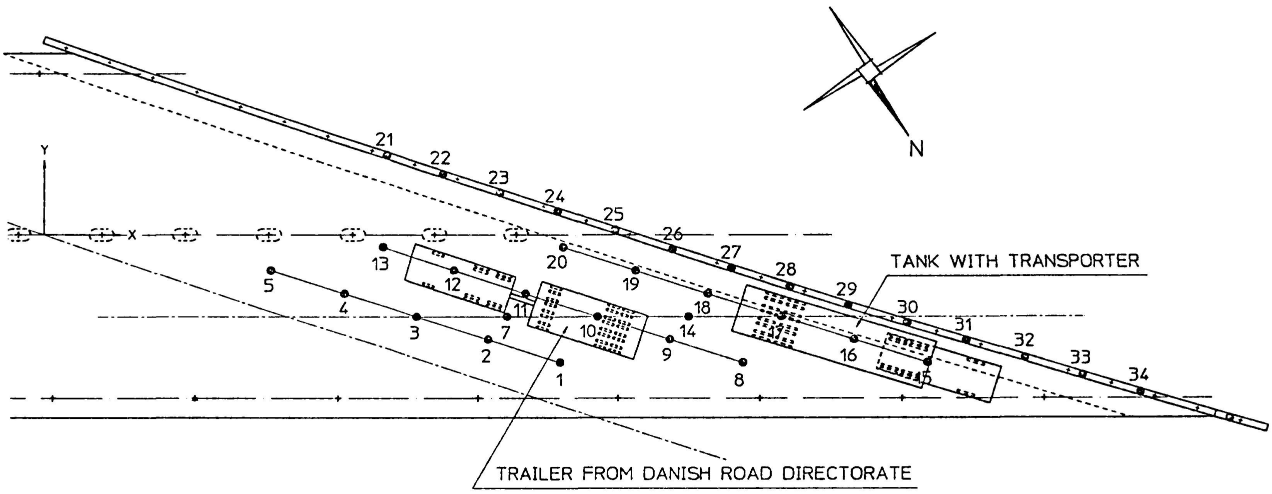


Fig 3 Typical set-up of the heavy vehicles (Test no 18).

The following measurements were carried out:
(The position of the measuring points are given in fig 3)

- Strains at 19 points at the lower side of the deck. 3 components were measured at each point. (points 1 - 19)
- Deflections of the edge beam (points 21 - 34), and at the points 15 - 20
- Strains at 14 points at the upper side of the edge beam. 1 component (the longitudinal one) was measured at each point. (points 21 - 34)
- Strains of unloaded parts of the construction (for reference purposes).

The strains were measured by DEMEC mechanical strain-gauges with an acurracy of 10^{-5}. The deflections were measured using a GEODIMETER theodolite.

Besides for calibration purposes, the unloaded tests served for the detection and quantification of temperature effects. During testing the upper side of the bridge-deck was subject to a temperature rise of about 10°, which represented potential temperature induced stains of the same order of magnitude as the maximum load-induced strains. The temperature rise underneath the deck, on the other hand, was only about 2°. By comparing morning and evening reference tests, it could be concluded that the bridge deck was subject to a slight compression in the x-direction, and a corresponding slight expansion in the y-direction. The magnitude of these strains was only 10 - 15 x 10^{-6}, which is considerably smaller than the maximun bending strains, thus leading to the conclusion, that the temperature rise leads to an increase in the internal stress level, rather than to stains. Consequently, considering the procedure of calibrating against unloaded tests, scattered througout the day, the temperature effects were neglected.

The reproduceability of the strain measurements was determined by means of a special test bar. Fluctuations within $\pm$ 5 x 10^{-6} were observed. The accuracy of the deflection-measurements was measured to $\pm$ 0.75 mm.

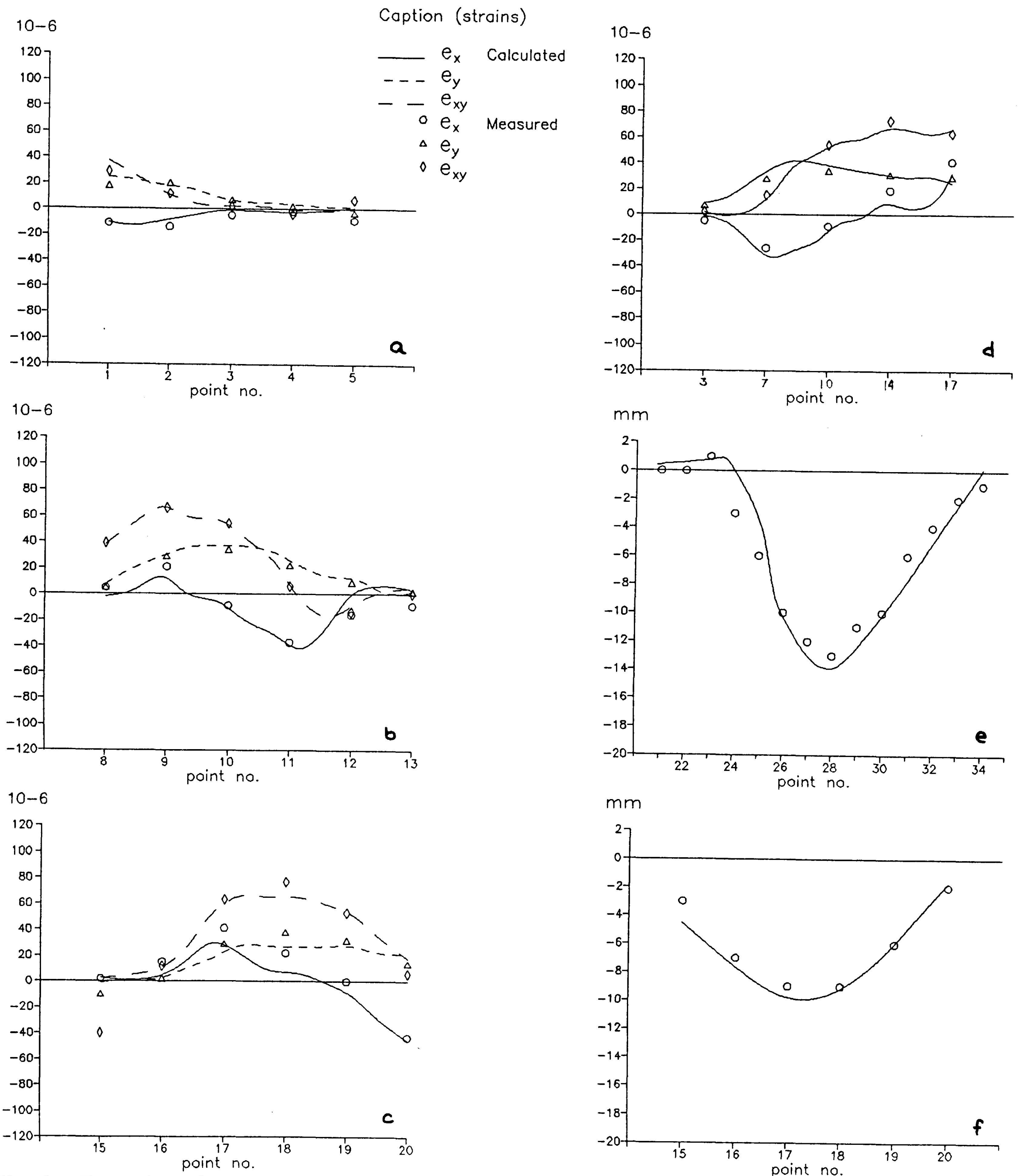


Fig 4: Test Results. Measured (markers) and calculated (lines) results. The x-axis indicates the number of the measuring point (see fig. 3). (a - d: strains, e and f: deflections).

4. RESULTS

The maximum edge beam deflection was measured to approx. 10 mm with the load consisting of the the steel plate loaded DRD-truck only, and to approx. 17 mm when both vehicles were present.

The corresponding strains were in the range of -50×10^{-6} to 60×10^{-6} and -60×10^{-6} to 130×10^{-6}, respectively.

The longitudinal strains measured at the upper side of the edge beam were in the range of -100×10^{-6} to 80×10^{-6}.

No cracks were detected, even though the loads by far exeeded the level of the serviceability limit state, and actually were close to the ULS design loads.

The results obtained from a selected test are shown in fig 4. Note that all strains are 'true' strains, measured in three directions: e_x parallel to the retaining walls (= the x-direction), e_y shifted an angle of -90 degrees with respect to the x-direction and e_{xy} shifted -45 degrees.

The test results have been compared to theoretical results provided through an FEM-analysis, in which the bridge was represented by 300 triangular plate elements in each of the unloaded quadrants of the bridge, and by 600 elements in the loaded section. The plate edges, i.e. the edge beam and the load distributing beam at the bearing line, were modelled by additional beam elements. The curved shape of the plate at the free edge was modelled using wedge shaped plate elements.

5. CONCLUSIONS

Fig 4 shows an exellent agreement between the measured and the calculated results, thus indicating that the behaviour of bridges, even with very acute intersection angles, may be successfully predicted by the use of standard FEM modelling. The good agreement also confirms, that the bridge was fully intact, without sign of cracking.

Finally, the tests demonstrate that the mechanical strain gauge technique, which was used, is a well suited tool, and provides a sufficient degree of accuracy, for the kind of load testing described in this paper.

6. AKNOWLEDGEMENTS

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Field Testing of Existing Bridge on Remaining Load Capacity
Essais de charge et capacité restante d'un pont existant
Feldversuch zur Resttragfähigkeit einer bestehenden Brücke

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SUMMARY
This study is devoted to defining the load sharing of secondary members, such as lateral bracings, sway bracings and concrete walls compared with the loading test results of an existing steel bridge. Furthermore, it is examined how to evaluate the effects of secondary members of the structural load capacity of the existing steel bridge in the modelling for structural analysis.

RÉSUMÉ
Cette étude a pout but de définir la répartition de la charge du trafic sur les membres secondaires, tels que contreventements, entretoises et murs en béton, par comparaison avec les résultats des essais de charge d'un pont métallique existant. Elle montre également comment estimer les effets des membres secondaires sur la résistance du pont existant dans le cadre de l'analyse de la structure.

ZUSAMMENFASSUNG
Diese Studie ist der Ermittlung der Mitwirkung von sekundären Bauelementen wie seitlichen Verstrebungen, Querversteifungen und Betonwänden unter Verkehrsbelastung gewidmet. Dazu werden Vergleiche mit den Ergebnissen von Belastungsversuchen an einer bestehenden Stahlbrücke gezogen. Weiters wird untersucht, wie die Auswirkung sekundärer Bauelemente auf die strukturelle Belastungskapazität der bestehenden Brücke bei der analytischen Modellierung berücksichtigt werden kann.

1. INTRODUCTION

In recent years, some of the composite plate-girder bridges on urban highways in Japan have suffered fatigue damages which have occuered on the connections between main girders and secondary members such as lateral bracings and sway bracings[1]. Local corrosion have also occuered in the secondary members. Furthermore, cracks and vehicular collision have caused damages to the concrete wall parapets.

Lateral bracings and sway bracings are provided to ensure the lateral stability of bridges while their erection work is in progress or after the construction has been completed. Wall parapets are constructed to assure the safety for vehicular falls from bridge floor. These damages, therefore, do not reduce the reliability of steel bridges in direct relation to vehicular traffic loads.

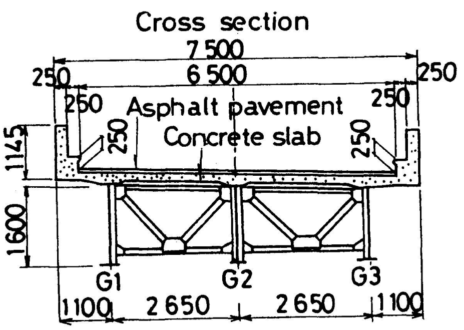
On the other hand, the secondary members form a three-dimensional structure with the main girders, and contribute to load distribution[2,3]. They are considered effective for increasing the structural load carring capacity of the existing bridges. Failure to conduct periodical inspections and to perform adequate repair or reinforcement of these secondary members may help accelerate the damages, resulting in decreasing load carring capacity.

Through the field loading tests for an existing bridge, it was clarified that an individual secondary members can share the traffic load. Furthermore, it was examined how to evaluate the effects of secondary members on the structural load carring capacity of the exsisting bridge by modelling the structural analysis.

2. TESTED BRIDGE OVERVIEW

2.1 Structure

A steel bridge on the Hanshin expressway in the Umeda district in Osaka was chosen as a testing object. The bridge is 28.73m in effective span and 7.5m in total width. It is a simply-supported composite plate-girder bridge composed of three main girders, five pairs of intermediate sway bracings placed at the distance of approximatelly 4.8m, two pairs of end sway bracings each



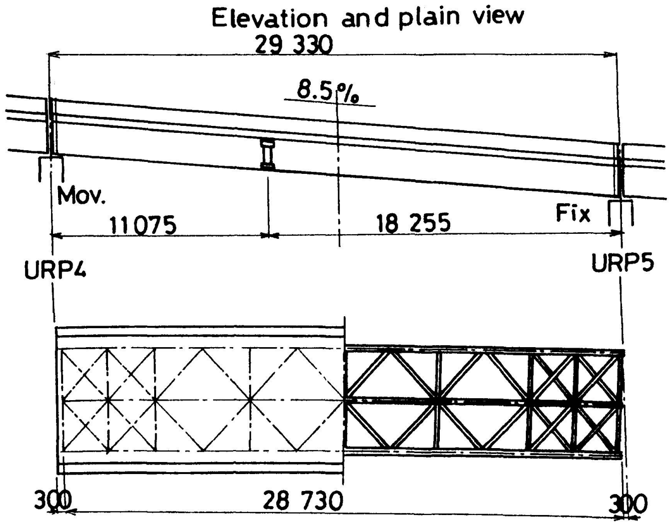


Fig. 1 General view of plate girder bridge

located at either girder end, and lateral bracings provided in a horizontal plane near the bottom flangein the exterior bays. The concrete deck slab is 170mm thick, which is overlaid with 75mm thick asphalt pavement. Concrete wall parapets of 250mm in thicknessand 1000mm in height are located alongside the edge of the concrete slab. Figure 1 illustrates the brige structure.

2.2 Historical background

This bridge has been served for vehicular traffic for approximately twenty-four years since its completion in 1965. The urban redevelopment of this district necessitated the traffic route to be altered, and this bridge to be demolished. Before this bridge was pulled down, loading tests and other various investigations were performed.

This bridge has not received many vehicular loads, because it was a ramp which provided vehicular access to the expressway. Since it was opened, the replacement of expansion joints was major maintenance work. Little deterioration has occurred in this bridge, and this bridge has been maintained in a sound condition.

3. TEST PROGRAM

3.1 Structural systems for loading tests

Figure 2 illustrates the structural systems for which loading tests were carried out. At first, loading test was performed to examine its load carrying capacity after 24 years of service. This is called the loading test of system 1. Next, loading tests evaluating the load share of secondary members such as lateral bracings, sway bracings and concrete wall parapets were carried out. Those tests are called the loading test of system 2, 3 and 4, respectively. In each system, lateral bracings, sway bracings and wall parapets were removed in a serial manner, and identical loading tests were excuted. By comparing the test results of each system, it is possible to clarify the extent of the load sharing at secondary members.

3.2 Loading

Loading trucks were used as a means of providing the required load. Figure 3 shows the dimensions of the

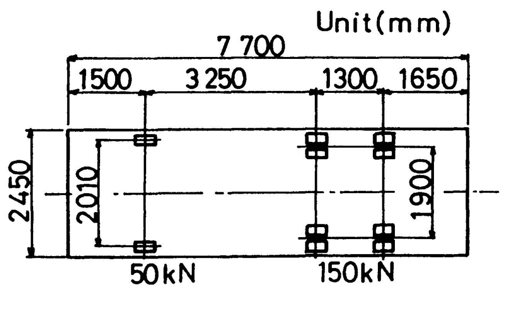


Fig.3 Loading truck

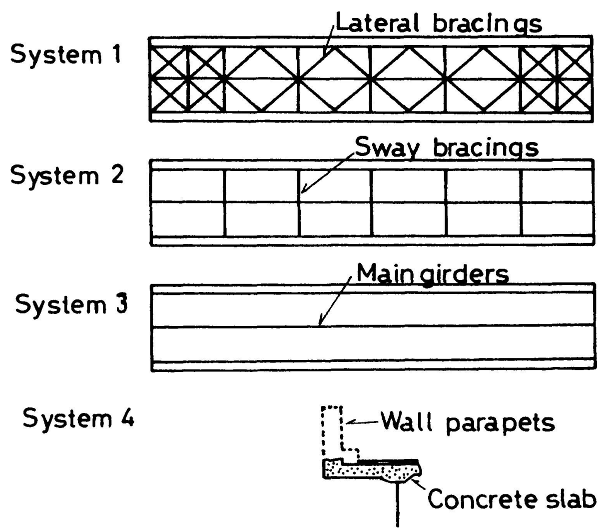


Fig.2 Structural systems for loading tests

loading truck. Figure 4 shows examples of loading truck positions. Tests are arranged so that loading is placed symmetrically on the travelling lane and in the passing lane, and also one lane only as an extreme case.

3.3 Measurement items

Deflections and strains in main girders are measured with regard to the following items:

- Deflections of three main girders in the center of the effective span.
- Strains in the bridge-axis direction of the bridge at both top and bottom franges of main girders in the center of the effective span.

4. ANALYSIS OF THE TESTED BRIDGE

Analysis was conducted by using the four models. Table 1 shows the structural models used for the analysis. Table 2 shows the material properties of steel and concrete, used for the analysis. Models G1, G2 and G3 are used for the grid-girder analysis. Figure 5 illustrates the frame model used for grid-girder analysis. Model G1 is a structural model generally applied to the design of steel plate-girder bridges. In this model, the effective width of the slab complies with the provision in the Japan Specification for Highway Bridges[4].
Model G2 includes the assumptions employed in model G1 as well as adding the bending-rigidity of concrete wall parapets to the exterior girders. Model 3 includes G2's assumption, and in this model the torsional rigidity increased due to the installation of lateral bracings between the main girders is added to the rigidity of main girders. Model F is a model for the three-dimensional finite-element analysis. The mesh division of model F is shown in Fig. 6. For concrete deck slab, concrete wall parapets and webs of main girders , quadrilateral plate elements taking

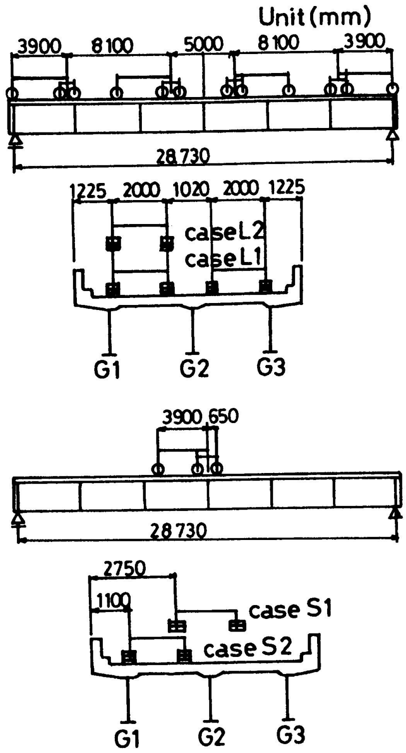


Fig. 4 Loading truck positions

Table 1 Structural models

Structural Model	Lateral Bracings	Wall parapets	Sway bracings
G1	N	N	C
G2	N	C	C
G3	C	C	C
G4	C	C	C

Note C:Considered
N:Not Considered

Table 2 Material properties

Material	Young's modulus	Poisson's ratio
Steel	2.06×10^5MPa	0.3
Concrete	2.94×10^4MPa	0.167

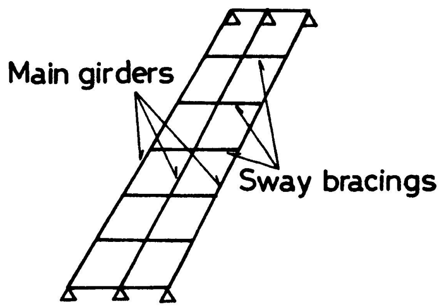


Fig. 5 Frame model for grid-girder analysis

with in-plane and out-of-plane rigidities are used. Top and bottom flanges of the main girders are divided into beam elements. The components of lateral baracings, sway bracings, are also divided into beam elements.

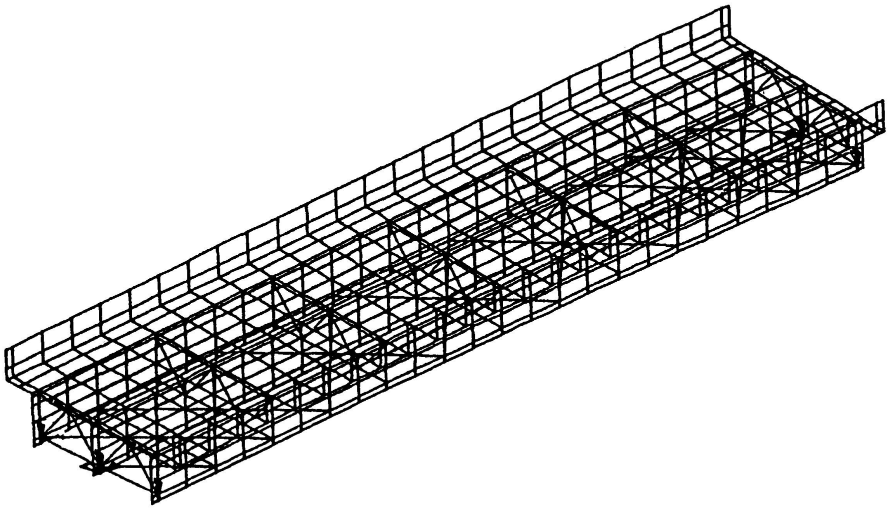


Fig. 6 Mesh division of Model F

5. RESULTS OF LOADING TEST

5.1 Load-carrying capacity in its existing condition

The measurement results of cross-sectional stress distributions in the main girders at the center of the effective span are shown in Fig. 7. The mesured values are smaller than the calculated values in design of this bridge. This indicates that the bridge which has served for twenty-four years completely meets the specified requirements. Differences between the measured values and calculated values are presumably resulted from the fact that effects of the secondary members on load sharing were disregarded in design.

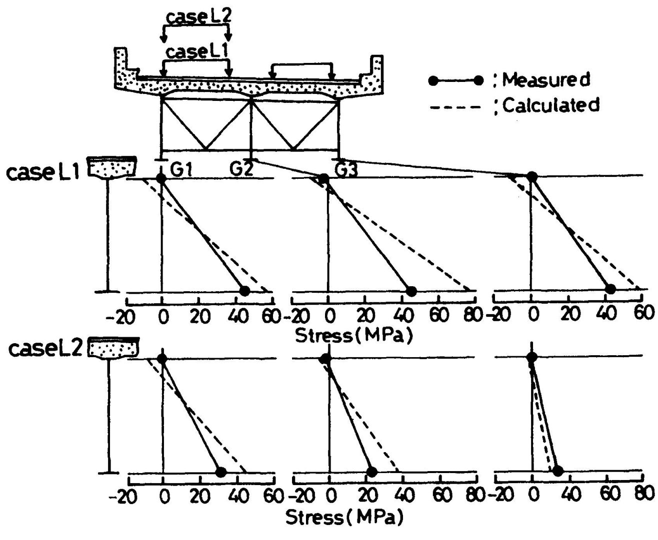


Fig. 7 Stress distributions in main girders

5.2 Effects of secondary members on load distribution

Figure 8 shows the results of the loading tests conducted for the each structural system.

Based on the test results of systems 1 and 2, the effects of lateral bracings on load distribution are described. In the symmetrical loading test (case S1) the measurement result of system 1 is almost similar to that of system 2. In the unsymmetrical loading test (case S2), the lateral distribution of system 1 differs from that of system 2. In the structural system 2 the deflection of the exterior girder(G1) increases. Stress in the bottom flange of girder G1 increases by about 10 percent.

The elimination of lateral bracings in system 2 affects the lateral distribution. This indicates that considering the effects of the lateral bracings on load sharing would be necessary to accurately evaluate load carrying capacity of existing bridges.

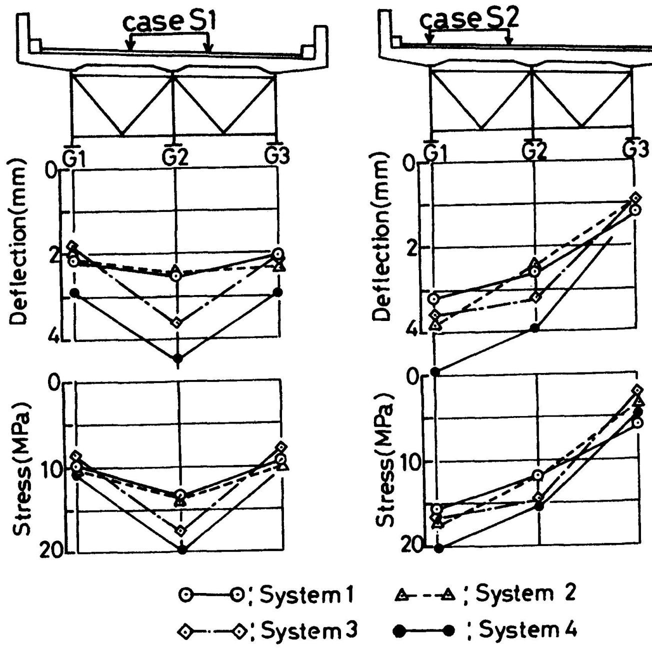


Fig. 8 Deflections and stresses in each system

For the comparison of the test results of system 2 with that of system 3, the effects of intermediate sway bracings on the load sharing are studied. In the results of system 3 in which intermediate sway barcings are eliminated, deflection of interior girder(G2) increases in both symmetrical and eccentric loading tests, and stresses in the bottom flange at the interior girder increase by about 30 percent in comparison with the result in system 2. Hence, the effects of intermediate sway bracings on lateral distoribution are significant. Based on the test result of system 4, the extent of the effect of concrete wall parapets on load sharing is studied. The rate of the increase in both deflection of main girders and stress in bottom flanges of the girders without concrete wall parapets are noticeable. This proves that the concrete wall parapets contribute greatly to the overrall stiffness of the bridge.

6. ANALYSIS RESULTS

Figure 9 shows the comparison between the calculated values and measured values of the deflections of the main girders in the structural system 1. Models G1, G2 and G3 represent the results of the grid-girder analysis, and model F represents results of the three-dimensional finite element analysis. Model G1 is generally used in the design of plate-girder bridges. With this model, the calculated values greatly differ from the measured values. The calculated values for the deflection are approximately two times as much as the measured values. The effects of concrete wall parapets are considered in model 2. In the symmetrical loading test(case L1), the calculated values derived from the analysis using model 2 are much closer to the measured values. In the unsymmetrical loading test (caseL2), the calculated values differ from the measured values.

In model G3, the torsional rigidity increased due to the installation of lateral bracings is considered. The calculated values at model G3 agree well with the measured values, proving that the use of model G3 would enable the accurate evaluation of the actual behavior of any existing girder bridge.
By using the model F, the better understanding of the structural behavior of the bridge could be achieved, but such an analysis requires a considerable set-up time and more expensible computational cost than model G3. Thus, the analysis with the use of model G3 is more appropriate and more economical method in the evaluation of the load carrying capacity of the existing bridges with the effects of secondary members.

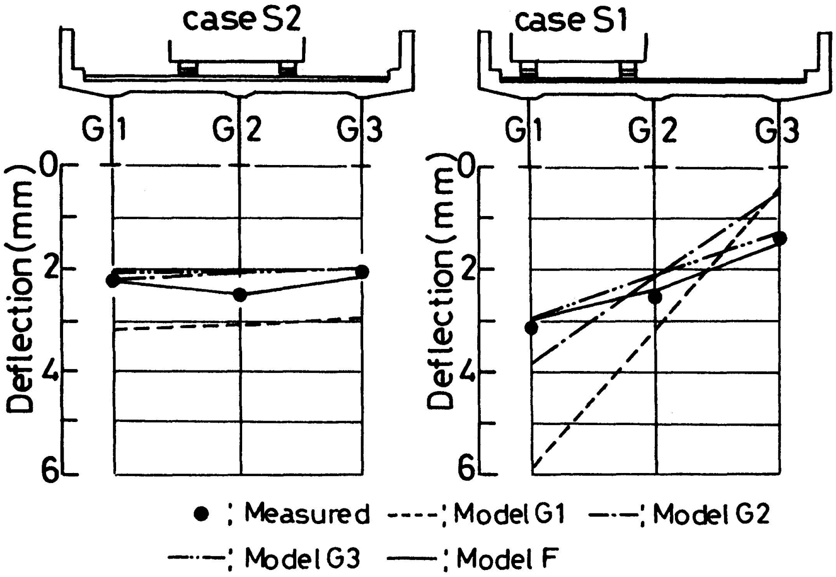


Fig. 9 Comparisons between measured values and calculated ones

7. CONCLUSIONS

From loading tests using an existing bridge, it can be seen that lateral bracings and intermediate sway bracings effectively act on the load distribution, reducing the stress generated in the main girders, and concrete wall parapets significantly contribute to the overrall stiffness of the bridge, reducing the stress generated in the main girders. Therefore, the analysis which also take of the effects of secondary members is required for the better understanding of the structural behavior of steel bridges. Both grid-girder analysis and three-dimensional finite element analysis may be available as methods to evaluate accurate load carrying capacity of existing bridges.

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