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Session D1

Assessment of Bridges I
Evaluation des ponts I
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Load Testing of Chicago's 100-Year-Old Mass Transit Structures

Essais de charge sur les structures centenaires du métro surélevé de Chicago

Belastungsversuche an Chicagos 100jährigen Hochbahnbauten

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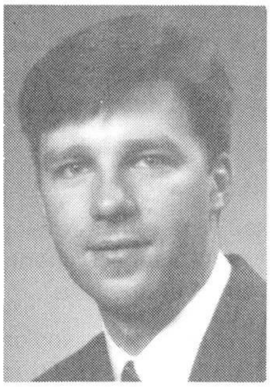

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SUMMARY

As part of an overall engineering assessment of the Chicago Transit Authority's elevated rapid transit system, a field load testing program was carried out to measure the response of the elevated steel structures to static and dynamic train loadings. This paper describes the structures tested, instrumentation and equipment used to obtain strain and deflection measurements, and test results. The study resulted in specific recommendations for future analytical analyses of the elevated structures.

RÉSUMÉ

Une évaluation globale du système ferroviaire surélevé en zone urbaine a été menée par la "Transit Authority" de Chicago. Un programme d'essais de charge a été effectué sur le site en vue d'enregistrer le comportement des structures métalliques soumises aux effets statiques et dynamiques de convois-types. L'article présente une description des ouvrages testés, l'instrumentation et l'équipement servant à mesurer les allongements et les flèches des éléments, ainsi que les résultats des essais. Cette étude fournit également des recommandations précises pouvant servir à des analyses futures sur des ouvrages ferroviaires surélevés.

ZUSAMMENFASSUNG

Als Teil einer ingenieurmässigen Gesamtbeurteilung des städtischen Hochbahnsystems der Chicagoer Transit Authority wurde ein Feldprogramm für Belastungsversuche durchgeführt, um das Verhalten der aufgeständerten Stahltragwerke unter statischen und dynamischen Eisenbahnlasten zu messen. Der Beitrag beschreibt die geprüften Tragwerke, die Instrumentierung und Ausrüstung für die Dehnung und Durchbiegungsmessungen sowie die Prüfergebnisse. Die Untersuchung ergab spezifische Empfehlungen für spätere Nachrechnungen solcher Hochbahnbauten.

1.0 INTRODUCTION

1.1 Background

The Chicago Transit Authority (CTA) operates and maintains the second largest mass transportation system in the United States: A system which comprises an asset base valued at over $15 billion (US dollars) and which serves 2.3 million passengers daily [1]. The largest portion of the system is an elevated rail structure, the majority of which was designed and constructed between 1895 and 1905. In order to determine the exact operating condition of every mile of the elevated rail system, the CTA conducted an exhaustive multimillion dollar engineering assessment. This assessment, the most comprehensive ever undertaken on a mass transit system, was carried out with the eventual goal of rehabilitating this vital infrastructure.

As part of the overall engineering assessment, a field load testing program was carried out to measure the response of the elevated steel structure under static and dynamic train loadings. To account for variations in the structural systems which comprise the elevated rail system, sixteen locations were selected for inclusion in the field testing program. Test results presented herein are specific to three tests conducted on the CTA's Loop Line. The Loop Line structures measure approximately 5 km in length and service the downtown business district of Chicago, Illinois. This paper describes the Loop Line structures, instrumentation and equipment used to obtain strain and deflection measurements, and test results. This study resulted in specific recommendations for future analytical analyses of the elevated structures.

1.2 Description of Tested Structures

The CTA's Loop Line is an open deck, elevated steel structure supporting two tracks. The structure was designed in late 1895 and early 1896 by renown bridge engineer, J.A.L. Waddell and put into service shortly thereafter. Medium steel corresponding to ASTM A-7 steel is used throughout the Loop Line structure.

The overall structure is comprised of multiple spans each having an average length of 15.2 m, with expansion joints located every third span. Each track is supported by two, open web truss stringers as shown in Figure 1. The stringer top chord is a built-up member consisting of two angles and a vertical plate. The stringer bottom chord and web members consist of double angles. Each stringer pair is braced laterally using angles in the plane of the top chord and diagonal cross bracing at stringer midspan and end supports. Stringers frame into riveted built-up cross-girders which span between riveted built-up columns. Riveted full-depth web angles are used to complete the stringer to cross-girder attachment. At expansion joints, the stringer is supported by a seated bearing connection.

Three representative segments of the Loop Line were selected for inclusion in the field testing program. Each test location included the instrumentation of two adjacent spans, one span having both stringer ends attached to the cross-girder and one span having one end attached to the cross-girder and the other at an expansion joint. Test locations were identified by the centrally located bent number, that is 0116, 0122, 0164. The segments were similar in construction except for their top chord flange angles. Test location 0122 represents construction with all components dating from 1897 while test locations 0164 and 0116 represent spans which have had their top chord flange angles replaced using riveted and bolted construction, respectively. Replacement flange angles were similar in size to the original angles.

2.0 INSTRUMENTATION AND FIELD TESTING

2.1 Objectives of Field Testing Program

A field testing program was carried out to measure the response of the elevated structure to static and dynamic train loadings. Strain and deflection measurements were recorded under static loading provided by a control train and dynamic loading using the control train and normal rush hour train traffic. Objectives of the testing program were as follows: Measure strains and deflections in stringers to verify the analytical analysis; Estimate the level of stringer end fixity at cross-girder connections; Develop influence lines for instrumented locations using the control train; Determine stress range experienced by tension members of the truss stringer; Measure longitudinal response to dynamic and braking loadings; and, Determine impact levels.

2.2 Description of Instrumentation

A total of 40 foil type, single element strain gages were installed at each of the three test locations. Gages were placed on the top and bottom chords at each stringer end and the midspan to monitor maximum negative and positive bending, respectively. Web elements expected to experience the largest tensile strains were instrumented. Gages were also placed at the column bases to measure maximum strains due to axial and bending forces. Deflections were monitored using seven linear variable displacement transformers (LVDT). LVDT's were positioned at the three supporting cross-girders and each midspan location to measure vertical displacements. Two LVDT's were positioned at the expansion joint to measure horizontal displacement of the cross-girder and relative bearing slip. To record the data, a van was equipped as a recording station. Lead wires from strain gages and LVDT's were routed to the recording station and connected to a data acquisition system.

2.3 Field Testing

An empty four-car CTA train was used as the control load for the dynamic and static testing. Each train car has a mass of approximately 24,500 kg or 6,125 kg per axle. For each car, individual truck axles are separated by 2 m with a distance of 10.3 m between truck centerlines. The truck centerline of adjacent cars is separated by 4.4 m.

For the control static tests, the test train was positioned at 1.2 m intervals along the structure and strains and deflections were recorded. A total of 90 intervals was used with the first and last interval location resulting in zero strain. For the control dynamic tests, the test train passed over the instrumented spans at various speeds ranging from a crawl, that is less than 8 km/hr, to a maximum speed of 56 km/hr. Dynamic braking tests were also conducted by having the test train obtain a speed of 56 km/hr then applying axle and track brakes simultaneously. This braking represents the most severe braking condition. Dynamic tests were also conducted under normal rush hour traffic. These tests were conducted to determine the influence of passenger loading on maximum stress levels. For CTA fatigue rating calculations, a passenger loading of 2,495 kg per axle is combined with dead load to obtain 8,845 kg per axle.

3.0 STRESSES UNDER CONTROL LOADING AND REVENUE TRAFFIC

3.1 Static Testing

Strains due to the control loading were plotted with respect to load position to develop influence lines for all gages at each test location. The ordinate and abscissa correspond to the calculated stress and load position of the train, respectively. Positive stress represents tension and negative stress represents compression. In general, data show five distinct humps as shown in Figure 2 for the two midspan locations at test location 0116. The first hump is the first truck (2 axles) of the leading car. The middle three humps represent pairs of trucks (4 axles) at the coupler connection between cars. The last hump is the rear truck (2 axles) of the last car. The middle humps show a larger magnitude because they represent the loading of four axles rather than two axles. Data plotted for other gaged locations and included in Reference [2] were similar to that shown in Figure 2.

Data from the stringer top chord revealed localized bending effects due to concentrated loads applied through the wood ties randomly located along the stringer length. Top chord flanges typically experienced compression while the base of the vertical top chord plate experienced tension.

A summary of the maximum stress ranges for test location 0116 is shown in Table 1. Data for the two other test locations were similar. The largest tensile stress range was 21.4 MPa and was measured in the end diagonal. The positive moment tensile stress range at midspan averaged 20.7 MPa while the negative moment tensile stress range averaged 10.3 MPa adjacent the stringer to cross-girder attachment.

Midspan deflections were corrected for support displacements and plotted with respect to load position for all test locations [2]. Maximum deflections did not exceed 4 mm. Both the deflection and stress plots indicate significant continuity at stringer end to cross-girder connections. The deflection plots show upward displacements of 1.8 mm when the train is positioned in an adjacent span. In general, upward deflections did not occur across expansion joints. Similarly, negative bottom chord plots in Figure 2 indicate continuity across stringer to cross-girder connections.

3.2 Dynamic Testing

Similar to the static testing, strains and deflections were plotted with respect to load position [2]. Dynamic stress levels under the control train increased from static measurements by approximately 15 percent for the three test locations. This is significantly less than the design impact level of 57 percent calculated using the requirements of the American Railway Engineering Association (AREA) specifications [3].

Data obtained under rush hour train traffic indicates increases in stress levels from the control test data of about 20 percent. The maximum stress range measured at midspan was 26.2 MPa compression and 28.3 MPa tension.

4.0 ANALYTICAL STUDIES

The deep riveted connections at the stringer ends provide a significant amount of fixity, affecting the stress levels and behavior of the stringers. Analytical models were developed to determine the level of end fixity by calibrating to field data. For spans with connections at each end, the effective end fixity was determined to be approximately 75 percent of the fully fixed end moment. At spans with an expansion joint at one end and assuming the expansion joint contributes no fixity, the riveted end connection was determined to contribute between 95 to 103 percent fixity. This finding indicates that some fixity must be provided at the expansion joint. Fixity at the expansion joint occurs as a force couple consisting of a tensile force in the rail and a shearing force transferred across the expansion joint at the steel-to-steel bearing.

5.0 CONCLUSIONS

Load tests of three separate segments of the Loop Line indicate similar results even though their top chord flange angle construction differs. Measurements obtained in spans having flange angles dating to the original 1897 construction were similar to those obtained in spans having replacement angles with bolted or riveted construction. Under rush hour trains, the maximum tensile stress range in the end diagonal and midspan bottom flange were similar and did not exceed 30 MPa. Comparison of static and dynamic data under control loading indicates 15 percent would be a realistic value for impact calculations. The riveted stringer end connection is such that significant continuity is provided at the stringer end support. By calibrating an analytical model to the field data, the relative end fixity was determined to be approximately 75 percent of the fully fixed end moment.

Research indicates that riveted bridge members are not likely to develop fatigue cracks in primary members when the stress ranges are less than 48 MPa [4]. Based on this research, it is projected that the primary stringer members used in the Loop structure as originally designed or currently reflanged, and under current loadings could be expected to have a remaining fatigue life of about 80 years. However, stringer connection angles and connection angle rivets may exhibit cracking or failure and require replacement before reaching this projected life.

The findings of this work resulted in the recommendation of guidelines for the analytical evaluation of the Loop structure. These recommendations include the more realistic impact and end fixity findings reported herein.

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TABLE 1 SUMMARY OF MEASURED STRESSES, TEST LOCATION 0116[1]

ELEMENT	CONTROL LOAD STATIC (MPa)	CONTROL LOAD DYNAMIC (MPa)	IMPACT (%)	RUSH HOUR DYNAMIC (MPa)
Midspan Bottom Chord	20.7	24.8	20	28.3
Midspan Top Chord Flange	-20.0	-23.4	17	-26.2
Midspan Top Chord Vertical Plate	10.3	11.0	7	13.8
End Diagonal	21.4	24.1	13	26.9
Stringer End Bottom Chord	-22.0	-25.2	-14	-27.8
Stringer End Top Chord Flange	10.3	11.7	13	13.1
Stringer End Top Chord Vertical Plate	-10.3	-11.0	7	13.1

Notes: 1. Measured stresses for Test Locations 0122 and 0164 were similar

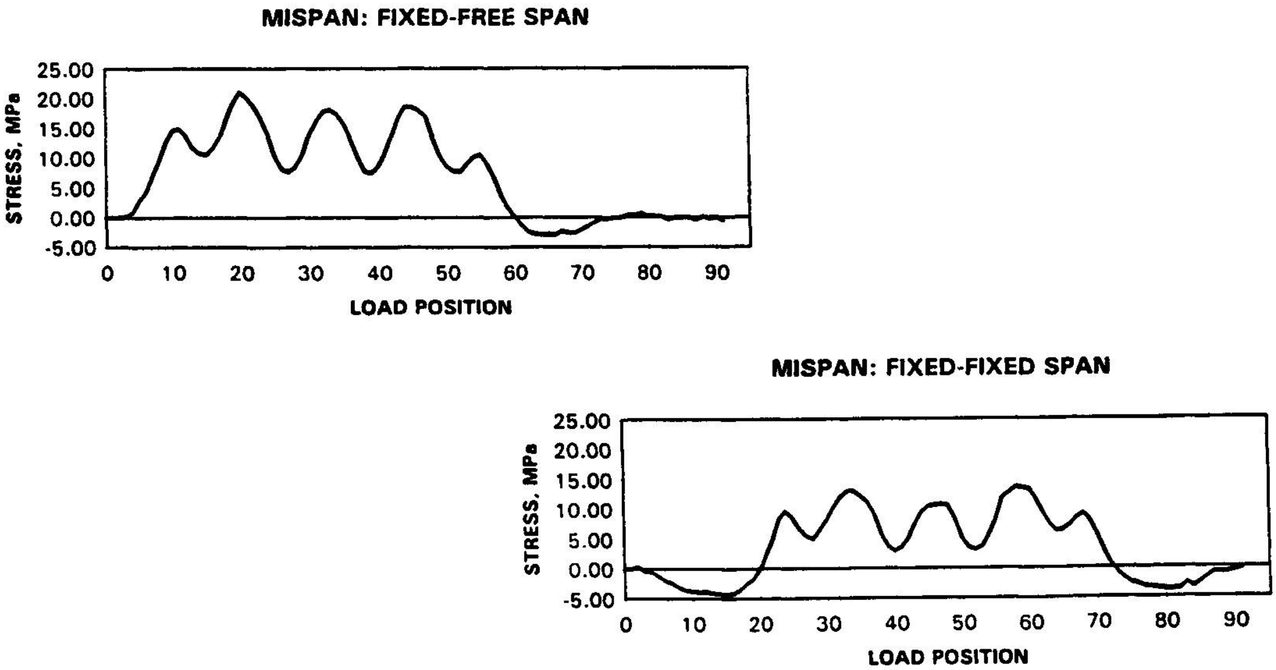

Figure 2 - Midspan stress response for midspan gages at Test Location 0116

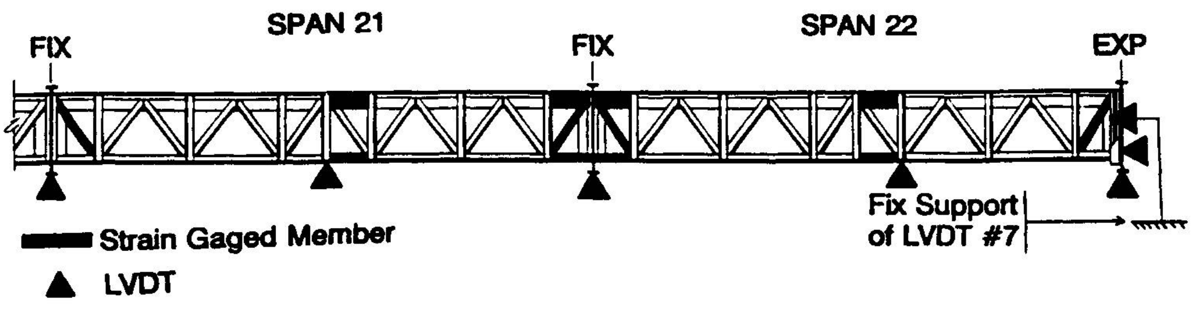

Figure 1 - Typical Loop structure and instrumentation plan

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Testing a Thirty-Year-Old Concrete Railway Bridge in China

Essais réalisés sur un pont ferroviaire en béton de trente ans en Chine

Versuche an einer dreissigjährigen Betoneisenbahnbrücke in China

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SUMMARY

In order to investigate the load capacity of existing concrete bridges, the static, dynamic and destructive tests of a 30-year-old concrete railway bridge which was deteriorated seriously were conducted, from which the stresses, deflections, and crack widths under design loading and ultimate loading were observed, also the load capacity as well as the strength of the concrete and steel. By analysing the test results, the methods to evaluate the strength and the degree of deterioration of existing concrete railway bridges are given in this paper.

RÉSUMÉ

Des essais statiques, dynamiques et conduisant à la ruine ont été conduits sur un pont ferroviaire en béton de trente ans afin d'étudier la résistance de ponts en béton très endommagés. Le programme de mesures comprenait les tensions, déformations et largeurs de fissures sous des charges de projet et des charges de ruine, de même que la capacité de charge, la résistance du béton et de l'acier. L'analyse des résultats d'essais et les méthodes d'évaluation de la résistance et du degré de détérioration des ponts ferroviaires en béton existants sont présentés dans l'article.

ZUSAMMENFASSUNG

Um die Tragfähigkeit bestehender Betonbrücken zu untersuchen, wurden an einer 30 Jahre alten, stark geschädigten Betoneisenbahnbrücke statische und dynamische Belastungsversuche bis zum Bruch vorgenommen. Das Messprogramm umfasste Spannungen, Durchbiegungen und Rissweiten unter Gebrauchs- und Bruchlast, die Traglast sowie die Beton- und Stahlfestigkeit. Der Beitrag beschreibt die aus den Versuchen abgeleiteten Methoden zur Ermittlung der Tragfähigkeit und des Schädungsgrades bestehender Betoneisenbahnbrücken.

1. INTRODUCTION

There ara a large number of old RC railway bridges in China now which were deteriorated to a ceertain extent, some of them had no records about the design and construction or the design grades of them were lower. To make use of them rationally, it will be a key problem to study their load capacity, degree of damage, and remaining life, etc. For solving these subjects, the static, dynamic and destructive tests of a 30-year-old RC railway beam bridge which is located in northeast China was carried out. In the tests, the stresses, deflections and crack widthes in the beam were measured, and the response of the bridge up to failure was also studied. After the tests were finished, the beam was broken to inspect the actual arrangement and corrosion of the steel, and 12 pieces of them were used to measure the mechanical properties of the steel. Based on the studying, we try to propose a method to evaluate the strength and deterioration of old RC railway bridges.

2. METHOD OF TESTS

The bridge in tests was a four-span and simple supported concrete beam, as shown in *Figure 1. The bridge was constructed* originally in 1933 and repaired later. Now, extensive cracking had developed on the underside of the beams. A part of the concrete in lower flange was peeied off, so the reinforcement were corroded seriously. All of the supports were damaged to some extent because of corrosion of the steel plate and damage of the concrete.

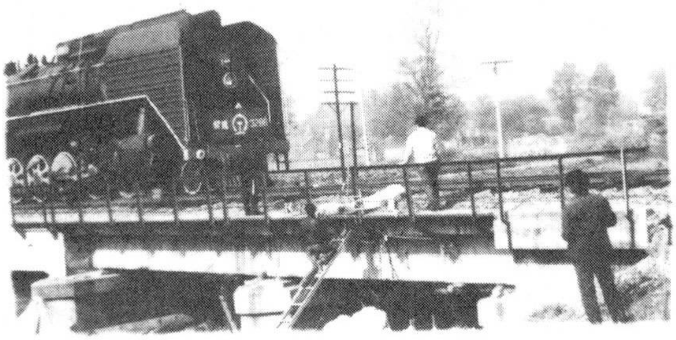

Fig.1 Test bridge

2.1 Static tests

The steam locomotive was used to apply the vertical load on the bridge in the locations where the maximum moment at the middle span and one quarter span were acquired respectively. Each load form was repeated 3～4 times. In the tests, the deflections, strains of concrete and steel and crack widthes were carefully measured.

2.2 Dynamic tests

Vibration of the bridge was excited when the test locomotive was runing through at a speed of 10～80 km/hour. The responses of the test beam were measured with accelerometers and CZ bridge vibrograph. At the same time, the deflection and strain of the beam under dynamic loading were also measured.

2.3 Destructive tests
The 3-point loading method was selected for modeling the actural load distribution for the design load. The tests under working load and design load were repeated for three times first, and then increasing the load step by step for cycling to investigate the effects on larger loads. At last, the destructive test of the beam was conducted.

3. ANALYSIS OF TESTED RESULTS

3.1 Results of static tests
The comparision for the measured and calculated results of the strain in reinforcing steel, deflection in middle span, maximum crack width and depth of neutral axis was as shown in table 1.

Item	Strain of Reba ($\mu\varepsilon$)	Deflection	Crack width (mm)*	Neutral Axis Depth (cm)
Measured Value	80.79	0.76	0.24	25.50
Calculated Value	196.46	1.03	0.04	29.22
η **	0.41	0.73	6.00	0.87

* The measured maximum residual crack width was equal to 0.2mm.
** η = Measured Value/Calculated Value

Table 1 some results from static test

Note that in table 1, the differences between the measured values and calculated values are very large except that of the depth of neutral axis.

3.2 Results of dynamic tests
Fig.2 gives the measured impact factor $(1+\mu)$-V curve. The maximum $(1+\mu)$ was as high as 1.43 and some crests existed in the $(1+\mu)$-V curve. The reasons were that the vibration of the bridge was excited by many factors, such as the speed of locomotive, differences in the axleweights, flatness of the rail on the bridge, and the s-shaped movement between the rail and the wheel, etc. As the freqency of the exciting forces was nearing the natural frequency of the bridge, the measured impact factor would be much larger, and a crest appeared in the $(1+\mu)$-V curve. By spectral analysis for the free vibration, the fundamental natural frequency and the damping factor of the test beam were acquired equal to 34.8 Hz and 0.061 respectively.

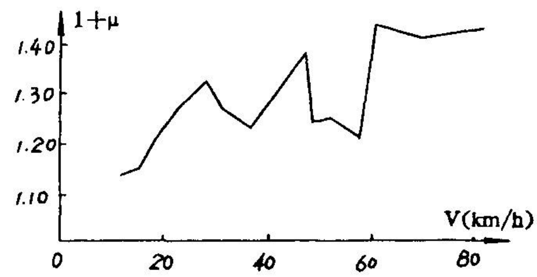

Fig.2 Measure impact factors

3.3 Results of destructive tests
Fig.3 gives the measured load-deflection curve(P-f curve) in destructive test It is shown that the measured deflections both in middle span and quarter

span increase linearly with the load when P_{max} was less then 518.8 kN. If exceeding this value, the measured deflection was increasing visibly and the deformation of the concrete and reinforcement would become nonelasticity, and some of the reinforcing steel in lower flange may be yielded this time. When the load P exceeded 601.8kN, the measured deflection was increasing very rapidly, and the beam had shown very evident yielding phenomenon. When load increasd to 684.7 kN, the beam had totally yielded. When P was equal to 761.7 kN, the beam had come into strengthening stage. The measured destructive load was equal to 1049.8kN. The measured deflections at quarter span and three quarter span were very close to each other, showing that no serious local deteriorations existed which would result in decrease in local stiffness.

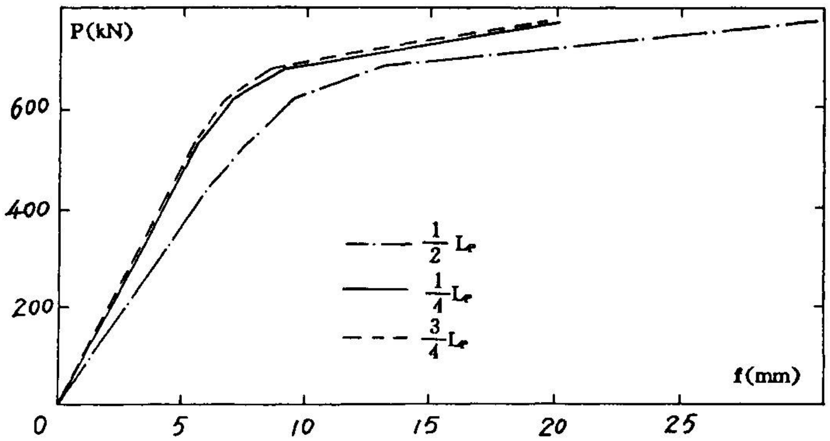

Fig.3 Measured P-f curve

Fig.4 gives the measured crack widthes in destructive test, where the vertical crack W_1 and W_2 as well as the diagonal tension crack W_1' were all that its crack width was maximum. As shown in Fig.4, when no load was act on the beam, the residural crack width was equal to 0.22 mm. When design load acting, it was as big as 0.38 mm. When the maximum load was less then 352.9kN, the beam is still in the range of elasticity although some new cracks had happened. When the load P was exceeded to 601.8 kN, the maximum crack widthes increased very rapidly, indicating that some of the reinforcing steel had yielded.

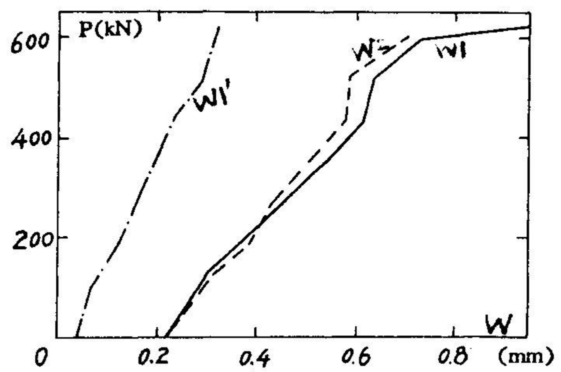

Fig.4 Measured P-W curve

The diagonal tension cracks appeared when the load P was about equal to 352.9 kN, but its maximum cracks widthes were much less than that of vertical cracks, so it had no influences on the load capacity of the bridge.

4. EVALUATION METHOD

4.1 Basical rules for evaluation

Considering the characteristics of the existing structures, the rules bellow must be followed. That is, the geometrical dimensions of the structure must be insitu measured, especially to the weeker sections; the strength of the meterials must be measured in situ; the degree of deterioration in reinforcing steel and concrete in the key sections must be considered.

4.2 Load capacity evaluation

The design and construction records can be used to evaluate the strength for the bridges that no serious deteriorations have happened and the records are applicable. But some times, we had no records about the design and construction or the deteriorations are very serious. For these cases, the methods bellow can be used to evaluate its load capacity.

In practice, many parameters(such as the stresses, deflections, crack widthes, fundamental frequency, etc) can be measured by tests. Suppose that the calculated parameters Y_{jo} is the function of x_i, where x_i are the key factors which affect the strength of RC beams(such as A_s, f_c', etc), that is:

$$y_{jo}=f_j(x_1,x_2...x_i...x_m) \quad (1)$$

Suppose that the number of the measured parameters is N, that is: $0 < j > N$, the measured parameters is y_j which is correspond to y_{jo}, we can get:

$$E_{rr}^2=\Sigma(y_j-y_{jo})^2 \quad (2)$$

$$E_{rr}^2=\Sigma\left[y_j-f_j(x_1,x_2...x_i...x_m)\right]^2 \quad (3)$$

The x_i can be calculated by following equation:

$$\frac{\partial E_{rr}^2}{\partial x_i}=0 \quad (4)$$

When the x_i are determined, the strength and serviceability of the bridge can be evaluated by the theory of reinforced concrete, that is:

$$F_d=f_d(x_1,x_2...x_i...x_m) \quad (5)$$

4.3 Evaluation for degree of deterioration

Structural damage evaluation - Here the structural damage represents the damage which results in decreases of the strength. The damage index D_s for this kind of deterioration can be definited as:

$$D_s = M_{us}/M_{uj} \quad (6)$$

where M_{us}=the actual strength. It can be gotten from evaluation;
M_{uj}=Calculated strength.

Then, an other definition is given as bellow:

$D_s>0.95$ no deterioration

$0.8<D_s<0.95$ permited deterioration

$D_s<0.8$ not permited serious deterioration

The calculated D_s by equation (6) to the test bridge is equal to 0.87. so we consider that the structural damage in the bridge is permited.

Overall deterioration evaluation - Here the overall deterioration represents a kind of the functional decrease, such as, the deformation and stress are *too large or the vibration is too violent due to the cracking or spalling or weathering of concrete, or due to the corrosion of reinforcing steel. The overall damage index can be definited as:

$$D_t=(EI)_m/(EI)_j \qquad (7)$$

where $(EI)_m$=measured stiffness;
$(EI)_j$=calculated stiffness.

Then, we definite:

$D_t \geqslant 0.95$ no deterioration

$0.8 \leqslant D_t < 0.95$ permited deterioration

$D_t < 0.8$ serious deterioration

To the test bridge, the calculated D_t is equal to 0.79. It is concluded that the overall deterioration is serious. By inspection, this deterioration was mainly caused by the weathering and spalling of concrete and the serious corrosion of reinforcing steel.

5. CONCLUSION

1. The deterioration in the test beam was serious;

2. The load capacity of the test bridge is enogh to the design load, showing that the potential strength of RC bridges is large;

3. The measured depth of neutral axis is a stable parameter and it can be used in the strength evaluation:

4. The residual crack widthes and the depth of carbonized concrete were large. The corrosion of reinforcing steel was very serious. The original load capacity and service life of the bridgehad been decreased.

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Analysis of Dismantled Bridge Girders

Analyse des poutres de ponts démontés

Analyse von demontierten Brückenträgern

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SUMMARY

Post-tensioned prestressed bridge girders subjected to a heavy long-term service were experimentally tested and numerically analysed. A nonlinear method based on the finite element beam layered model and a three-dimensional nonlinear brick elements of a commercial package were used to verify the experimental results. Load-deflection curve, detailed examination of crack propagation, influence of environment, bearing capacity and the influence of bonded and partially bonded tendons were among the most important factors.

RÉSUMÉ

Des poutres précontraintes de ponts soumises à de lourdes charges de longue durée ont été soumises à des essais et analysées numériquement. Une méthode non-linéaire par éléments finis et un modèle tridimensionnel ont été utilisés pour une comparaison avec les résultats d'essai. Les facteurs les plus importants étaient la dépendance entre charge et flèche, l'examen détaillé du développement des fissures, l'influence du milieu, la force portante limite et l'influence des câbles partiellement et complètement injectés.

ZUSAMMENFASSUNG

Die mit nachträglichem Verbund vorgespannten und sich im Betrieb unter langfristigen und schweren Lasten befindenden Brückenträger wurden experimentell geprüft und numerisch analysiert. Die nichtlineare Methode, die auf dem Stab-Schnitt-Modell und auf den nichtlinearen dreidimensionalen Elementen von Computer-Programmen beruht, wurde zur Überprüfung von experimentellen Ergebnissen verwendet. Die Belastungs-Durchbiegungs-Kurve, die Detailuntersuchung der Rissentwicklung, der Umgebungsfaktor, die Grenztragfähigkeit und der Einfluss von injektierten und teilweise injektierten Kabeln gehörten zu den wichtigsten untersuchten Faktoren.

1. INTRODUCTION

Experimental testing of dismantled structures can bring valuable information about their behavior under specific conditions. Present study describes the experimental testing and following numerical modeling of two girders of a dismantled road overbridge near Lipnik and two girders of a road bridge near Vsestary, Czech Republic. These testing provided us with a unique chance for detailed investigation, evaluation of structural performance and verification of loading capacity of prestressed concrete girders after a long service life. An advanced numerical analyses using nonlinear FEM technique respecting real behavior of girders were carried out.

2. EXPERIMENTAL TESTING

2.1 Lipnik overbridge

2.1.1 General description

Lipnik overbridge was built as a temporary one. For 20 years it was subjected to heavy loading and aggressive environment being above a railway. It was necessary to dismantle the overbridge mainly due to damages of supports and bridge accessories. Two selected girders were tested and numerically analyzed. Girders were slightly damaged during dismantling.

2.1.2 Materials

The quality of concrete was very high, up to 10 per cent higher than the design required with the strength in a range from 62 to 71 MPa. Real dimensions of the girders were carefully measured and compared to design parameters. Surface degradation of concrete by chlorides was not deep due to the high quality of concrete and did not influence the bearing capacity of the girders. The worst chloride damages were found in anchor regions but the strength of prestressing cables was not decreased. Material properties like strength of concrete in tension and compression and stress-strain diagram of prestressing steel were examined. Nine pieces of 50mm diameter cylinders were used for material tests of concrete of each girder. Prestressing steel properties were determined from 42 test specimens for girder L1 and 45 test specimens for girder L2.

2.1.3 Load tests

Girders were gradually loaded and unloaded respecting the load increment corresponding to 20 per cent of the design load. The girders L1 and L2 collapsed due to crushing of concrete at the load level of 260 per cent. The first visible cracks were identified in the cross-section of rusted stirrup applying the load level of 120 per cent. Other cracks were found at the load level of 140 per cent. The girders were cut in two cross-sections after the tests had been finished to investigate the location of prestressing cables. Some cables were found to be fully bonded, some of them partially bonded and some of them unboded, eventually rusted. These tests were prepared and supervised by the second author, and more details can be found in [1].

2.2. Vsestary bridge

Vsestary bridge was built of 19m span girders in 1969. A poor design and the details workmanship caused water leaking into the structure. Some of the edge girders were damaged by frozen water and lengthwise cracks were detected especially in the bottom slab. The bridge was partially dismantled and two girders were selected for experimental testing. Material tests, measurements of dimensions and the way of loading were similar to the previously described testing. The first girder (V1) was loaded up to 262 per cent of the design load when the loading was stopped due to large deflections but the girder did not collapse. Girder V1 represented moderately damaged girders of the bridge. Girder V2 suddenly collapsed at the level of 217 per cent of the design load due to the crushing of concrete. This girder was highly deteriorated by leaking water and chlorides and partially damaged during dismantling. The first author had a chance to participate the testing and collected all necessary information.

3. NUMERICAL ANALYSIS

All four girders were numerically tested using alternative methods based on the nonlinear finite elements procedures. Real dimensions of the girders taking into account local damages, measured locations of prestressing cables and results of material tests were used as an input data.

3.1 Beam layered model

A beam FEM layered model similar to [2], that has been successfully used and verified for several analyses of reinforced and prestressed girders ([3]) were also adopted here .

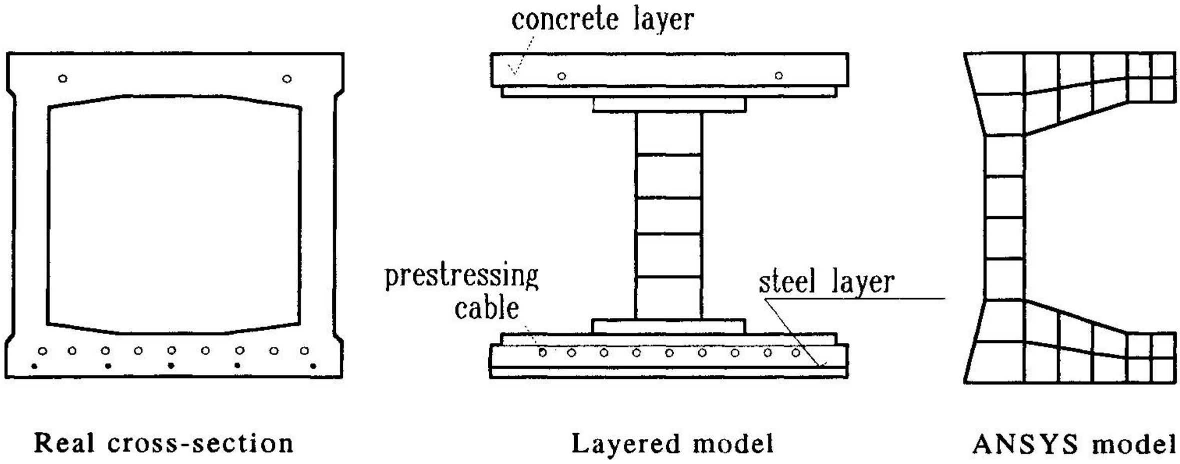

Fig. 1 - Modeling of cross-section

3.1.1 General description

The model respects nonlinear behavior of materials, time dependent volume changes like creep and shrinkage, a history of loading and prestressing and partially bonded/unbonded cables. Time-dependent analyses with estimated values of loads and creep parameters were performed to calculate the level of stress in prestressing cables at the beginning of load tests. Real cross-section of girder L1 and corresponding layered and ANSYS models are shown in Fig. 1. Fig. 2 demonstrates girder L1 with its prestressing cables.

3.1.2 Simulation of load tests

A combination of incremental method and Newton-Raphson iteration technique was used to perform the solution. Load increments in the numerical analysis corresponded to the load increments of the test, usually 20 per cent of the design load. Equivalent load was applied as concentrated forces at the same location like the forces of load test.

3.1.3 Numerical simulation

A full Newton-Raphson method with the change of a stiffness matrix after each iteration was found to be the most suitable method because of quite fast convergence requiring up to 15 iterations per load step. Beam models are simple from numerical point of view. A full nonlinear analysis was completed within a few minutes on PC 486 computer.

3.2 Nonlinear brick model

A commercial finite element method package ANSYS [4] was also used to confirm numerical results. 3D nonlinear reinforced concrete brick element SOLID65 was adopted to discretize the girders and model the properties of concrete. Offset nodes beam element BEAM44 was used for prestressing cables. The same level of load increments and the full Newton-Raphson iteration technique were applied here. FEM discretization of girder V1 using 2842 brick elements and 2504 beam elements is shown in Fig. 3. 3D analyses offer very sophisticated results but are quite complex from numerical point of view. The results were available on the same computer like beam analysis in 67 hours. One should consider whether improved results worth such increasing of computation time. ANSYS models were developed as a part of [5].

3.3 Parametric studies

3.3.1 Time-dependent analysis

Several parametric studies have been carried out for a better understanding of the behavior of the girders. The most important study was related to the calculations of the level of prestressing before tests started. Various values of creep parameters and loads were considered in these analyses to obtain an estimation of stress in prestressing cables at the beginning of load tests.

3.3.2 Real cracks modeling

The propagation of cracks were carefully investigated during the load tests. The location of cracks were monitored for all load increments. This enables us to carry out the analysis respecting real cracks regions. For this parametric study the layered beam model was used Cracked layers were not determined by calculation but were directly input as pre-defined data for all load increments. This study helped us to assess the accuracy of the method.

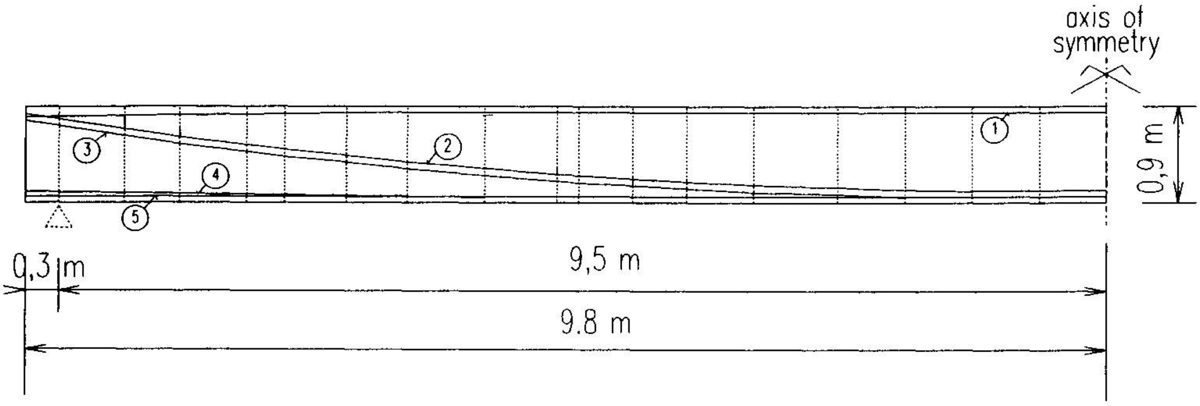

Fig. 2 - Girder L1 - Layered Model

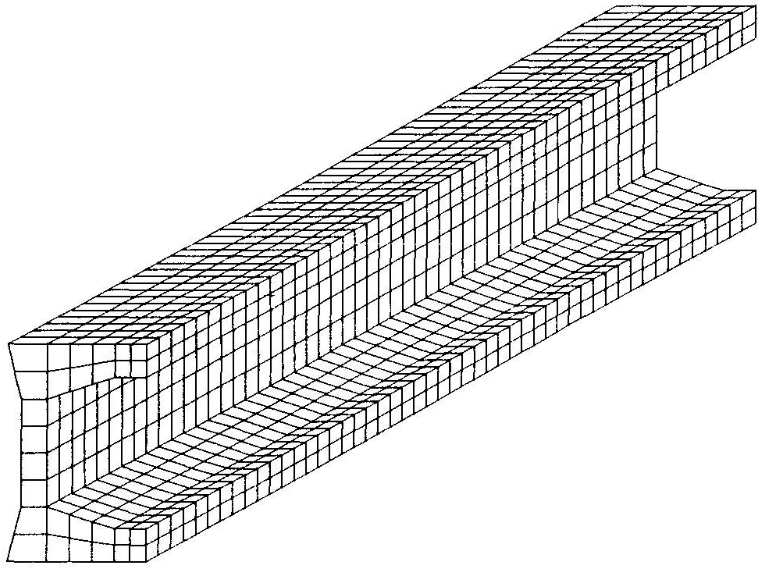

Fig. 3 - ANSYS model

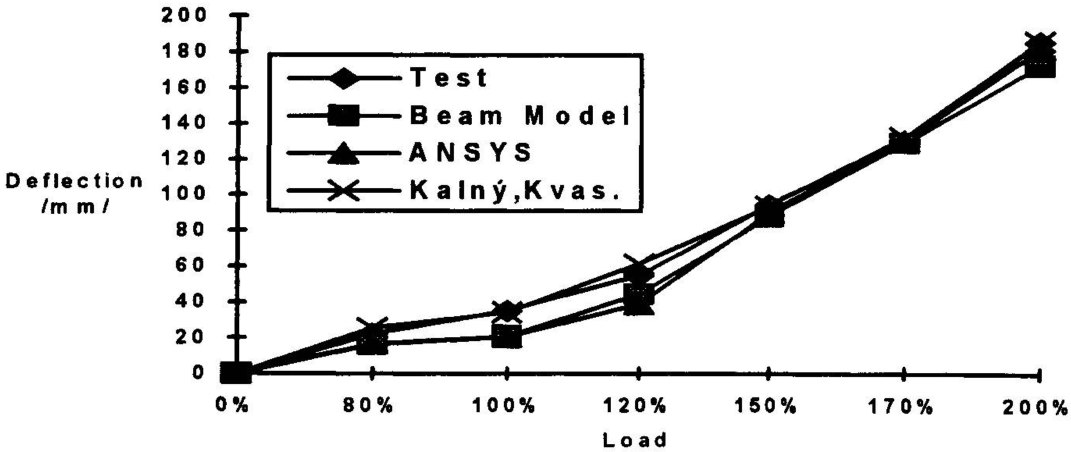

Fig.4 Load-deflection Curve for Girder V2

3.3.3 Bonded and unbonded cables analyses

Girder L1 was analyzed considering the prestressing cables fully bonded, unbonded and partially bonded as described during experimental investigation. The bonded cables were included in the stiffness matrix and perfect connection with concrete was assumed. The unbonded cables were not included to the stiffness matrix and their strain was dependent on the deformation of the girder and the value of friction parameter. A special procedure was developed to model the behavior of partially bonded cables.

4. RESULTS

Load-deflection curves for girder V2 are shown in Fig.4. Girders V1 and V2 were analyzed also by Kalný and Kvasnička [6] using nonlinear 2D plane strain elements for concrete and beam elements for prestressing cables. Girders L1 and L2 satisfied ultimate load requirements as confirmed by experimental tests as well as numerical analyses. Deterioration of anchor regions was not acceptable for further serviceability of the girders.

5. CONCLUSIONS

Experimental as well as numerical analysis confirmed expected general assumptions. Numerical results were in a good agreement with experimental results and the influence of the most important factors investigated showed an assumed behavior. It was confirmed that numerical models suitable and verified for the analysis of prestressed girders can also be adopted in the process of evaluation of dismantled structures after a long service life in case of taking into account the above factors. These numerical models can, under certain circumstances, replace or reduce expensive experimental load tests.

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Rating Concrete Slab Bridges

Appréciation des ponts-dalles en béton

Einstufung von Betonplattenbrücken

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SUMMARY

Extensive experimental and analytical investigations have been conducted on the behaviour of concrete slab bridges, especially older slab bridges designed for AASHTO H15 truck loads. Experiments included testing 12 slab bridges, with and without skew, under elastic loads using truck loads, and testing to collapse one five-span continuous slab bridge. The analytical phase of the investigation included development of a three dimensional modelling technique, identifying reasons for observed large reserve strength in concrete slab bridges using the yield line analysis approach, and undertaking development of a reliability-based procedure to rate concrete slab bridges accurately.

RÉSUMÉ

Le comportement structural d'anciens ponts-dalles, dimensionnés pour une circulation (selon AASHTO, camions H15) a été étudiée de manière approfondie. Les essais ont porté sur 12 ponts droits et biais, sous sollicitations élastiques provoquées par le passage de camions, ainsi que sur un pont-dalle à cinq travées continues soumis à un chargement jusqu'à rupture. L'analyse fait appel à une technique de modélisation tridimensionnelle, mise au point pour identifier les causes des importantes réserves de résistance observées dans les ponts-dalles en béton, et pour définir les données d'évaluation précise de ces ponts-dalles à partir de la théorie de fiabilité.

ZUSAMMENFASSUNG

Zum besseren Verständnis des Tragverhaltens älterer Plattenbrücken, die für H15-Lastwagen (gem. AASHTO) bemessen waren, wurden ausgiebige experimentelle und analytische Untersuchungen durchgeführt. Der experimentelle Teil umfasste 12 Brücken, gerade und schiefwinklige, im elastischen Beanspruchungsbereich, sowie den Test einer über fünf Felder durchlaufenden Plattenbrücke bis zum Bruch. Im analytischen Teil wurde eine 3-D-Modellierungstechnik entwickelt, die Ursachen für die grossen beobachteten Tragreserven mit Hilfe des Bruchlinienverfahrens ermittelt und Ansätze zur genauen Beurteilung von Plattenbrücken aufgrund der Zuverlässigkeitstheorie unternommen.

Introduction

Many states own concrete slab bridges which were designed and constructed more than 40 years ago. Although their performance over the years has been exceptionally good, current rating procedures indicate that these bridges do not possess sufficient capacity to carry modern traffic loads. Comprehensive analytical and experimental investigations were carried out to develop simple models and a methodology for accurately assessing the load carrying capacity of these bridges at both service and ultimate load levels. To assess the performance of concrete slab bridges under normal traffic loads, a series of slab bridges representing different configurations was selected and tested under truck loads at both crawling and high speeds, hereafter referred to as *Service Load Tests*. The results were then utilized to develop simple models to predict response of concrete slab bridges to truck loads.

To assess the performance of these bridges at higher load levels and establish the available true factors of safety, a decommissioned concrete slab bridge built in 1938 was tested to collapse, hereafter referred to as *Ultimate Load Tests*. This particular bridge consisted of five spans, three continuous spans and two simply supported spans at either end. The end span of the continuous portion and one of the simply supported spans were subjected to numerous tests, including ultimate load tests, to collapse. Results indicate that these bridges possess large reserve capacities. The behavior of the bridge during the ultimate load test was very ductile and exhibited more than 12.7 cm of displacement before collapse (span length of 9.2 m).

Following these studies an investigation was then conducted to establish a reliability based procedure for rating concrete slab bridges using field testing.

The following sections present brief summaries of each phase of the investigation and conclusions. More detailed information is provided in Ref. 1.

Service Load Tests

A total of 12 concrete slab bridges representing different geometry and structural configurations were selected for truck load tests. All 12 bridges were three span continuous. Table 1 gives a list span lengths, skew, slab thickness and year of construction for each of these bridges.

Loading of each bridge was accomplished by 1) using two trucks traveling side by side or 2) one truck on the bridge at a time. The weight of each truck used for testing bridges No. 1 through 6 was approximately 222.5 kN. The weight of each truck used in testing bridges 7 through 12 was approximately 170 kN. Table 1 lists the maximum observed deflection at mid span of the middle span for all 12 bridges tested. In general, the observed deflections were small.

Results of these tests were used to develop guidelines for elastic, three-dimensional finite element analysis of three-span continuous slab bridges with minimal input, and provide comparisons between two and three dimensional analyses approaches.

Finite Element Analysis

A preprocessor was developed requiring minimal input consisting only of span lengths, thickness at different regions of the slab, cross sectional properties of the railing system (if desired), and material properties. Using this minimal input, the preprocessor would then generate the necessary information to conduct three dimensional analysis of the bridge using the SAP90 program. In this modeling technique, the slab portion of the bridge is modeled using 780 four-node shell elements that are a combination of membrane and plate-bending elements. A three dimensional prismatic beam column element, which includes effects of biaxial bending, torsion and axial and shear deformations, is used to model the railing system. Table 2 gives comparisons of the results of three dimensional analyses of bridges No. 1 through 6 tested under elastic loads in terms of the maximum deflection at center of the mid span. Also shown in Table 2 are comparisons of the maximum moments as obtained from two and three dimensional analyses approaches. The three dimensional analyses were conducted using the package developed. The two dimensional analyses were conducted using the distribution factor, E, implied by the AASHTO manual (E=1.22+0.06 x S, where S is the span length and with the limitation that E should not exceed 2.13 m). As noted from Table 2, good correlations are obtained between test and three dimensional analyses results. Further, Table 2 indicates that the maximum moment from two dimensional analyses is considerably larger than that obtained from the three dimensional analyses approach. Oftentimes, such a reduction in maximum moment using three dimensional analysis is sufficient to upgrade the load carrying capacity of the bridge.

Ultimate Load Test

A reinforced concrete slab bridge built in 1938 and located in northwestern Nebraska was used for the ultimate load test. Figure 1 shows the bridge dimensions. The superstructure is a cast-in-place five span reinforced concrete slab comprised of three continuous spans and two simply supported end spans. The abutments were constructed integrally with the superstructure. The piers were made of reinforced concrete and supported on driven steel H piles. The curb on each side of the roadway was cast monolithically with the slab.

A number of core samples taken from the bridge indicated average compressive strength of 22 MPa for the concrete. The bridge had been decommissioned since 1972 and no maintenance had been performed. As a result, an extensive amount of damage in the form of delamination of cover concrete at many locations over the slab surface was visible. The petrographic study indicated that most of the damage could be attributed to cyclic freeze-thaw damage and inadequately air entrained concrete. A number of steel samples taken from the bridge indicated that the mechanical properties of the reinforcing bars met the ASTM A616 Grade 50 standards, whereas the specified yield strength was only 70% of the actual values.

In accordance with AASHTO requirements, initial rating analysis of the bridge indicated a rating factor of 0.67 for HS20-44 truck loads, i.e. the maximum truck load permitted to cross the bridge was 67% of the weight of an HS20-44 truck (320 kN).

Using a specially built reaction frame around the structure, one of the end spans of the continuous portion of the bridge (hereafter referred to as continuous span) and one of the simple spans (hereafter referred to as simple span) was loaded to collapse. The loading of the continuous span was accomplished using 8 hydraulic rams positioned on the span to simulate two HS20 trucks side by side and having the front axle already off the span (i.e. only two axles of each truck remaining on the span). The locations of the rams were determined from detailed linear and non-linear analysis(see Ref. 1 for more detail). Figure 2 give the total applied load versus the deflection response of the continuous span at approximately the mid span. According to AASHTO rating procedures, the maximum truck weight permitted on this bridge would be approximately 214 kN. However, as shown in Fig. 2 the total applied load was approximately 3980 kN (that is, the equivalent of having two rear axles of 7 HS20-44 AASHTO trucks side by side on the bridge).

Figure 3 shows the total applied load versus the deflection response of the simple span measured at mid span. For the simple span the maximum truck load permitted by AASHTO and that obtained from tests were approximately 224 kN and 2220 kN, respectively.

Following the ultimate load tests a series of non-linear analyses were conducted [1] to investigate the reasons for the observed large reserve strength for the two spans tested to collapse. Results of these analyses indicated that the large ultimate strength of concrete slab bridges could be attributed to (in order of importance) a) actual material properties of the reinforcing bars versus the assumed values in current rating procedures, b) influence of non structural members such as railing systems and c) strain hardening of reinforcing bars.

Reliability based Rating Procedure

As discussed briefly above, the results of experimental and analytical investigations indicated that concrete slab bridges possess large reserve strength which is not reflected by current rating provisions outlined by AASHTO. In this section, the general outline of a reliability based procedure currently being developed is provided.

Service load test results indicate that the three dimensional analyses could very accurately predict the load effect while the ultimate load test assisted in identifying the reasons for large available reserve capacity in concrete slab bridges, which is due primarily to the difference between assumed material properties used in the rating process and actual values. On the other hand, the parameters involved in rating could be divided into two general categories a) load effect and b) strength, such as flexural and shear capacities.

Load effects such as maximum moment could be determined using simulation. In this approach, the possible truck combinations (referred to as events) are assumed. Figure 4 shows six events considered. The frequency distribution for weight and axle spacing of trucks are obtained from available data. A weighting factor is then assigned to reflect the probability of occurrence of each event. The distribution for the headway, h, is assumed to be uniform. Maximum moment due to each event is calculated using the influence surfaces which is three dimensional. As a result, the distribution factor as currently used in the rating process is completely eliminated. Using the Monte Carlo Simulation (MCS), the frequency distribution for maximum positive and negative moments within the slab is then obtained. The development of the frequency distribution for flexural capacities of the concrete slab reflect actual material properties. Having the frequency distribution for both the load effect and resistance, the rating factor is then established. The procedure described above could be conducted without resorting to field testing of bridges. However, a version of this method is under consideration whereby the lower end of the frequency distribution for resistance is truncated using information gained from field testing. In this approach, a known load could be applied to the bridge. If the bridge exhibits satisfactory behavior under this load, then one has proven that flexural and shear capacities of the bridge could not be less than a certain value, therefore justifying truncation of the lower end of the frequency distribution. Consequently, the rating factor could be improved.

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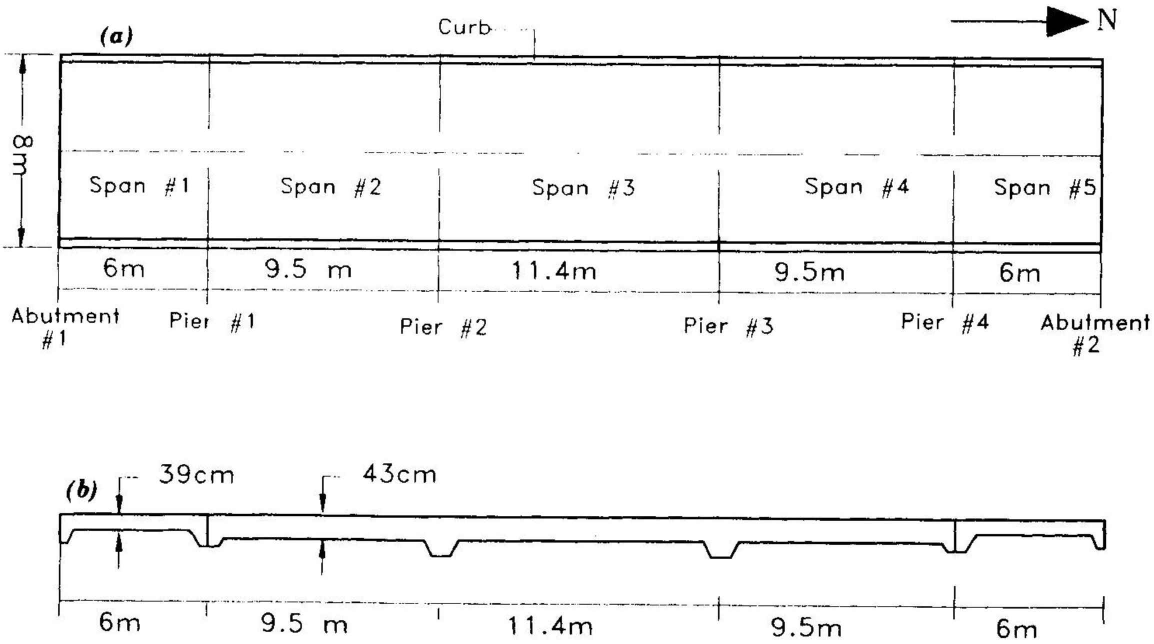

Fig. 1 - Dimensions of the bridge tested to collapse

Fig. 2 Load deflection response of the continuous span during the ultimate load test

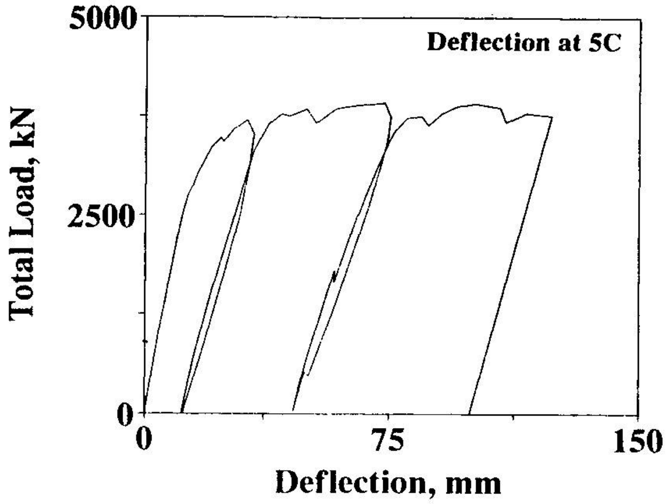

Fig. 3 Load deflection response of the simple span during the ultimate load test

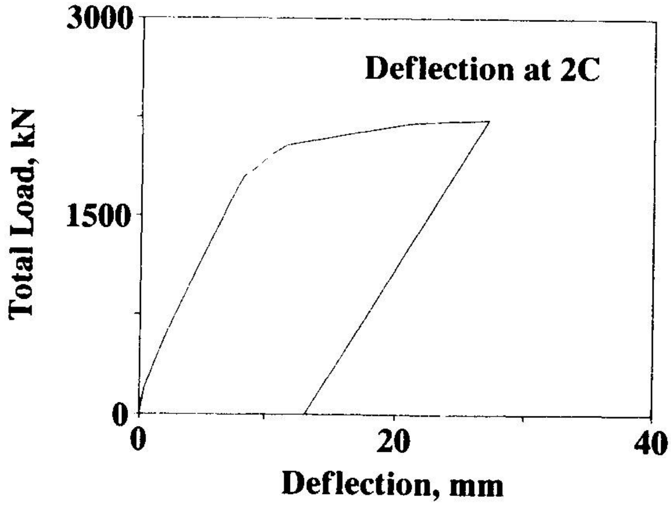

Fig. 4 Possible truck events considered in simulation process

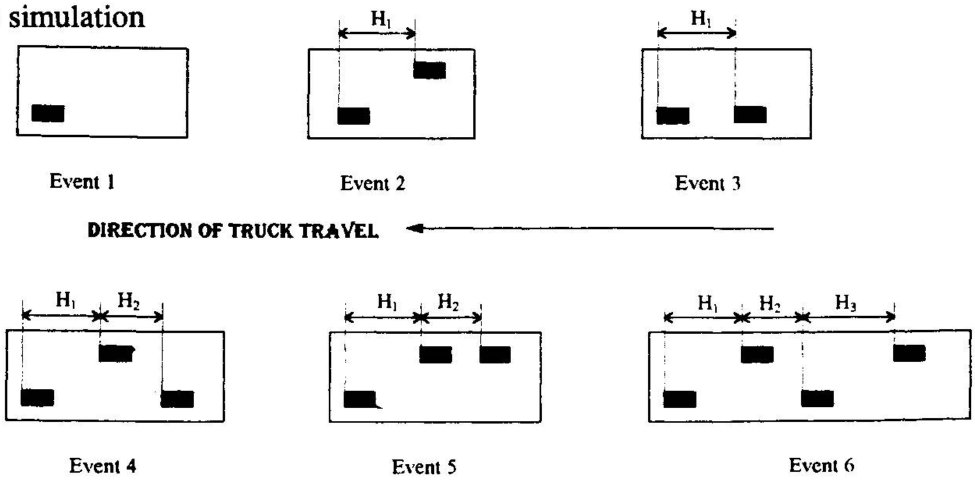

Table 1- Concrete Slab Bridges tested under elastic loads

Bridge Number	Length of First Span, m	Length of Middle Span, m	Length of Third Span, m	Skew Degree	Slab Thickness, cm	Year Constructed	Maximum Deflection, mm
1	9.6	12.8	9.6	30	39.4	1985	1.5
2	9.6	12.8	9.6	0	39.4	1985	1.6
3	6.6	9.0	6.6	0	33.0	1941	1.8
4	7.9	11.0	7.9	0	43.2	1967	2.0
5	9.1	12.3	9.1	0	40.6	1946	2.0
6	5.4	7.5	5.4	0	27.9	1965	0.6
7	5.3	7.3	5.3	30	31.8		0.1
8	8.4	10.8	8.4	10	35.6		0.4
9	11.0	14.6	11.0	25	43.2		0.7
10	5.8	7.9	5.8	35	25.4		0.7
11	5.4	7.5	5.4	30	29.2		0.5
12	9.1	12.2	9.1	50	38.1		0.5

Table 2- Comparison of the test and finite element analysis results

Bridge No.	Maximum Deflection at Mid Span, mm		Maximum Moment (m-kN/m)	
	Test	Three Dimensional Analysis	Three Dimensional Analysis	Two Dimensional Analysis
1	1.5	2.1	36.8	73.1
2	1.5	2.7	41.8	73.1
3	1.8	1.9	35.4	48.4
4	2.0	1.8	44.7	50.1
5	2.0	2.1	46.7	66.5
6	2.0	1.0	25.4	37.8

High-Strength Concrete Bridge Design: a Contribution to Sustainable Development?

Projet de ponts en béton à haute résistance: une contribution à un développement durable?

Bemessung von Brücken aus hochfestem Beton: Ein Beitrag zu nachhaltigem Bauen?

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SUMMARY

Nowadays more and more attention is being paid to sustainability. It is, however, difficult to assess the contribution of the different alternatives to the sustainability of concrete structures. A preliminary design was made for lightweight concrete, normal concrete and high strength concrete. The total energy content of the box girder was calculated for a concrete box girder bridge. Due to the small differences in energy content between the alternatives in relation to the poor reliability of input data, a conclusion on the sustainability of high strength concrete in bridge design could not yet be drawn.

RÉSUMÉ

De nos jours, les projets semblent être établis avec un plus grand respect pour l'environnement. Il reste cependant difficile d'évaluer les différentes options et leur impact sur l'environnement. Un dimensionnement préliminaire a été fait pour un pont en caisson en béton léger, normal et à haute résistance. Le contenu énergétique total dans le caisson a été calculé. La faible différence de contenu énergétique entre le béton normal et le béton à haute résistance, ainsi que la mauvaise qualité des données ne permet encore aucune conclusion à propos de ce projet.

ZUSAMMENFASSUNG

Heutzutage hat man für die Nachhaltigkeit immer mehr Interesse. Altenative Projekte von Betonbauwerken sind aber in Bezug auf die Nachhaltigkeit schwierig zu beurteilen. Für eine Kastenträgerbrücke ist ein Vorentwurf für Leichtbeton, Normalbeton und hochfestem Beton gemacht worden. Der gesamte Energieverbrauch für die Kastenträgerbrücke ist berechnet worden. Der Unterschied zwischen den Alternativen ist klein, insbesondere, wenn man die geringe Zuverlässigkeit der Ausgangsdaten berücksichtigt. Eine Aussage ob hochfester Beton zu nachhaltigem Bauen einen Beitrag leistet ist deshalb noch nicht machbar.

1. INTRODUCTION

The world at loan from our children, rather than owning it for our own benefits. This thought lies at the basis of much effort to realise sustainable development. With the aim to preserve the environment as much as possible, goals where formulated in the Netherlands on a national level in the "National Environmental Policy Plan" [1]. In this plan three main lines can be distinguished: Integrated life cycle management, Energy conservation and Quality improvement.

At Rijkswaterstaat (the directorate general for public works and water management) in the Netherlands it is felt as a dedication to translate the goals for sustainable development to reality in common practice. So, for the design of the so-called "Second Stichtse Bridge", three alternatives where considered. A preliminary design was made for respectively normalweight concrete (grade B65; cube characteristic compressive strength is 65 MPa), a lightweight concrete (grade B45) and a high strength concrete (grade B85). Apart form various structural considerations, the contribution of the alternatives to sustainability was investigated. In this respect particularly the energy-content of the concrete box girder bridge was compared for the three alternatives.

As far as environmental friendly design is concerned, it must be mentioned that there are various criteria. So it is possible that an alternative has a positive contribution to one criterion, while for another environmental criterion it has a negative effect. In this paper, first some general remarks will be given about sustainable design. Thereafter a short description of the project "Second Stichtse Bridge" is given, followed by a discussion about a possible contribution to a sustainable development by using high strength concrete for concrete box-girder bridges. Then the aspect energy-content is further elaborated for the three alternatives. For the sake of clarity it should be mentioned that sustainable development so far is seldom taken into account in the design process, where economics and performance aspects still play the major role. Nevertheless, it is hoped to show that with the contribution in this paper also considerations about environmental aspects can be incorporated in the design process.

2. OPTIONS FOR AN ENVIRONMENTAL-FRIENDLY DESIGN

2.1 General

Environmental aspects in the design process can pursue different environmental objects. Arbitrarily the following options can be distinguished:
a) design for a long functional life
b) use of fewer raw materials
c) design for recycling and re-use
d) minimizing disturbance of the surrounding
e) minimizing construction and demolition waste

Design requirements can be formulated for the different options. Here, only some examples will be given. Design requirements for a long functional life (a) are for instance: flexibility, easiness to repair, use of durable materials. Slender structures and demountable structures are respectively examples of the items (b) and (c).

The different options for an environmentally friendly design can be conflicting. For instance, a continuous beam over a number of supports is preferable from the point of view of a fewer use of raw materials (slenderness). However, for a design on recycling (of elements) or easy demolishing a row of simply supported beams can be preferred.

2.2 Bridge design

Fortunately, in many cases an economic design goes together with environmental-friendly design. In bridge design, for instance, the use of a box girder bridge with a non-constant height is common practice. In such a structure the capacity of the raw materials is utilized optimally firstly by bringing the material in the cross-section far from the neutral axis (box). Secondly, the amount of material is minimized by adaption of the height of the box girder to the moment distribution in a longitudinal direction.

Within the concept of a concrete box girder bridge it is still possible to choose for different materials. Even for the primary constituents of the concrete there are several possibilities. In this respect the type of cement, Portland cement (PC) or blast furnace slag cement (BFSC), the amount of cement (related to the concrete grade) and the type of aggregates can be mentioned. For the latter one there are e.g. river gravel, crushed natural stone and artificial manufactured lightweight aggregates. The question now arises which of these materials are preferable from the point of view of sustainability. In the subsequent paragraphs an attempt is made to answer this question for one of the environmental aspects, namely energy conservation.

For the design of concrete box girder bridges, traffic loads, dead load and temperature loading has to be taken into account. In general, the dead load is much larger than the traffic load. So, when the weight of the bridge can be reduced it will significantly influence the required amount of prestressing.

3. PROJECT "THE SECOND STICHTSE BRIDGE"

3.1 General

To obtain a good alternative traffic connection between the northern and southern provinces in the Netherlands, national highway 27 was built in the beginning of the eighties. For the crossing over lake "Gooimeer", a box girder bridge in lightweight concrete (called Stichtse Bridge) was used. Due to increased traffic intensity the capacity of the highway has to be enlarged and the national highway has to become a motorway with separated lanes. As a result a second box girder bridge has to be built besides the first one.

For aesthetical reasons a requirement for the Second Stichtse Bridge is that its shape (in longitudinal direction as well as in height) has to be identical to the first one. It concerns a three span bridge (see Fig. 1) with maximum middle span of 160 m. The structural height at the centre of the bridge is 2.5 m. For the First Stichtse Bridge a lightweight concrete (B32.5) with a sintered expanded clay for the aggregates was used. Due to new regulations it was not possible to use the same design. Therefore, a different type of concrete has to be used. In a feasibility study a lightweight concrete B45 (LWC) with sintered fly-ash aggregate, a normal-weight concrete B65 (NC) and a high strength concrete B85 (HSC) were compared. Due to the fact that the weight of these concretes was more than the weight of the original type of lightweight concrete and taking into account that structural height was fixed, the strength class had to be increased.

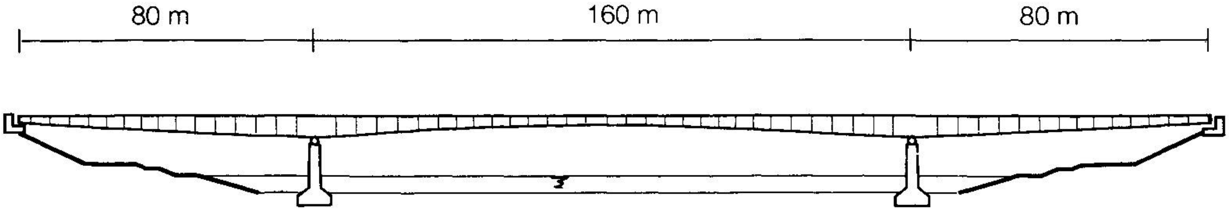

Fig. 1 Longitudinal cross-section of the "Second Stichtse Bridge"

For several reasons, of which a number are related to sustainability, it has been decided to built the bridge in high strength concrete.

3.2 Three alternatives

The preliminary design for the high strength concrete resulted in a cross-section near the support as presented in Fig. 2. The thickness of the bottom flange varies from 220 mm in the middle of the bridge to a maximum above the supports. This maximum thickness as well as the thickness of the webs can be obtained from Table 1. The thickness of the upper slab is almost equal for the three alternatives.

The amount of concrete and prestressing steel for the three alternatives is given in Table 2. The amount of mild steel reinforcement and prestressing in lateral direction is equal for all three alternatives. The weight only represents the concrete box-girder. As can be seen, the high strength concrete bridge has the lowest weight and gives a weight reduction of 14% compared to the bridge in concrete grade B65. Because the strength of HSC has already after 2 days a high value the prestressing elements can be placed with small anchorage-elements in the upper slab. Compared to the usual position of the elements in the web, this results in a simplified cable curve, giving a 8% reduction in losses of prestress. Also the reinforcement details are more simple.

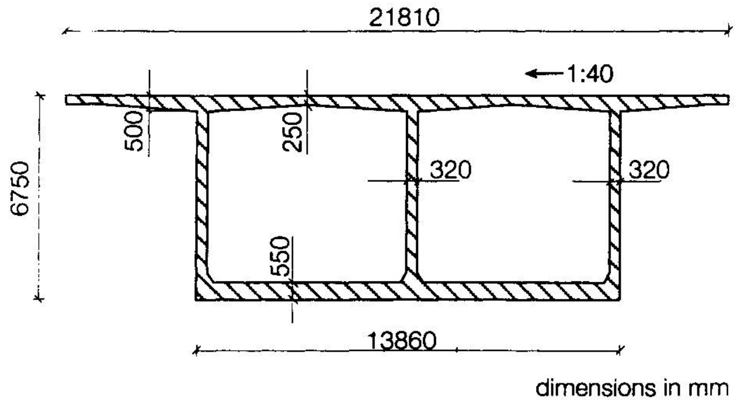

Fig. 2 Cross-section above support for B85 (HSC).

Table 1 Cross-sectional area dimensions of the box girder.

	B45 (LWC)	B65 (NC)	B85 (HSC)
varying bottom slab thickness (mm)	220 - 1000	220 - 650	220 - 550
web thickness (mm)	500	400	320

4. SUSTAINABILITY/ENERGY-CONTENT

4.1 General

As presented before the use of high strength concrete in a box-girder bridge results in a reduction of the concrete volume, number of prestressing cables and weight as compared to normalweight concrete (B65). So this gives a positive contribution to fewer use of raw materials. Furthermore, the piers and foundations of the bridge can be made less heavy due to the reduced weight of the box girder. Compared to the First Stichtse Bridge the length of the concrete segments will be enlarged from 3.4 m to 5 m, reducing the construction time with three months.

A method for comparison of products from an environmental point of view is the Life Cycle Analysis (LCA). In such an analysis all environmental impacts (e.g. depletion of energy sources, green house effect, acidification, waste production) during the total life cycle (from production of raw materials to demolition and reuse) are collected and weighed. Performing a total LCA is very laborious. Another problem is that the required information is not always available.

In the subsequent paragraphs the energy content (and thus its contribution to energy depletion) of the three bridges will be compared. In a way this can be seen as a very limited LCA. Though in case of high strength concrete the amount of concrete is less, the energy content can still be higher due to an increased cement content. Therefore it is interesting to see how the energy-balance works out for the three alternatives.

Table 2 Quantities of concrete and prestressing steel (only longitudinal direction) in the box girder and assumed concrete composition.

		B45 (LWC)	B65 (NC)	B85 (HSC)
Concrete	(m^3)	6582 (112%)	5893 (100%)	5145 (87%)
Prestressing steel	(ton)	418 (88%)	475 (100%)	380 (80%)
Weight*	(ton)	13950 (91%)	15320 (100%)	13430 (88%)
cement content PC	(kg/m^3)	90	90	238
BFSC (66.6% slag)	(kg/m^3)	270	270	237
total cement content in box girder	(ton)	2370 (112%)	2121 (100%)	2444 (115%)
type of coarse aggregate		sintered fly-ash	river gravel	crushed gravel
sand content	(kg/m^3)	750	750	750
Coarse aggregate content	(kg/m^3)	600	1050	1050

* Including lateral prestressing (97 ton) and reinforcement (600 ton). For the lightweight and normalweight concrete respectively a specific weight of 1.95 and 2.4 ton/m^3 is taken.

4.2 Energy-content of raw materials

Before presenting data for the energy content of the raw materials some comments have to be made. It is only intended to make a rough comparison of the energy content, rather then to find exact values. The values are taken from the literature, which sometimes originate from the beginning of the eighties. It may be possible that due to new techniques the energy consumption for a certain process is smaller nowadays.

Values for the (primary) energy content of the constituent materials of concrete and reinforcing and prestressing steel are presented in Table 3. As can be seen the energy content of sand and (crushed) gravel is very small as compared to the energy content of cement. If, however, lightweight aggregate is used, its contribution to the total energy content of concrete cannot be neglected. For an expanded clay a value of 3416 MJ/ton is given in [2]. The value for the sintered fly-ash aggregate in Table 3 is deduced from information by the manufacturer of the aggregate. Though the energy-content is significantly higher for sintered fly-ash aggregates than for gravel, it should be mentioned that the manufacturing of this type of aggregate helps to solve a waste problem.

4.3 Comparison of energy-content

In order to calculate the energy content of the bridge deck it is necessary to know the concrete composition. In the study the mixes were not yet defined in detail. For the design it is sufficient to know the type of concrete and the strength class. Given a certain strength it is still possible to vary the cement content or cement type, which influences the energy content of the concrete. The concrete compositions as assumed in this study can be found in Table 2. The total amount of cement in the bridge deck is highest for HSC, despite the lowest concrete volume.

Based on the information as presented before, a total energy content for the bridge deck can be calculated (see Table 4). It appears that the alternative in high strength concrete has the lowest energy content. However, due to the poor reliability of the input data, the influence of the assumed mix proportions on the energy-content and the fact that not everything has been taking into account (e.g. an increased mixing time in case of HSC), the differences in total energy content cannot be regarded as significant.

Table 3 Data for the (primary) energy content of concrete components, reinforcement and prestressing steel [2,3]

		reference	energy-content (MJ/ton)
Aggregates	- sand	[3]	16
	- gravel	[3]	16
	- crushed gravel	-	32[1)]
	- sintered fly-ash	[4,5][2)]	963[2)]
Cement	- Portland	[2]	4046
	- Blast-furnace slag		
	* 66.6 % slag	[2]	2590
Steel	- reinforcing steel	[2]	30000
	- prestressed	[2]	34000

1) Based on the assumption that for crushing as much energy is required as for winning and washing together.
2) According to the manufacturer [4] 7 kg oil, 10 kg coal and 42.5 kWh electricity is used to produce 1 ton sintered fly-ash aggregates. By assuming respectively 42.3 MJ/kg oil, 29.3 MJ/kg coal and 8.8 MJ/kWh electricity [5], the primary energy content for sintered fly-ash aggregates is calculated.

Table 4 Energy content (GJ) for the box girder bridge deck and the three alternatives.

	B45 (LWC)	B65 (NC)	B85 (HSC)
aggregates	3882	170	235
cement	7000	6267	8113
reinforcement	18000	18000	18000
prestressing elements			
- longitudinal	14212	16150	12920
- lateral	3298	3298	3298
total energy content (GJ)	46392 (106%)	43885 (100%)	42566 (97%)

5. CONCLUSIONS

From the point of view of sustainability there are different options in relation to the design of (concrete) structures. For the concrete box girder bridge in this study, it appears that the application of high strength concrete requires less concrete and steel, which is good for reduction of depletion of raw materials. As far as total primary energy content is concerned no conclusions can be drawn based on the performed analysis. The differences for the three alternatives are too small in relation to the reliability of the input data. Therefore, the question whether the application of high strength concrete in bridge design contributes to sustainable development cannot be answered from this point of view yet. However, HSC will result in an improved durability of the structure, so that from the point of view of 'designing for a long life' a positive contribution to sustainability is certainly made.

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Structural Evaluation of a Prestressed Concrete Bridge

Évaluation structurale d'un vieux pont en béton précontraint

Konstruktionsuntersuchung einer vorgespannten Betonbrücke

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SUMMARY

The paper presents a procedure for the structural evaluation of an existing prestressed concrete bridge. The method is based on probabilistic analysis for the calculation of the safety margin of the Serviceability and Ultimate Limit States. Bayesian techniques to update the statistical parameters of the resistance and load variables are introduced. The assessment of a continuous prestressed concrete voided slab bridge is presented to describe the general procedure and the structural performance under traffic load, including real data. The bridge was recently demolished because of planning reasons and before its demolition a large number of experiments were performed.

RÉSUMÉ

L'article présente une méthode d'évaluation structurale de ponts existants en béton armé ou précontraint. La méthode est basée sur l'utilisation d'études probabilistes pour le calcul du niveau de sécurité des états limites de service et ultimes. On introduit les techniques bayésiennes pour l'actualisation des paramètres de résistance ou de charge. On présente l'évaluation structurale du tablier d'un pont en béton précontraint pour la description du procédé général et pour obtenir le niveau de sécurité du pont pour les charges du trafic routier, en utilisant des données réelles. Avant que le pont ne soit démoli, pour des raisons urbanistiques, des essais furent réalisés.

ZUSAMMENFASSUNG

Ziel dieses Artikels ist, einen Ablauf zur Konstruktionsuntersuchung einer existierenden vorgespannten Betonbrücke vorzustellen. Die Methode beruht auf der Wahrscheinlichkeitsanalyse und der Berechnung einer Sicherheitsspanne für die Gebrauchstauglichkeit und den Grenzzustand. Die Aufbereitung der Wahrscheinlichkeitsparameter für Widerstand und Belastungsvariablen erfolgt durch Bayes'sche Methoden. Die Beurteilung einer ständig vorgespannten Betonbrücke in Leichtbauweise wird vorgestellt, um den allgemei-nen Ablauf und das Verhalten unter Verkehrslast zu beschreiben. Dabei werden realitäts-treue Daten verwendet. Bevor die Brücke zerstört wurde, wurde eine grosse Anzahl von Versuchen durchgeführt.

1. INTRODUCTION

The structural capacity evaluation of existing prestressed concrete bridges requires more accurate methods than those provided by the design Codes. The use of load and resistance models and safety factors of the Codes, that have been calibrated for structural design, is not a rational method to obtain the load carrying capacity or the structural performance of existing bridges. The main reason is that reliability methods have been used in the calibration of Design Codes considering global uncertainties and data coming from different sources. In the other hand, an important part of the concrete bridge stock in developed countries have been designed using different structural verification criteria, safety factors, nominal loads, materials, etc. Most of these bridges will be calified as deficient using current Standards.

The direct application of **reliability methods** provides a consistent procedure for this purpose taking into account the geometrical, material and load uncertainties for each case of study. The more relevant parameters can be updated using data coming from experimental test (load test, concrete cores, steel bars specimens), traffic measurements, etc. These data reduce the uncertainties in the evaluation. As a consequence, more efficient and realistic structural evaluation can be performed.

2. STRUCTURAL EVALUATION PROCEDURE

Reliability method provides tools to obtain a rational measure of the safety level in existing structures. The more accepted safety measure is the Reliability Index (β) that is generally defined as a function of the probability of failure (P_f) [1]:

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

Φ^{-1} = Inverse Standard Normal probability density function.

The reliability level accepted in the evaluation should be the same as the accepted values for the design of new bridges. Current Codes in developed countries have been calibrated considering a maximum probability of failure between 10^{-4} and 10^{-6} in the lifetime [1][2]. These values are equivalent to a Reliability Index between β=3,8 and 5.

2.1 General Procedure

The structural capacity is evaluated based on Ultimate Limit State formulation. The failure function is formulated as the following expression:

$$R - S = 0 \tag{2}$$

Where: R = Structural Response (Resistance)
S = Load Effects

R and S are modelled as random variables to obtain the Reliability Index, using data coming from inspections, tests, traffic measurements, etc. (Figure 1) [3].

If semiprobabilistic methods are used, design values of these variables (R_d and S_d) are compared. In that cases, design values are obtained with the nominal values and safety factors specified in the Codes [1] [4].

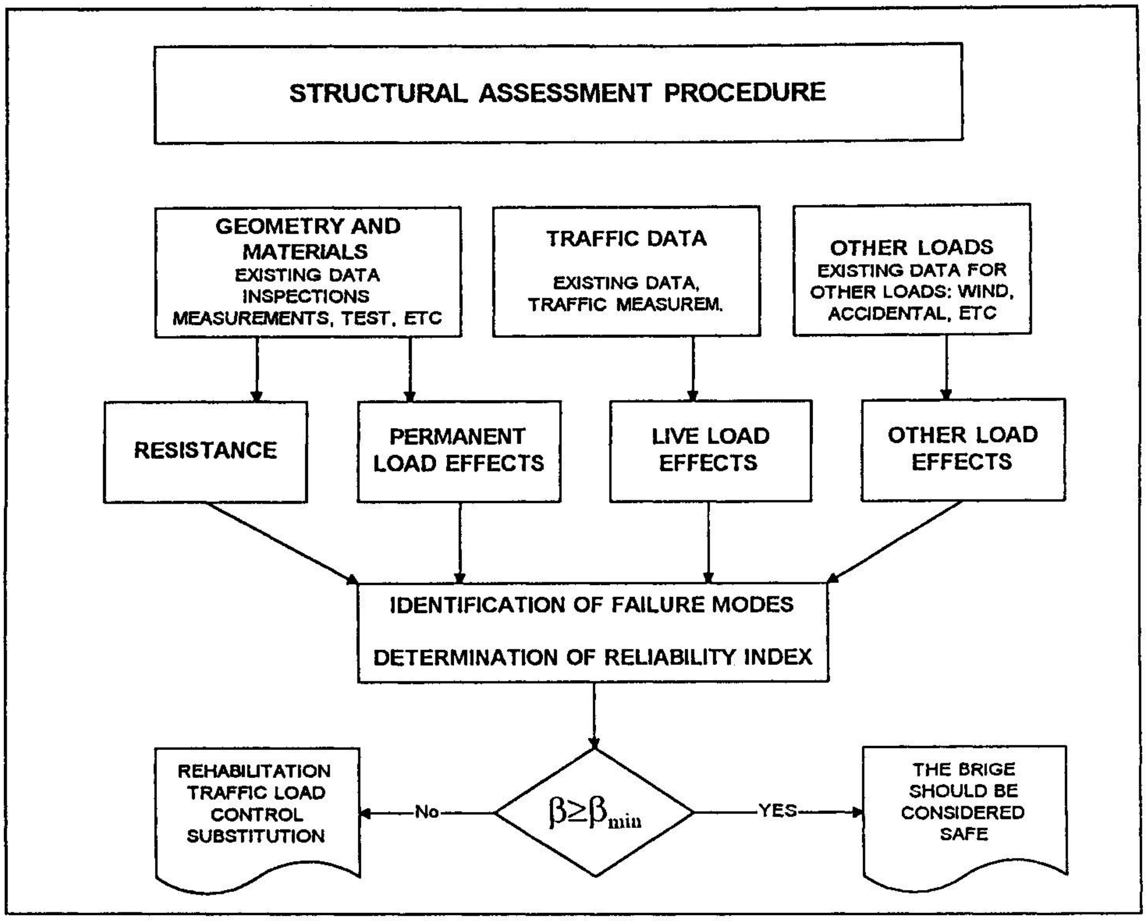

Figure 1.- General Assessment procedure

3. APPLICATION EXAMPLE

The presented structural evaluation procedure is applied for the assessment of a prestressed concrete continuous curved bridge [3]. The bridge was built in 1969 and was demolished in 1993 because of urbanistic reasons. The bridge deck has 4 spans of 17 + 21.3 + 26.6 + 21.3 m long and the radius of curvature in plant is practically constant and equal to R= 120 m. The deck is simply supported in piers, with one circular column per support axis, and two bearings in the abutments. The typical cross-section is a voided slab (Figure 2). The voids are eliminated at supports.

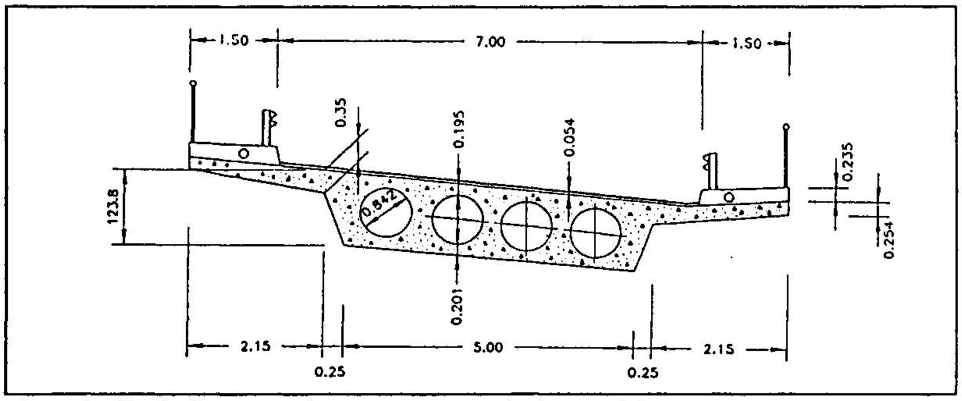

Figure 2.- Typical cross-section. The geometry is drawn with mean values.

In this paper, the ultimate flexural capacity of the bridge deck is evaluated. In the followings' steps the evaluation procedure is summarized.

3.1 Existing data, Inspection, Experimental Test. Updating Information

The more relevant information for the geometrical and material property's identification has been collected coming from existing drawings, results of the quality control during construction, inspections before and during demolition and some experimental test just before demolition. A complete statistical analysis was performed for the more important data. In some cases, general uncertainties were assumed coming from previous studies [3]. Finally, the statistical parameters were updated using bayesian techniques.

Geometrical parameters. The more significant results are the followings [3]:

1.- Values of the total depth of the deck are higher than those specified in drawings (H_{nom} = 1.20 m). The statistical parameters were: H_{mean} = 1.236 m and the Coefficient of Variation V_{HF} = 1,7%.
2.- The nominal diameter of voids was D_{nom}= 850 mm. The statistical parameters were: D_{mean}= 842 mm and the Coefficient of Variation V_D = 3,8%
3.- Higher covers of the top steel bars were measured. The nominal cover was R_{nom} = 30 mm. The statistical parameters were: R_{mean} among 80 and 132 mm, and the Standard deviation σ_R=15mm.
4.- Higher covers of the bottom steel bars were measured. The nominal cover was R_{nom} = 30 mm. The statistical parameters were: R_{mean} 37,3 mm, and the Standard deviation σ_R =12 mm
5.- The position of the prestressing steel was practically coincident with the expected value in the critical sections. The Standard deviation was σ_{dp} =16 mm.

Mechanical properties. The more significant results are the followings:

1.- The compressive resistance of the concrete was measured by testing 21 cores. The specified value was f_{ck} = 35 MPa and the updated parameters were: $f_{c,\,mean}$= 45,1 MPa and the Coefficient of Variation V_{fc} = 11,2%.
2.- The resistance of the reinforcing bars was measured in 8 tests. The specified value of the yield stress was f_{yk} = 400 MPa and the updated statistical parameters were: $f_{y,\,mean}$= 438 MPa and the Coefficient of Variation V_{fy} = 5,2%.
3.- The resistance of the prestressing steel was measured in 4 tests. The specified value of the yield stress was f_{ypk} = 1500 MPa and the updated statistical parameters were: $f_{yp,\,mean}$= 1459 MPa and the Coefficient of Variation V_{fyp} = 2,4 %.

The updated values were obtained using bayesian techniques and assuming usual values of uncertainties observed in a large data bank collected in Spain [3].

3.2 Cross-Sectional response

The ultimate flexural capacity (M_u) of the critical cross-sections has been evaluated in a probabilistic manner using updated geometrical and mechanical properties data. The responses have been obtained using Monte-Carlo techniques including the uncertainties in geometry of the cross-section (depths, voids, widths, etc.), position of reinforcing bars and prestressing steel, uncertainties in resistance of concrete and steels and, finally, including the model uncertainty (Figure 3). The method to obtain the flexural response takes into account the non-linear behaviour of materials and is according to Model Code of the CEB recommendations [4]. The partial results are not included due to the lack of space.

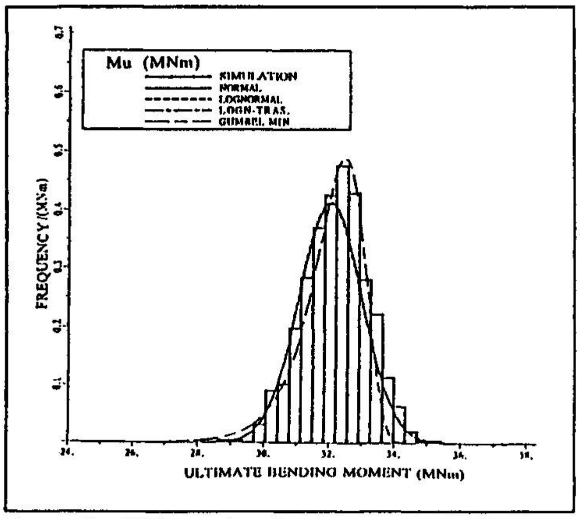

Figure 3.- Ultimate bending moment at mid-span 3 obtained by simulation.

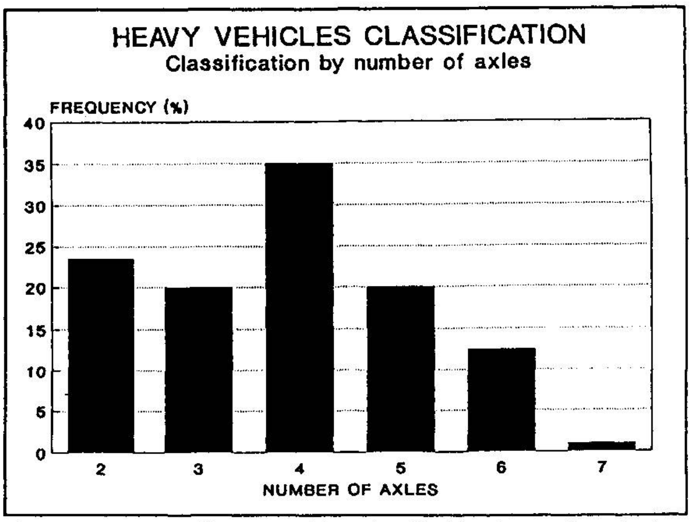

Figure 4.- Heavy traffic composition classified by the number of axles.

3.3 Load effects evaluation

The load effects due to permanent actions have been obtained considering real geometry and depth of pavement. The statistical parameters have been calculated using Monte-Carlo techniques. On the same way, using traffic load data of some highways in Spain and simulation techniques has been possible to obtain the live load effects in different traffic situations (fluid traffic and traffic jams). The simulation program developed for this purpose has been checked with the results of other authors and similar traffic situations [3]. In that case, the traffic configuration is according to Figure 4. With the obtained results has been possible to develop a simplified load model (equivalent uniform and axle tandem loads) that have been used in the non-linear analysis of the structure until failure. Due to the lack of space the partial results are not presented.

3.4 Failure mode identification. Global Structural Analysis.

The safety of the bridge deck has been evaluated for 5 different failure modes that are illustrated in Figure 5. In addition, the ultimate flexural capacity of the bridge deck has been evaluated for 3 different structural analysis (elastic, plastic and non-linear). The failure functions and the mathematical procedure for the three different structural analyses were presented in [3] [5].

3.5 Reliability analysis

The safety level is expressed in terms of the Reliability Index, as defined in section 1. The value of β, using Hasofer-Lind definition, has been obtained with the FORM method [1]. The more relevant results are summarized in Table 1 for a time reference period of 50 years.

FAILURE MODE	β ELASTIC ANALYSIS	β PLASTIC ANALYSIS	β NON-LINEAR ANALYSIS
MODE 1	8.9	10.4	8.4
MODE 2	7.2	7.9	7.8
MODE 3	7.9	9.1	8.6
MODE 4	9.5	10.9	8.4
MODE 5	9.4	10.9	9.1

Table 1.- Reliability Index for different modes of failure and the 3 structural analysis for a reference period of 50 years.

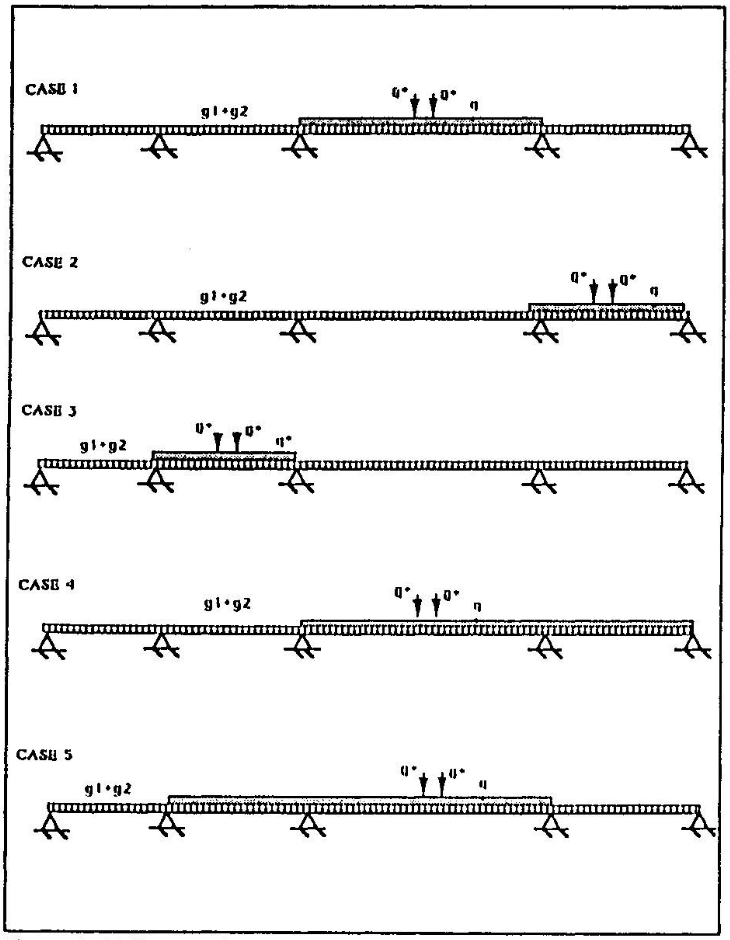

Figure 5.- Failure modes considered in the assessment of the deck

In the evaluation of both the load effects and the structural response model uncertainties have been included using models accepted in the calibration of some modern Codes. As a conclusion, the Reliability Index is depending on the structural analysis and, in some cases, the elastic analysis is not conservative because it can not predict the real critical cross-section or the exact mode of failure. The bridge deck should be calified safe (β=7.8) for the traffic loads in the considered period of time.

4. CONCLUSIONS

The Reliability Index is the more convenient measure of the safety of existing bridges. This parameter can be obtained using data coming from inspections, test, traffic measurements, etc. In some cases general uncertainties can be considered if data is not available. The paper presents some real data that can be useful in other similar cases.

5. ACKNOWLEDGEMENTS

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System Behaviour in Masonry Arch Bridges

Comportement des ponts à arc en pierre

Über das Tragverhalten gemauerter Bogenbrücken

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SUMMARY

The paper discusses the various interactions which are present in masonry arch road bridges. These occur between the wheel and the pavement, the pavement and the soil fill, distribution in the fill itself, interaction between the fill and the arch producing active, at-rest and live load pressures at different points, load distribution within the arch barrel itself, interaction between the arch barrel and the spandrel walls and between the complete structure and its foundations.

RÉSUMÉ

L'article examine les interactions diverses qui caractérisent les ponts routiers à arc réalisés en pierre. Celles-ci se produisent entre la roue et le revêtement de la chaussée; entre la chaussée et le remblayage; dans le remblai lui-même; il y a aussi l'interaction entre le remblai et l'arche produisant des pressions de poids actifs, statiques et dynamiques à des points différents, la distribution de la masse avec le berceau de la voûte, l'interaction entre le berceau de la voûte et le tympan du pont et entre la structure entière et ses fondations.

ZUSAMMENFASSUNG

Der Beitrag behandelt die verschiedenen Interaktionen, die in Bogenbrücken auftreten. Interaktionen finden zwischen den Radlasten und dem Fahrbahnbelag, zwischen dem Fahrbahnbelag und der Erdaufschüttung, der Aufschüttung und dem Tonnengewölbe, dem Gewölbe und den Seitenwänden, dem System von Gewölbe und Seitenwänden und den Widerlagern und zwischen Widerlagern und Fundament statt. Die Lastverteilung in der Aufschüttung ist ebenfalls von Bedeutung, da sie die Lastverteilung auf das Tonnengewölbe bestimmt.

1. INTRODUCTION

The structural behaviour of arches has been a matter of concern to scientists and engineers ever since the blossoming of science in the Renaissance. The development through Hooke's line of thrust and Couplet's mechanism analysis to Castigliano's elastic analysis of arch rings, described so thoroughly by Heyman[1], has concentrated the minds of engineers on the arch itself, whereas in most structures the arch is merely a small part of the whole. The two-dimensional view is particularly inappropriate to arch bridges; nonetheless, it still holds sway. Arch bridges are such complex structures and yet such cheap ones that the huge expense of a three-dimensional analysis, though it is now possible, is rarely, if ever, economic.

The objective of this paper is to set out the various elements of the structural system of an arch bridge, the way they interact and how we might put quantitative boundaries on the interactions which we either cannot, or cannot afford, to explore thoroughly.

2. THE TWO-DIMENSIONAL SYSTEM IN ARCHES

Arches are statically indeterminate, particularly so since the interaction between the foundations and the arch have a rather greater effect on structural behaviour than do the same interactions in more modern structures. If an arch rib were a perfect fit between its foundations in the dead load state, it would behave elastically under modest live loads. If we analyse the arch elastically, we neglect a major part of its behaviour. To emphasise this quantitatively, a typical 10 metre span semi-circular masonry arch, treated as if it were an elastic medium and subject to 3mm spread of the abutment, looses all its net thrust at the crown and behaves exactly like an elastic beam. Because this is both a quantitative and a qualitative change in behaviour, it shows clearly why we can never determine, or even make a reasonable approximation to the actual state of stress, in a working arch bridge.

Soil:structure interaction is a matter of increasing interest to engineers. This is normally viewed in the light of the response of a structure to foundation movement and the response of the foundations to stress in the structure. In arch bridges, these effects are compounded by the fact that most bridges have soil fill above the arch to make the road up to formation level. The result of this is that as the arch flexes under load, the pressure applied to it by the soil varies. Measurements on large scale model bridges have shown that a patch load applied at the road surface is distributed through the fill and appears on the back of the arch as a somewhat more distributed pressure patch. Surrounding this patch, the arch is pushed away from the soil and the pressure is slightly reduced. On the far side of the span, the arch moves into the fill and this causes a passive response from the fill which helps to stabilise the arch. Figure 1 shows an arch subjected to load in this way, with the corresponding pressures, ring stresses and deformation. It should be noted here that the deformation pattern will be the same regardless of

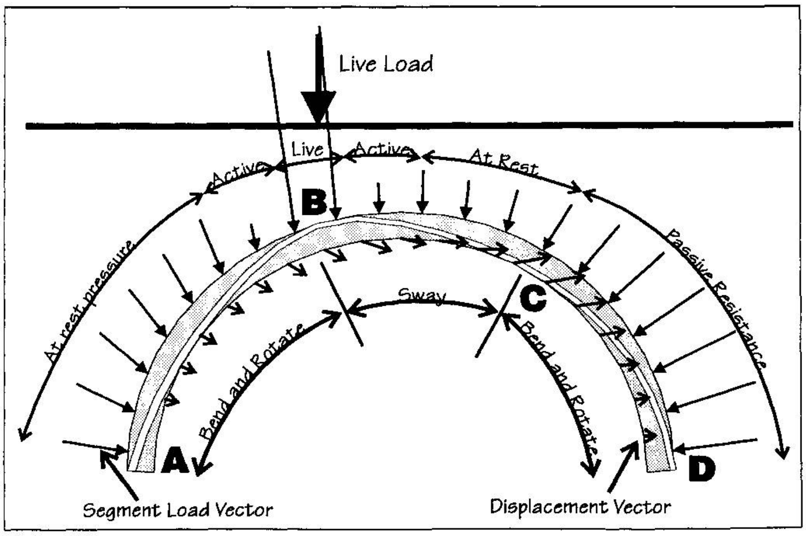

Figure 1 Arch movement and corresponding soil pressures under load

the material from which the arch is built. Even a steel ring would be pushed down at the load point and would rise into the fill at the opposite side. The fact that a masonry ring cannot carry tension reduces the stiffness in places and therefore increases the curvature.

2.1 Initial stress effects and abutment movement

Given that the thrust in an elastic 10 metre span arch can disappear with only 3mm of movement at the abutments, we must acknowledge that no real abutment can be regarded as sensibly rigid. The behaviour of the arch ring must therefore be modified by the fact that it stands on non-rigid foundations. When the centring is removed from an arch, the load is transferred from a temporary support into thrust in the arch ring and a reaction from the abutments. As a result, the ring must shorten and bend, and the abutments must move apart as well as downwards. The net effect of this, as described elsewhere[2], is to force the thrust upwards at the crown and downwards at the springings so that even before any live load is applied, the bending stiffness of the arch is likely to be reduced at these points; that is to say there will be some loss of section at the crown and the springings.

As a live load is applied, the effect is to shift the thrust and therefore the low stiffness zones as shown in Figure 2. Only as the live load approaches the ultimate value for the system described will stiffness reduce dramatically in the opposite sense at the springing remote from the load so that the arch ring tries to form a mechanism and is only held in place by the fill. At this stage, arch ring deformation may become gross and something approaching the full passive pressure may be mobilised from the soil fill before final failure of the system. It should be noted, however, that gross deformation which is required to develop passive pressure modifies the geometry of the arch, reducing the effective rise and increasing the thrust. The law of diminishing returns comes into action rapidly, so that a load deflection curve for the arch would be well into the falling branch before passive pressure is fully realised.

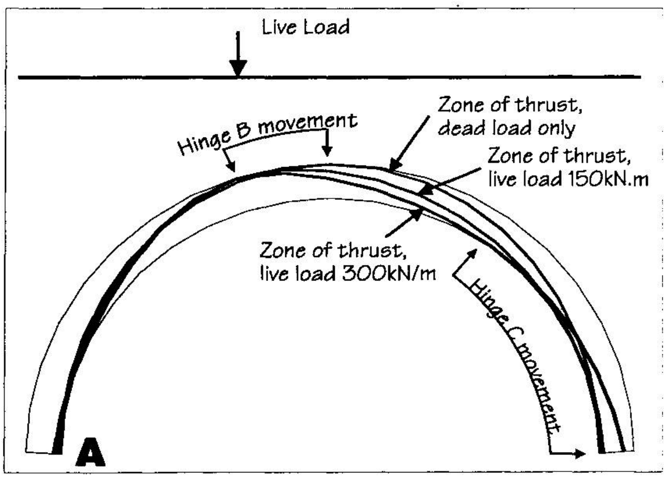

Figure 2 Movement of "hinges" as load is applied

2.2 Three dimensional effects

Models have been constructed at moderate scale with an arch ring which truly represent this two-dimensional behaviour, and all the effects described above could be observed and, indeed, measured. A three-dimensional model was constructed with spandrel walls but otherwise identical to one of these 2-D ones and the behaviour observed was substantially different. As before, where the patch load was applied, the resultant appeared as a patch load on the upper surface of the arch but the patch was much less distributed than in the previous case. The reason for this change has not yet been tested but it would seem likely to relate to an increased stiffness of the arch ring caused by the presence of spandrel walls.

The smaller patch of pressure on the back of the arch caused by the live load was surrounded by an area of reduced pressure where the arch deformed away from the fill. What was perhaps more surprising was that the passive pressure induced at the opposite side of the arch was reduced. It is thus clear that although there was negligible transverse distribution of load through the fill, the structure was managing to produce a distribution. Masonry is assumed to have no tensile strength. This is obviously something of a simplification since, in one direction at least, interlocking of the masonry units will mean the strength is rather more than that which can be mobilised in the bond between the units and the mortar. Nonetheless, it is reasonable to assume that the transverse bending stiffness of the arch is small.

How then can the structure provide lateral distribution. To understand this it is necessary to look first at the motion vectors of various parts of the arch shown in Figure 1. When this is done, it becomes clear that a major component of the deformation is in the form of sway and is much less characteristically normal to the surface of the arch than might at first appear. In order for transverse distribution not to take place it would therefore be necessary for shear deformation to take place within the membrane of the arch itself, and we would expect the arch to exhibit enormous stiffness against such deformation. The critical area in this respect is that between points B and C in the arch (Figure 1).

A look at the aspect ratio of this element of the arch will give a picture of what sort of distribution might be achieved. A typical wide bridge might carry four lanes of 3 metres on a span of 15 metres. This central swaying zone of the arch would then have an aspect ratio roughly 5 metres deep by 12 metres wide. If the spandrel walls were to provide support over roughly a metre at each edge, this section of the arch would become, in plane, an extremely deep beam, the behaviour of which would be governed by shear deformation, even if it had substantial tensile strength. Most arch bridges are 8 metres wide or less, so that the aspect ratio of the swaying section is likely to be at least 2:1 in any real bridge. This implies that sway mode edge stiffness provided by the spandrel walls will be sufficient to ensure that negligible sway actually takes place. All the passive action required to stabilise the arch comes from the walls rather than the fill. This implication is borne out by the practical measurements which have been made.

3. LOAD PATHS IN ARCH BRIDGES

Having described the stiffening mechanisms of the arch, it becomes possible to trace the load path through the structure in terms of lines of thrust. A vertical patch load applied at the load surface must be supported by a vertical reaction vertically below it from the arch barrel. The live load thrust in the arch must therefore be concentrated in a relatively small area underneath the load. From here the thrust will fan out towards both abutments, the degree of fanning will depend on the in-plane stiffness of the arch barrel and the stiffness of the structural elements and foundations supporting it.

The spandrel walls will be stiff in the plane of the arch. Whatever foundations they have as they extend back behind the abutment wall, the spandrels will have noticeably larger foundations than a similar width of abutment. Thus the spandrels must be substantially stiffer than those parts of the structure supporting the rest of the arch width.

Provided the arch does not crack at the inside face of the spandrels, it is reasonable to model the spandrels as adding a horizontal component of force to the arch thrust at point C, and similarly attracting a substantial proportion of the thrust at the springing of arch closest to the application of load, A. The potential for producing this horizontal component can be checked by considering the possibility of horizontal shear failure longitudinally in the wall. It is, however, quite unlikely that such a failure would take place in a real bridge.

4. MODELLING FOR ANALYSIS, ASSESSMENT AND DESIGN

Modelling this complex three-dimensional behaviour in such a way as to assess failure criteria remains extremely difficult. Computer software to simplify such analyses is under development in Dundee, but in the meantime we must consider whether such refinement is truly necessary, rather than just desirable.

We have seen that if an arch is built as a two-dimensional structure with no edge stiffening, it behaves in the anticipated way, generating active and passive soil pressures at appropriate points around the arch ring. If, as sometimes happens, the arch were to crack in line with the inside face of the spandrels, much of the interaction with the spandrels would break down. A detailed discussion of these possibilities is presented in reference [3].

It might be necessary, for practical purposes, to take account of only the two-dimensional structure and such lateral load distribution as the arch may be able to produce within the width of the structure. It is therefore proposed that for assessment and design purposes the arch should be analysed on a width equal to the distance between the inside face of the spandrels which may be reasonably taken as 1m inside the overall width at both faces. Such traffic load as can be applied between these boundaries should then be assumed to be distributed across that whole width. It should be noted that in many bridges it will not be possible to load more than one vehicle in this effective width. If two vehicles pass on the bridge, one wheel of each may well be positioned over a spandrel wall (Figure 3). The analyst should recognise that the effect of the wheel loads is not distributed uniformly across the full width of the arch at the point of application as is implied by this assumption. It should equally be recognised that the structure will have additional capacity as a result of the alternative load paths which are available to it, including friction between the soil fill and the spandrels. Therefore the assumptions are conservative.

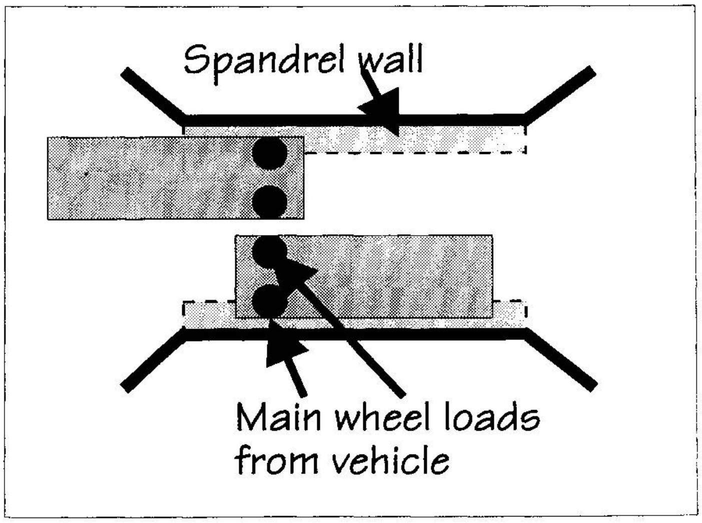

Figure 3 Vehicle loads on a typical arch bridge

5. CONCLUSIONS

1 Masonry arch bridges are complex three-dimension structures.

2 The gross difference in stiffness of various elements of the structure substantially affect its load distribution characteristics.

3 It is reasonable to assume that any point load applied is distributed across the full width of the arch. Such distribution will not be as a result of transverse bending but of tangential shear, and will take place at an angle of at least 45° to the direction of span.

4. If a detailed analysis is required, the supporting action of the spandrels can be assessed in terms of longitudinal sliding shear resistance.

5 Much further work is required before a complete three-dimensional analysis of the structure is possible.

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Evaluation of Existing Bridges under Actual Traffic

Évaluation des ponts existants sollicités par un trafic réel

Beurteilung bestehender Brücken unter wirklichem Verkehr

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SUMMARY

This paper presents a method for the evaluation of the reliability of existing bridges under the highest traffic loads and a method for the fatigue life assessment under repeated loads. A bridge classification is proposed for management. The fatigue life is assessed by a very simple calculation, taking into account the stress ranges produced by the fatigue load models defined in Eurocode 1-3 and the actual traffic on the bridge. The proposed method is also usable in the design of new bridges. An example illustrates the proposals.

RÉSUMÉ

Cet article présente une procédure d'évaluation de la sécurité des ponts existants sous l'action des charges maximales du trafic et une procédure d'évaluation de la durée de vie sous les charges répétées. Un classement des ponts est proposé pour la gestion. La durée de vie est estimée par un calcul simple qui tient compte des étendues de contrainte produites par les modèles de fatigue de l'Eurocode 1-3 et le trafic circulant réellement sur le pont. La méthode proposée est également utilisable pour l'étude de nouveaux projets de ponts. Un exemple illustre les propositions.

ZUSAMMENFASSUNG

Dieser Bericht stellt ein Verfahren für die Bestimmung der Zuverlässigkeit bestehender Brücken unter maximalen Verkehrslasten, und ein Verfahren für die Berechnung der Lebenszeit unter Verkehrslasten vor. Eine Brückenklassifizierung für die Verwaltung wird vorgeschlagen. Die Lebensdauer wird durch eine einfache Berechnung bestimmt, die die Spannungsschwingungen, erzeugt durch Ermüdungslastmodelle gemäss Eurocode 1-3 und dem wirklichen Verkehr, berücksichtigt. Der Vorschlag ist auch für die Bemessung neuer Brücken brauchbar. Ein Beispiel zeigt diesen Vorschlag.

1. INTRODUCTION.

Since 40 years, the road traffic has known a quick development, particularly in Europe. The development consists in the increase of the number of lorries, in the weight of the lorries, in the advent of tandem and tridem axles, and more recently in the increase of the percentage of loaded vehicles. The existing bridges have been designed taking into account load models corresponding, in the best situation, to the loads of lorries allowed at that time. The question of the reliability and the durability of the existing bridges arises for all bridges, but mainly if the gap between the loads of the models and the actual traffic is high. It is the case in Belgium [1]. During the development of the Eurocode 1-3 - Traffic loads on bridges - a lot of traffic loads have been recorded and used in order to define scientifically the characteristic loads and the fatigue loads [2]. Any existing Belgian bridge should satisfy usual safety factors under the loads defined in the Eurocode.

The aim of this paper is to show how it is possible to verify the ultimate limit state resistance and to estimate the fatigue life of critical details, taking into account the actual traffic on the bridge. The conclusions of such a study should either set forward the details that need particular attention during the bridge inspection because their fatigue life is short, or if necessary, define a limit of the loads allowed on the bridge.

2. DESIGN LOADS.

Since 1952 to 1993, the Belgian bridges have been designed considering the traffic loads defined in the code NBN 5, where in each lane a five axles vehicle of 320 kN (120 + 2 x 60 + 2 x 40) and a distributed load of 4 kN/m² were foreseen, these loads being multiplied by a dynamic factor never higher than 1,25 [1]. Figure 1 compares the total load Q located on a lane 3,5 meters wide and L meters long corresponding to NBN 5, to the loads given by the vehicles allowed to run in Belgium CR, to the actual vehicle loads running on European highways with a return period of one day Q_f (frequent load), and a return period of 1000 years Q_k (characteristic load) [3]. The figure shows that code loads are lower than the allowed loads, and sometimes close to $Q_f/2$ and $Q_f/3$.

The comparison of the loads located on a two lanes road on a L meters length, shows that the Eurocode loads, comprising in the first lane 2 axles of 300 kN spaced by 1,2 m. and a distributed load of 27 kN/m. and in the 2^d lane 2 axles of 200 kN spaced by 1,2 m. and of a distributed load of 2,5 kN/m², are always higher than the loads produced by a jam including 10 % of lorries.

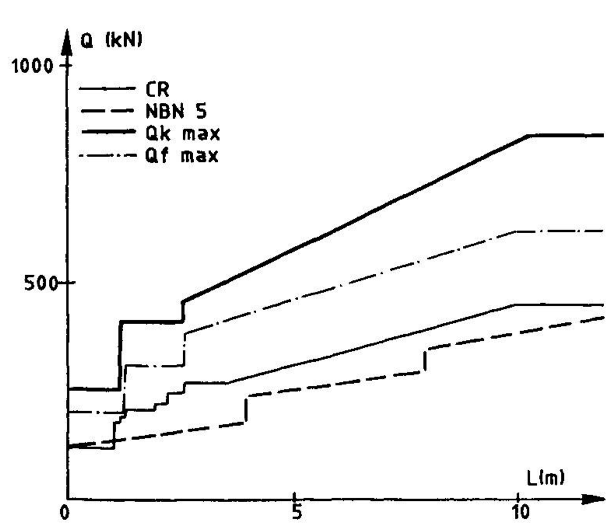

Figure 1 : Local loads.

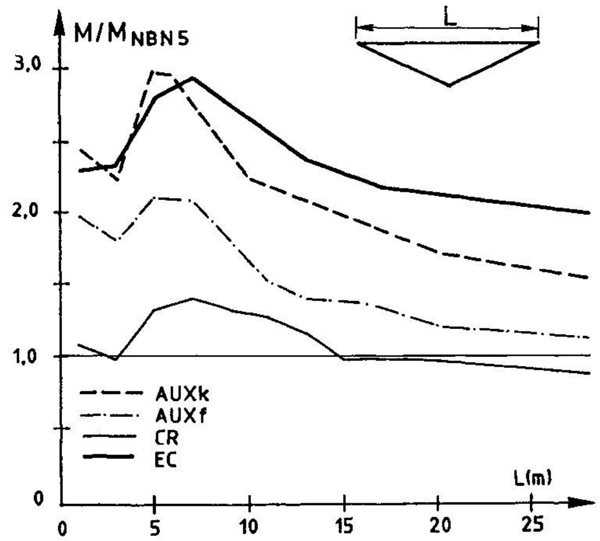

Figure 2 : Bending moment at mid-span.

Figure 2 compares the bending moment, dynamic effect included, obtained under the different loads at midspan of a simply supported beams supporting one traffic lane. The following conclusions are also valid for a lot of other influence lines [4]. The ratio of the load effect obtained by a load model and the NBN 5 loads are between, 1 and 1,4 for the allowed loads CR if L < 20 m., between 1,4 and 2,1 for the frequent loads of a traffic Auxf if L < 16 m. and between 2 and 3 for the characteristic loads of a traffic Auxk or of the Eurocode EC, if L < 25 m.

3. BELGIAN BRIDGES CLASSIFICATION.

Existing bridges have not been designed for the Eurocode loads. Nevertheless, failure produced by traffic loads are very rare up to now because a high safety factor is included in the design, so that the actual reliability of the bridges is comparable in Belgium and in the other European countries [5]. The Belgian National Application Document of EC 1-3 is now drafted to define 4 classes useful for the bridges managers :

class 1 concerns the design of new bridges, where the Eurocode loads are considered ;
class 2 concerns the repair of bridges, where the infrequent loads of Eurocode are considered (return period of 1 year : $\alpha_Q = \alpha_q = 0{,}8$) ;
class 3 concerns all existing bridges that are acceptable, where the frequent loads of Eurocode are considered (return period of 1 week : $\alpha_Q = 0{,}75$; $\alpha_q = 0{,}4$) and the safety factor γ_G and γ_Q are reduced ;
class 4 concerns the bridges where the vehicle weight allowed to run on the bridge is limited.

Table 1 : Bridge classes - N.A.D.

	1 european	2 belgian	3 acceptable	4 limited
$\alpha_{Q1} = \alpha_{Q2}$	1	0,8	0,75	0,75 C_R
α_{Q3}	0	0	0	0
α_{q1}	1	0,8	0,4	0,4 C_R
$\alpha_{q2} = \alpha_{q1} = \alpha_{qr}$	1	1	0,4	0,4 C_R
γ_G	1,35	1,35	1,1	1,1
γ_Q	1,35	1,35	1,2	1,2

A statical analysis has shown that the failure probability of a bridge of class 3 under the highway traffic is equal to 3.10^{-5} [3], value which is close to the one recommended by ISO [6]. For load effects influenced by not more than one vehicle, the frequent load should correspond to the vehicles defined in the Eurocode for the fatigue assessment FLM2. The load limitation in class 4 distinguishes between the load effects influenced by one or more than one vehicle, located in the same lane (long influence lines) or in two lanes (large influence surfaces).

4. FATIGUE LIFE ASSESSMENT.

4.1. Introduction.

The reliability of bridges satisfying class 3 is low, but acceptable. For the management of the bridges it is necessary to know the durability under the actual traffic flow. Eurocode 1-3 defines 5 fatigue load models [2]. FLM1 and FLM2 define frequent loads ; if the stress range produced by this load is below the fatigue limit no fatigue damage is expected. FLM3 and FLM4 define equivalent loads or mean loads, usable for the fatigue assessment. FLM1 is derived from the characteristic load model : $Q_f = 0{,}7\ Q_k$ for the axle loads and $q_f = 0{,}3\ q_k$ for the distributed loads. FLM2 and FLM4 define a set of 5 lorries by geometry, axle loads and frequencies, and are usable for effects produced by one lorry alone. FLM3 comprises a single symmetrical vehicle with 4 axles of 120 kN ; but we have demonstrated elsewhere the need of the second vehicle for long spans with only 30 % load and located 40 m. behind the first vehicle [3]. FLM5 considers a whole load spectrum and is used only for special cases. The fatigue life should be calculated for each detail of an existing bridge knowing

the influence area and the traffic flow. The total number of lorries crossing on the bridge should be estimated by visual counting. This counting should give the frequency of each type of vehicle. Taking into account the loads defined for FLM3 or FLM4 in EC 1-3, a fatigue assessment is possible. A more complete information of the vehicle loads requires records by a weight device, but this is only necessary if the lorry loads are very different of the loads given in the Eurocode, which correspond to a heavy long distance traffic in Europe.
The method proposed for the assessment of the fatigue life is described here for existing bridges, but it is also applicable for the fatigue verification needed in the design of new bridges. As the method considers fatigue resistance defined by SN curves, it concerns steel as well as concrete elements.

4.2. Fatigue life assessment.

The fatigue life assessment is carried out in two steps. The first step should conclude whether fatigue damages are expected or not. If fatigue damages are expected, the fatigue life is calculated.

1° Fatigue life is unlimited if $\Delta\sigma_{M1} \leq \Delta\sigma_D / \gamma_{Mf}$, where,

$\Delta\sigma_{M1}$ is the highest stress range produced by FLM1,
$\Delta\sigma_D$ is the fatigue limit under constant amplitude,
γ_{Mf} is the partial safety factor.

For short spans, i.e., for spans shorter than 30 m. if the influence line comprises areas alternatively positive and negative or for the addition of the two contiguous spans shorter than 30 m. if the sign of the areas are the same, FLM1 may always be replaced by FLM2.

2° Fatigue life is calculated.

$$\Delta\sigma_{M3} \leq \left(\frac{N_D}{N_t}\right)^{1/m} \frac{1}{C_t} \cdot \frac{\Delta\sigma_D}{\gamma_{Mf}}, \text{ where,}$$

$\Delta\sigma_{M3}$ is the highest stress range produced by the vehicle of FLM3 defined in EC 1-3 completed by a 2d vehicle located 40 m. behind [3],

N_D and $\Delta\sigma_D$ corresponds to the fatigue limit on the SN curve ($N_D = 5.10^6$ in EC3 [7]),

$$C_t = \left(\frac{\Sigma n_{ij} . Q_i^m}{\Sigma n_{ij}}\right)^{1/m} \left[\frac{\Sigma n_{i1} . Q_i^m}{\Sigma n_{i1}}\right]^{-1/m}$$ is a factor to consider for traffic with an other composition as the long distance traffic,

Q_i is the equivalent weight of vehicle i if the stress range is produced by all axles of each vehicle ; if each axle produces a stress range, each axle weight is considered successively,
n_{ij} is the frequency of vehicle in traffic j,
n_{i1} is the frequency of vehicle i in long distance traffic,
m defines the slope of the SN curve (m = 5 in EC 3 and m = 5 to 9 in EC2 [7] [8]).
N_t is the number of cycles produced by the traffic.

The value of N_t is different if one or more lanes influence the stress range :

a) If the stress range is produced by the traffic running in two slow lanes of the road and if the ratio $\Delta\sigma_{M3/2}/\Delta\sigma_{M3/1}$ between the effects of each lane is higher than 0,5, the vehicle of FLM3 is running successively in each slow lane, and

$$N_t = N_{t1}\left[1+\left(\frac{\Delta\sigma_{M3/2}}{\Delta\sigma_{M3/1}}\right)^m\right]$$

$N_{t1} = N_a . N_{obs} . k_1 . k_2$, where

N_a is the life time in years,
N_{obs} is the number of lorries per year running on one slow lane,

k_1 is the ratio of the number of loaded lorries to the total number of lorries ; for heavy traffic $k_1 = 2/3$

$k_2 = 0,6 + \frac{1}{0,25L}$ if 1,18 m. < L < 10 m., L the span length in meters,

$k_2 = 4$ if $L \leq 1,18$ m.

$k_2 = 1$ if $L \geq 10$ m.

b) If condition a) is not satisfied, only the traffic running on one lane is considered, and the traffic running in fast lanes may be neglected because it represents not more than 10 % of the traffic flow of the slow lane, $N_t = N_{t1} = N_a \cdot N_{obs} \cdot k_1 \cdot k_2$

In all cases, the life time becomes $N_a = \frac{N_{t1}}{N_{obs} \cdot k_1 \cdot k_2}$

If, for local effects, the stress range depends highly on the geometry of the vehicles, the vehicle of FLM3 is replaced successively by each vehicle of FLM4 with the appropriate frequency in order to obtain the equivalent stress range.

5. EXAMPLE.

Figure 3 shows the cross section of a common highway bridge in Belgium.

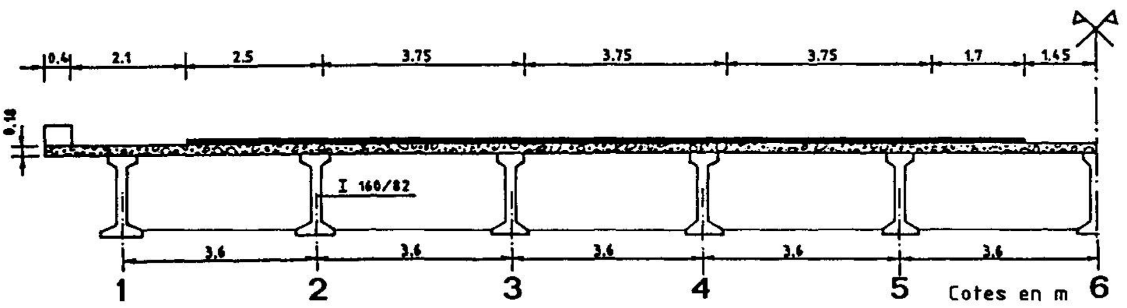

Figure 3 : Cross section of a Belgian bridge.

Table 2gives for three sections,

M_G moment under dead load,

M_Q moment under load model of NBN 5,

M_R ultimate resistance moment,

M_{Qk} moment under characteristic load of EC 3-1,

$\Delta\sigma_{M3}$ stress range in the reinforcement corresponding to FLM3 of EC 3-1,

N_{veh} number of lorries before fatigue crack.

The main reinforcement in the span satisfies class 2, the section on the support satisfies class 3, but transverse bars in span do not satisfy class 3and a reduction of the loads of the vehicles allowed to cross the bridge is required following class 4.

A more accurate analysis using FLM4 shows that 85 % of the total damage produced by a long distance traffic results from the 2^d axle of half trailer vehicles.

The fatigue life for main bending is longer than 100 years for highway traffic (40.000 lorries per week) but for transverse bending the fatigue life is very short : 4 years for main road traffic of category 3 (2.500 lorries per week). The durability of such a bridge depends on the contribution of the pavement with the concrete slab in a composite effect : each reduction of the stress range by 8 % doubles the fatigue life.

Table 2 - Moment in kN m/m.

	Section on support 2	Section in span	
		M_x	M_y
M_G	- 7,16	3,40	0
M_Q	- 47,04	31,36	-
$M_R = 1{,}5\ (M_G + M_Q)$	- 81,3	52,14	10,42
M_{Qk}	- 75,11	40,23	17,60
M_{Qf}	- 55,81	27,11	13,20
1) $1{,}35\ (M_G + M_{Qk})$	- 111,1	58,9	23,76
2) $1{,}35\ (M_G + 0{,}8\ M_{Qk})$	- 90,8	**48,0**	19,00
3) $1{,}1\ (M_G + 1{,}2\ M_{Qk})$	**- 70,7**	36,3	15,84
$\Delta\sigma_{M3}$ (N/mm²)	105	87	190
$N_{véh}$ (10^6)	170	1080	0,54

6. CONCLUSIONS.

The proposed method allows a classification of the existing bridges regarding their ultimate limite state. The very simple method for the fatigue assessment of existing bridges proposed is also usable for the fatigue verification in the design of new bridges. The example has shown the difficulties of an actual assessment of existing bridges, where the durability depends of the beneficial effect of the asphalt surfacing.

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Site Specific Traffic Load Models for Bridge Evaluation

Modèles de charge de trafic actualisés pour l'évaluation de ponts-routes

Nachgeführte Verkehrslastmodelle zur Beurteilung bestehender Brücken

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SUMMARY

This paper describes work carried out to develop a method for considering actual traffic loads during bridge evaluation. This method is based on the use of load correction factors which have been determined using a probabilistic analysis of bridge loads and resistance. Load correction factors and used in order to modify load effects calculated using a design traffic load model. This approach enables the accurate evaluation of bridges which carry a known traffic. If this traffic is less aggressive than that assumed by the design loading code, acceptable reliability may be verified for structures which are damaged or deteriorated, thus avoiding the need for strengthening or traffic restrictions.

RÉSUMÉ

Cet article décrit une méthode considérant les charges de trafic actuel pour l'évaluation de ponts existants. Elle est basée sur l'utilisation de facteurs de correction, déterminés par une analyse probabiliste des charges de trafic et de la résistance des ponts. Ces facteurs sont utilisés pour modifier l'effet des charges calculées avec des modèles de charge de trafic selon des normes de dimensionnement. Cette approche permet une évaluation précise de ponts sur lesquels circule un trafic connu. Si ce trafic est moins agressif que prévu par les normes de dimensionnement, une sécurité suffisante peut être vérifiée pour des structures endommagées, évitant ainsi des renforcements ourestrictions de trafic.

ZUSAMMENFASSUNG

Der Artikel beschreibt eine Brückenbeurteilungsmethode, welche das reelle Verkehrsaufkommen berücksichtigt. Die Methode verwendet Lastkorrekturfaktoren, die auf einer probabilistischen Untersuchung des Verkehrsaufkommens und des Tragverhaltens von Brücken basiert. Die Lastkorrekturfaktoren modifizieren die in den Lastnormen berücksichtigten Lastmodelle. Diese Annäherung ermöglicht eine genaue Beurteilung von Brücken unter bekanntem Verkehrsaufkommen. Falls das Verkehrsaufkommen kleiner als jenes der Lastnormen ist, kann die genügende Zuverlässigkeit eines beschädigten Tragwerks nachgewiesen und Verstärkungsarbeiten oder Verkehrsbeschränkungen verhindert werden.

1. INTRODUCTION

1.1 Motivation

Road traffic load models which are used for design are inherently conservative because of the high uncertainty about traffic loads at the design stage and because the models must be valid for structures of all types and sizes. The increased cost of construction due to the use of a conservative design load model is small and necessary to allow for uncertainty and to simplify the design process. However, once a structure is in service, the cost of an over-conservative evaluation could be much greater, thus justifying the use of an approach which considers actual traffic and the effects it produces in a given structure.

The most accurate way for an engineer to consider actual traffic would be to carry out a probabilistic analysis using site traffic data. However, this is a time consuming process, involving a considerable understanding of probabilistic methods, and could only be justified for the assessment of a major structure. The aim of the study described in this paper was therefore to develop a simple method for the consideration of site specific traffic loads as a function of parameters describing the bridge and traffic, referred to as site characteristics.

1.2 Approach

The proposed evaluation method uses correction factors which are applied to effects calculated using the design traffic load model in order to consider actual traffic. Figure 1 illustrates the probabilistic approach adopted for deriving these factors. This approach is based on the comparison of live load carrying capacity (R-G) and applied traffic loads (Q). An underlying criterion is that the target reliability implicit in a bridge evaluation must be equal to that implied by the existing design codes. The main stages of this approach are outlined below.

① The reliability of a structure designed using the design codes is calculated, considering an aggressive highway traffic which is taken as the traffic represented by the design loading code. The reliability index thus calculated is denoted β_{des}.

② The calculation is repeated considering an updated traffic representing the actual loading of an existing bridge. A reliability index β_{actual} is calculated, which is generally greater than β_{des} because the traffic which actually passes over a structure is less aggressive than that assumed at the design stage.

③ The aim is then to find a factor, α, by which the live load carrying capacity of the structure can be divided in order to produce a reliability index, β_{eval}, equal to β_{des} for the actual traffic.

④ The factor α could also be defined as that by which the actual traffic loading could be multiplied in order to produce a reliability β_{eval} for $(R\text{-}G)_{des}$. If we compare ① and ④ we note that $\alpha Q_{actual} \approx Q_{des}$.

⑤ We can therefore allow a live load carrying capacity which is lower than that assumed for design and still have a reliability $\beta_{eval} = \beta_{des}$.

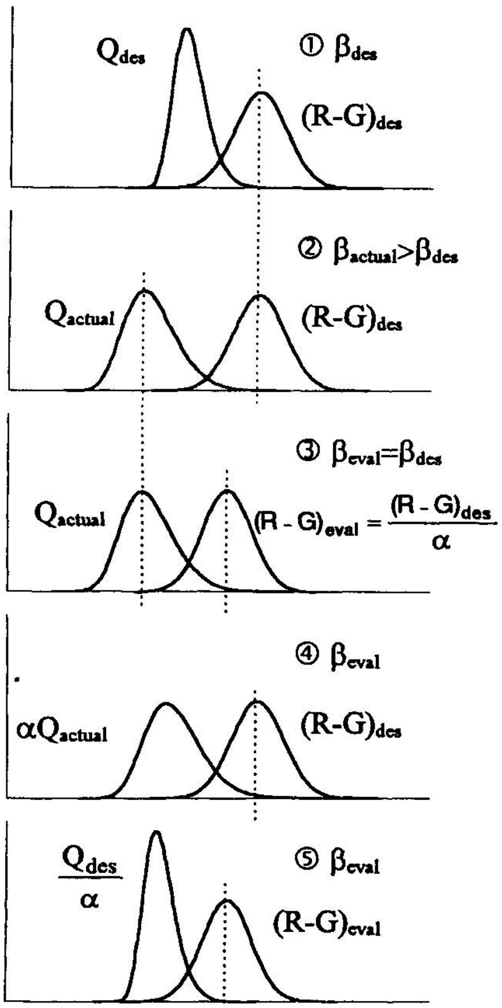

Fig. 1 Probabilistic basis for the determination of traffic load effect correction factors

Correction factors derived in this way can then be used in a deterministic evaluation of a bridge, using the same partial factor formulation adopted in design codes ;

$$S_d = S\left(\gamma_G G_m + \gamma_Q \frac{Q_r}{\alpha}\right) \le \frac{R}{\gamma_R}$$

2. TRAFFIC LOAD EFFECTS

Traffic load effects on a given bridge may be described by a certain frequency distribution, which in turn determines the load effect values to be considered in limit state calculations. The first goal of this study was therefore to identify the most suitable probabilistic model for this frequency distribution. Subsequently, a parametric study of the influence of certain site characteristics on this probabilistic model was carried out.

It would be very difficult, if not impossible, to use analytical methods in order to derive frequency distributions of traffic load effects from a complete statistical model of traffic loads. An analytical approach would only be feasible for a simplified representation of traffic, which could compromise the validity of results. The study described in this paper has therefore been carried out with the aid of a numerical simulation program in which random traffic loads were generated for defined traffic types and effects calculated for different structures. A description of this program and the modeling of traffic loads is given in [1, 2].

Traffic load simulations have shown that a type III extreme value distribution provides the best probabilistic model for maximum traffic load effects. This type of distribution results when maximum values are sampled from a frequency distribution having a finite upper bound which is approached polynomially [6]. The cumulative probability density function for maximum values, s^*, is :

$$F_{s^*}\left(s^*\right) = \exp\left[-N\cdot\left(\frac{W - s^*}{\chi\cdot W}\right)^k\right]$$

In this expression it can be seen that a type III extreme value distribution is characterized by four parameters; W, k, χ, and N. The parameter W is the finite upper bound, k is an inverse measure of the dispersion of the distribution and the parameter χ influences the position of the mean with respect to the maximum value, W. The parameter N is a measure of the return period for the maximum value, which in this case is the number of vehicles which pass over a structure within the period of interest.

Simulation results have been used to investigate the relationships between site characteristics and the parameters of fitted type III extreme value distributions of maximum traffic load effects. It was found that W can be calculated using the 99.9% fractile values of total vehicle weight and vehicle weight per unit length. These values of point load and uniformly distributed load are placed independently on the appropriate influence surface and the most unfavorable load case is adopted (for short span bridges, the point load predominates). The parameter k is proportional to the number of vehicles that contribute to the maximum effect and the standard deviation of vehicle loads. For the effects and structures simulated, k varied between 8 and 40. The parameter χ is determined by the form of the frequency distribution of vehicle loads, and varies between 0.75 and 1.1.

The results of fitting type III extreme value distributions to simulated traffic load effects were used for the probabilistic analysis of bridge loading and resistance, with distribution parameters being varied in order to represent different types of traffic. This is described in the next section.

3. PROBABILISTIC ANALYSIS OF BRIDGE LOADING AND RESISTANCE

As described in section 1.2, the determination of load correction factors was based on a probabilistic analysis of bridge loading and resistance. A first order second moment method was used in order to calculate a reliability index for different types of bridge, sections, materials and load effect. This index was used as the basis for comparing the effect of different types of traffic. Bridge deck sections were designed according to Swiss design codes in order to identify the critical limit state functions and to determine appropriate values for design variables.

The probabilistic characteristics assigned to design variables are summarized in Table 1. Values were selected as a result of a literature study of work by others [3, 4, 5, 7]. These values are not critical since the reliability indices were used only for comparing different types of traffic, but it is important that their selection is realistic so as to reflect the relative importance of traffic loading within limit state functions. Different traffic types and flow conditions were adopted in order to cover highways, main roads and feeder roads, with unrestricted and restricted traffic (limited to 16 tonnes maximum gross vehicle weight and vehicle crossing prevented). The different types of traffic considered are presented in Table 2.

Variable		Dist. type	Bias (mean / nominal)	Coeff. of variation
Steel elastic limit	rebar	LN	1.25	0.10
	prestress	LN	1.05	0.04
	plate	LN	1.19	0.08
Concrete strength		N	1.28	0.11
Sectional dimensions		N	1.0	0.01
Traffic loads		Ex III	see section 2	
Self-weight	steel	N	1.05	0.03
	concrete	N	1.05	0.10
Permanent loads		N	1.05	0.25

Table 1 Probabilistic characteristics of design variables

Type	Lanes	Route	Limits	N_{vehs} ($x10^6$)	Years
0	2 →	highway		250	50
1	1 →	highway	≠	2.5	1
2	2 →	main		250	50
3	2 ↔	main		250	50
4	2 ↔	feeder		125	50
5	1 →	main	≠	125	50
6	1 →	feeder	≠	65	50
7	2 →	main	16t	250	50
8	2 ↔	main	16t	250	50
9	2 →	feeder	16t	125	50
10	2 ↔	feeder	16t	125	50
11	1 →	main	16t, ≠	125	50
12	1 →	feeder	16t, ≠	65	50

↔ : bi-directional, → : unidirectional
16t : total weight restricted to 16 tonnes
≠ : vehicle crossing prevented

Table 2 Types of traffic

Limit state functions were formulated for midspan moment and support moment for composite, reinforced concrete and prestressed concrete box-section and slab-on-beam continuous bridges of different span lengths. All bridges studied carried two lanes of traffic. In total, 13 different types of traffic were considered for 19 limit state functions.

4. TRAFFIC LOAD EFFECT CORRECTION FACTORS

Using the probabilistic approach outlined above, traffic load effect correction factors were derived for different types of traffic. Figure 2 shows the factors calculated for support bending moment in composite slab-on-beam bridges. It can be seen that there is very little variation in correction factor as a function of bridge span and that the variation is mostly due to a change in traffic type. This was found to be the case for all structures considered. Similarly, correction factors were found to be approximately equal for midspan and support moments in the same structure. However, it was found that factors were significantly higher for box section bridges than for slab-on-beam bridges, particularly for the case of a traffic where vehicle crossing is prevented.

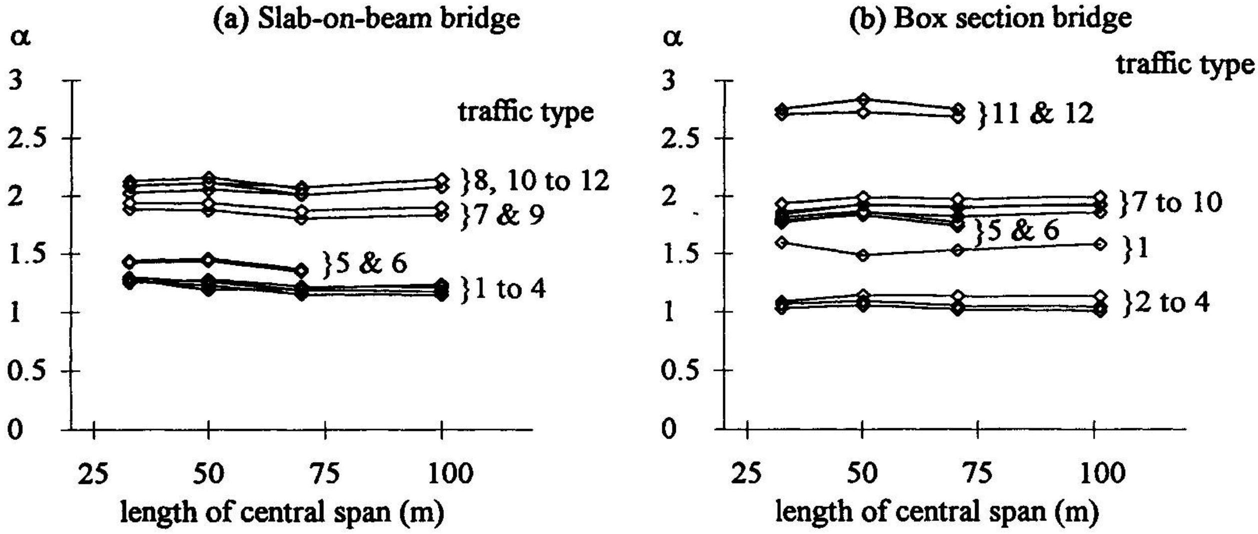

Fig. 2 Traffic load correction factors calculated for support bending moment in composite bridges

Figure 3 shows, for all the cases considered, the calculated factors as a function of traffic type. It can be seen that even though there is some variation in values as a function of effect type, bridge type and length of central span, a clear relationship between traffic type and minimum correction factor emerges.

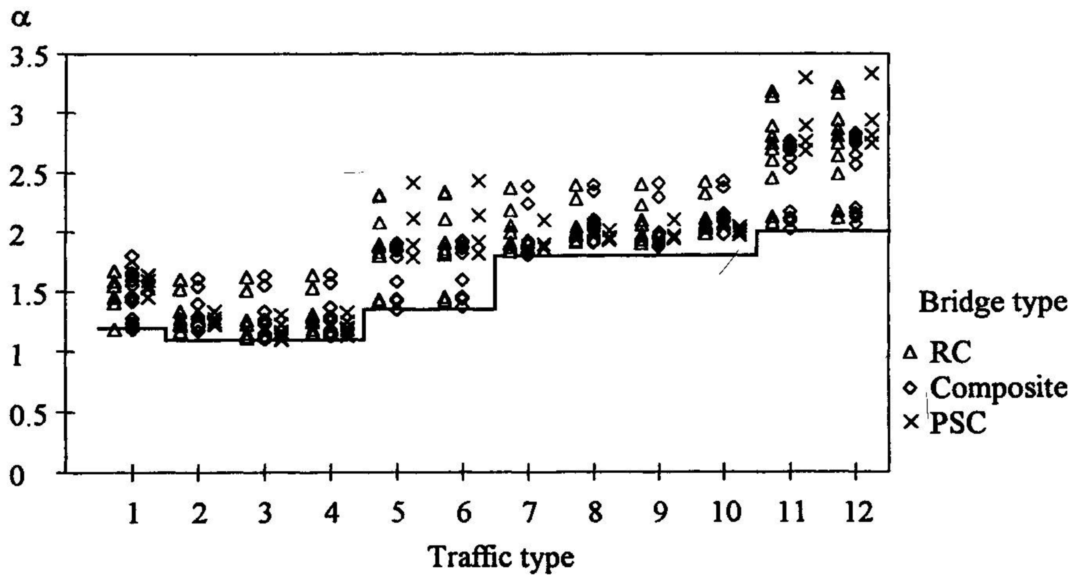

Fig. 3 Traffic load effect correction factors

The design traffic load effect correction factors ranged between 1.1 and 3.3 as shown in Figure 3. For the purpose of providing the simplest set of values for practical bridge evaluation it was decided to propose minimum factors as a function of only traffic type. These values are given in Table 3. It would however be possible to make more distinction between different types of bridge, and possibly even the type of load effect, in order to have a greater range of correction factors, and this is currently under review.

Route	Vehicle crossing prevented	Free traffic	16 tonne limit
Highway	no	1.00	-
	yes	1.20	-
Main or feeder	no	1.10	1.80
	yes	1.35	2.00

Table 3 Traffic load effect correction factors as a function of traffic type

The approach used for deriving correction factors relies solely on a comparison of different types of traffic and is largely independent of partial factors adopted by the design codes. The same method could therefore be used for deriving correction factors for other loading codes.

6. CONCLUSIONS

The results of this study are summarized as follows :

- Relationships have been found which enable the frequency distribution of maximum traffic load effects to be determined as a function of site characteristics. These relationships were used as the basis of a comparison of the effect of different traffic types within a probabilistic analysis of bridge loading and resistance.
- Traffic load effect correction factors have been determined which enable the effects calculated using the Swiss design traffic load model to be modified as a function of site characteristics for the purpose of deterministic bridge evaluation.

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