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Theme A

Basis of Evaluation

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Belgian Procedure for the Evaluation of Existing Steel Bridges

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Summary

In Belgium the gap between actual traffic loads and the code loads used for the design of existing bridges is very large. In the draft of the National Application Document (NAD) of ENV 1991-3 a bridge classification useful for a bridge manager is foreseen. This paper presents the background of this classification and a work in progress now in Belgium, concerning the development of methodologies for design and assessment of existing road bridges under actual traffic conditions.

1. Introduction

Road traffic has changed significantly in the last 40 years, particularly in Europe. This change consists of an increase in the number of lorries, and of 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. Existing bridges have been designed taking into account load models corresponding, in the best situation, to the loads of lorries allowed at that time or to old military loads. The question of the reliability and the durability of existing bridges arises for all bridges, but mainly if the gap between the loads of the models and the actual traffic is high. This is the case in Belgium [1]. During the development of the Eurocode ENV 1991-3 - 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] [3].

Many Belgian bridges do not satisfy the ultimate limit state and the fatigue resistance following requirement of the design Eurocodes in preparation (ENV 1992-2 for concrete bridges [4] and ENV 1993-2 for steel bridges [5]) when the loads defined in ENV 1991-3 are considered, but looking also the real traffic conditions during the supposed remaining bridge life. In order to develop a procedure for the classification of the existing bridges following the Belgian National Application Document of ENV 1991-3 [6] case studies are performed.

The aim of this paper is to present the background and the development of the classification, in the frame of the NAD.

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 generally not higher than 1,25 [1]. A vehicle, composed by 3 axles of 200 kN, spaced by 1,5 m and 6 m located on one slow lane, without any other life load, was foreseen as an exceptional vehicle, and was used in the design of some motorway bridges. The code loads are very low, and some times, since 1968, below the loads allowed by the Belgian Highway Code, i.e. : 120 kN for one single axle, 160 to 200 kN for a tandem axle, 200 to 270 kN on a tridem axle and 440 kN on a vehicle.

A new code has been developed in Belgium and published in 1993, the NBN B03-101, where the axle loads are increased (2 x 150 kN on each lane), the distributed load is a little lower (3,5 kN/m²), and a single heavy vehicle is foreseen alone on the bridge (6 x 150 kN) [7]. This code has been used for some bridges since 1986 [8].

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 HC, to the actual vehicle loads running on European motorways with a return period of one week Q_f (frequent load) and a return period of 1000 years Q_k (characteristic load) [9].

Figure 2 compares the bending moment, dynamic effect included, at the mid-span of a simply supported beam supporting one traffic lane, obtained under several loads. The ratio of the load effect obtained by one load and the NBN 5 loads may reach 1,4 for the allowed loads HC, 2,1 for the frequent loads of a heavy highway traffic Q_f, and 3,0 for the characteristic loads of this traffic or the loads of the main load model of ENV 1991-3 EC. The highest ratio corresponds to short spans (5 to 10 meters), and the ratio decreases when the span length increases. These conclusions are also valid for a lot of other influence lines [10].

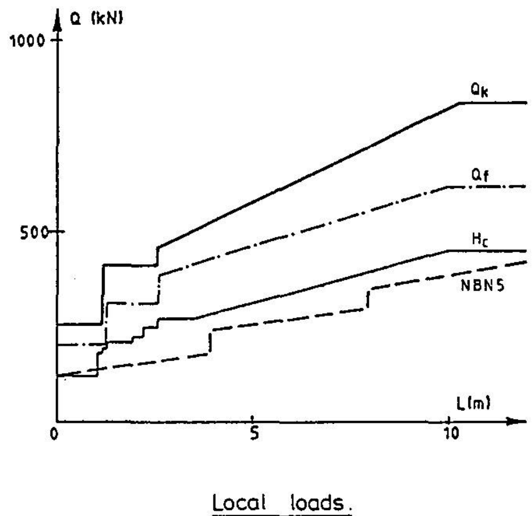


Fig. 1 Local loads

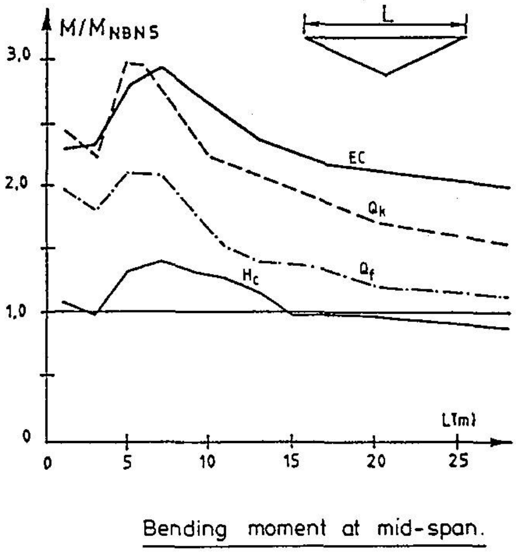


Fig. 2 Bending moment at mid-span

As the same loads are foreseen on each lane by the NBN 5 code and the loads foreseen in ENV 1991-3 are decreasing with the number of lanes, the ratio between the load effects is below 3 when two lanes are considered, and from 4 lanes the NBN 5 loads produce higher load effects than the Eurocode loads. Elsewhere, the frequent loads are lower than the NBN 5 loads on two lanes, longer than 14 meters, i.e. in main girders (see Table 1).

Number of lanes	NBN 5	ENV 1991-3	
		Q_k	Q_f
1	320 + 14 L	600 + 28 L	450 + 11 L
2	640 + 28 L	1000 + 37 L	750 + 15 L
3	960 + 42 L	1200 + 46 L	900 + 18 L
4	1280 + 56 L	1200 + 54 L	900 + 21 L

Table 1 *Total loads in kN and kN/m (lane width : 3,5 m., lane length : L > 16 m)*

3. Definition of the 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 [11]. The Belgian National Application Document of ENV 1991-3 is now drafted to define 4 classes useful for the bridges managers [6] [12].

The classes correspond to a gradual reduction of the reliability in the ultimate limit state, where the characteristic loads and the safety factor decrease :

- Class 1 concerns the design of new bridges. The loads correspond to the Eurocode loads, with two little modifications : the heavy loads foreseen on lane one are applied inside the limits of the actual carriageway, excluding hard strips, and no axle loads are foreseen on lane 3 ($\alpha_{Q3} = 0$).
- Class 2 concerns repair of bridges. Lane one should be located on the notional lanes, excluding hard shoulders. The loads are reduced to the infrequent loads : $\alpha_Q = \alpha_{q1} = 0,8$, but $\alpha_{qi} = 1$ for $i > 1$. This reduction corresponds to a shorter return period (1 year instead of 1000 years) and to a good roughness of the pavement. Under a heavy highway traffic the reliability factor of the bridge reaches here 4,7 instead of 6 for class 1.
- Class 3 concerns the existing bridges where no traffic limitation should be required. The loads correspond to the loads defined for class 2, but here the position of lane 1 may be imposed. For local effects, that result from the action of one single vehicle alone, the set of 5 lorries defined in Fatigue Load Model 2 is considered, because this model is more accurate in this case (table 3). If FLM 2 is used, the load of one axle should be multiplied by a dynamic factor $\Delta\varphi = 1,3$. For class 3, a reduction of the safety factors γ_G and γ_Q is admitted, so that the reliability factor decreases to 4, that corresponds to the ISO Recommendations [13].
- Class 4 concerns existing bridges, where traffic is limited. Here, the work in progress should propose practical solutions concerning the loads limitations.

Table 2 summarises the definition of the bridge classes in discussion for the Belgian NAD.

Class	1 European	2 Belgian	3 acceptable		4 limited traffic	
Load models	LM1 and LM2	LM1 and LM2	LM1	FLM2	LM1	FLM2
$\alpha_{Q1} = \alpha_{Q2} = \beta_Q$	1	0,8	0,8	1	C_Q	C_Q
α_{Q3}	0	0	0	-	0	-
α_{q1}	1	0,8	0,8	-	C_q	-
α_{qi} (i > 1)	1	1	1	-	C_q	-
γ_G	1,35	1,35	1,1		1,1	
γ_Q	1,35	1,35	1,2		1,2	

***Table 2** NAD Bridge classes*

4. Durability

The bridge classification has been defined in function of the ultimate limit state design. But the existing bridges have proved that they are able to support the actual traffic loads. Following the requirements explained above, an element on an existing bridge presents a lower reliability if it is admitted in class 3 instead of class 2. The bridge manager has to receive an answer to the question : how long should the resistance be sufficient to avoid collapse ? This question receives an answer by a fatigue assessment.

Up to now, few fatigue assessments have been performed on existing bridges. The fatigue life depends on the stress ranges $\Delta\sigma$ resulting from the traffic loads. Therefore the expected fatigue life is longer for an element admitted in class 2 than in class 3.

A fatigue life assessment needs information concerning the traffic in the past and in the future. It should be possible to collect data on the traffic running on the bridge today, by counting the number of the main types of lorries, and also by recording the axle loads of the vehicles. In some cases, the loads may be estimated from existing data and from the values given in the Eurocode ENV 1991-3 for a set of 5 lorries (table 3). In all cases estimations are needed for the past and the future.

For some roads, traffic data is available. This data can be used for evaluating fatigue life by applying the procedure presented at the IABSE Symposium in San Francisco [14]. The accuracy of the assessment depends on the accuracy of the available data.

5. Classification of existing bridges

The comparison of the loads prescribed by the Belgian code NBN 5 and the Eurocode ENV 1991-3 shows that existing bridges supporting heavy motorway traffics present weak points mainly located at the decks, concrete slabs or orthotropic steel decks, while main girders of large bridges should present a high safety factor.

The evaluation of an existing bridge starts by checking all sections and details, where some should satisfy the requirement of class 1 or class 2.

If that is not the case, the requirements of class 3 are to be checked. In general the infrequent loads of LM1 are considered and, only if the load effect analysed may result from the action of one vehicle alone on the bridge, the set of lorries (FLM2) is to be considered.

If the requirements of class 3 are not verified, the bridge falls in class 4 and traffic limitations are needed, if the bridge has to be maintained in service as long as possible with acceptable safety conditions. The evaluation should lead to one or several of the following limitations :

1. position of the slow lane, where the heaviest lorries are running,
2. restriction for crossing and overtaking of lorries,
3. distances between lorries in one lane,
4. gross weight of a vehicle.

These limitations create different types of hindrance to the traffic.

The elements presenting a bad classification and a short life time should be pointed out, and the bridge manager should pay particular attention to these elements during the periodical inspections. On the basis of all this information the bridge manager can make the right decision : changing the traffic flow conditions, limiting the vehicle loads, rebuilding the bridge, etc.

Class 4 may limit the gross weight of the lorries, because the ultimate limit state is not satisfied for some load effects, by considering the vehicles of FLM2 (Table 3). The limitation may result from the action of a single axle (190 kN), a tandem axle (280 kN), a tridem axle (360 kN) or the gross weight (630 kN). The definition of the highest vehicle gross weight should take into account two points :

1. the lightest vehicle comprising the critical axle :
   - a two-axled vehicle of 280 kN, if the limitation results from a simple axle,
   - a three-axled vehicle of 360 kN, if the limitation results from a tandem axle,
   - a half trailer of 630 kN, if the limitation results from a tridem axle.
2. the weight of the vehicles allowed to run in Belgium are around 40 % lower than the weight of the vehicles of FLM2.

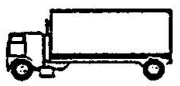
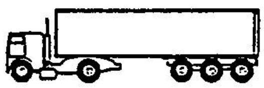
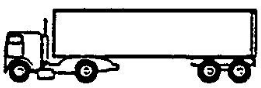
	Vehicle	EC : FLM2	Belgium : HC		EC / HC
1		90 190 280	70 120 190		- 1,58 1,47
2		80 140 140 360	60 100 100 260		- 1,40 1,40 1,38
3		90 180 120 120 120 630	50 120 90 90 90 440		- 1,50 1,33 1,33 1,33 1,43
4		90 190 140 140 560	70 120 100 100 390		- 1,58 1,40 1,40 1,44
5		90 180 120 110 110 610	50 90 100 100 100 440	70 120 70 90 90 440	- - - - - 1,45

Table 3 *Vehicle loads*

6. Need for case studies

The definition of the bridge classes is introduced in the draft of the Belgian NAD of ENV 1991-3. In annex M of the NAD the rules described above are in discussion. In order to finalise the draft, a reliability study is needed, based on case studies.The open questions concern the classification of existing bridges in class 3 and class 4.

The questions to solve for class 3 concern :
- the factor to apply on the main load model (LM1) :
  on the axle loads : $\alpha_Q = 0{,}7$ to $0{,}8$
  on the distributed loads : $\alpha_{q1} = 0{,}4$ to $0{,}8$
  $\alpha_{qi} = 0{,}4$ to $1{,}0$
- the definition of a dynamic effect to apply on some axles of FLM2,
- the transverse position of the loads of lane 1 or FLM2 : everywhere on the carriageway, or on the actual slow lane,
- the values of the safety factors : $\gamma_G = 1{,}1$, $\gamma_Q = 1{,}2$.

The questions to solve for class 4 concern :
- the kind of limitation,
- the definition of admitted loads,
- the position of the admitted loads.

The bridges for the case studies in Belgium are chosen between bridges built during the last 30 years in the frame of the development of the motorway network. The evaluation is performed in several successive steps using the modern design means available :
1. the main load model, LM1 of ENV 1991-3 is used in order to check the requirements of the design Eurocodes for ULS [4] [5], that leads to class 1 or class 2,
2. for elements that don't satisfy the condition above, the requirements of class 3 are checked,
3. for elements that don't satisfy the condition of class 3, traffic limitations should be defined as explained for class 4 in section 5,
4. a fatigue assessment is performed following the design Eurocodes [4] [5],
5. taking into account all data available, a fatigue assessment is performed following the procedure described in [14] and [15] : in this frame the following fatigue load models are used :
   - for concrete slabs : FLM3 or FLM4
   - for orthotropic steel decks : FLM2 or FLM4
   - for main girders in steel : FLM1 or FLM3
   The actual frequency of the type of vehicles should be introduced directly in FLM4, and by the means of a correction factor in FLM3.
   It has been shown that, if the number of vehicles per year is very high, FLM1 or FLM2 leads to less severe and more realistic conclusions than FLM3 [15] [16],
6. for some important elements, detailed ultimate limit state analysis and fatigue assessment should be performed by considering complete load spectra available for real recorded traffic with the means of a simulation programme [9]. This analysis is needed in order to find the right conclusions for the definition of the bridge classes.

7. Work in progress

A research programme is running now in Belgium in order to develop a methodology for design and reassessment of bridges, with the participation of the Universities of Liège and Ghent. The choice of the bridges to analyse has been made in conjunction with the Belgian Authorities, members of the group, which provide for the detailed construction drawings. The choice will be made for bridges most sensitive to the increase of the traffic loads, where consequently the security could be the least, and where the life duration could be the shortest.

Using a previous study, the three next bridge types are retained :

1. Slab and beams bridges, with prefabricated prestressed concrete beams, with or without bracing, where the slab, beams and bearings will be examined.
2. Frame bridges with or without earth overloading, with deck and walls to be examined.
3. Steel bridges with orthotropic deck, with principally deck and bracing to be examined.

The results obtained for slab and beams bridges should lead to conclusions concerning some composite bridges.

Three steel bridges should be analysed :
- one movable bridge with open stringers,
- one box girder bridge with trapezoidal stringers,
- one box girder bridge with open stringers.

More information is given in [17].

The objectives of the case studies will be the following :

1. to develop a detailed calculation procedure, applicable to the reassessment of the existing bridges and for the design of new bridges, so the bridge engineers would have completely treated examples at their disposal ;
2. to show shortcomings of the Eurocodes being prepared and to bring up solutions ;
3. to show eventual difficulties in applying some prescriptions, with possible solutions ;
4. to test alternative procedures, and to highlight their advantages ;
5. to test the fatigue calculation described in Bruls' doctorate thesis [9], to propose formal rules for Eurocodes ENV 1992-2 and 1993-2 [4] [5] and to specify the cases where fatigue calculation is needed ;
6. to define a procedure for the estimation of the life duration of an existing bridge, related to the peculiar traffic breakdowns using the bridge since its erection ;
7. to study the importance of the dynamic magnification included in the Eurocode 1.3. loads in the particular case of the deck of frame bridges with an earth overloading [2],
8. to show the weak points of the existing Belgian bridges and to propose solutions to the bridge managers for increasing the life duration ;
9. to bring out conclusions about the design of the examined bridge cases, and to propose eventually more performing solutions for the design of new bridges ;
10. to define the best system of road-signs able to express the strength requirements.
11. as the fatigue calculation of bridges is not yet generally used, it is useful to precise the conditions where no fatigue calculation is needed.
12. furthermore, for simple and frequent structure elements, solutions completely worked out will be given in tables.

8. Conclusions

This paper has shown the gap between the actual traffic loads and the code loads used up to now in Belgium for the design of bridges. A bridge classification for existing bridges is in discussion in the frame of the NAD of the Eurocode ENV 1991-3 - Traffic loads on bridges. In order to solve practical problems of such a classification, a research programme supported by the Authorities is in progress. Case studies are being performed on bridges that are most influenced by the high axle loads of the actual traffic. The case studies should show where and in which type of bridge the weakest points are located, and should give a better idea on the risk of damage. This information is needed for the bridge management in order to avoid structural failure and traffic restriction, for example. Although the first results are more optimistic than the direct comparison of the loads, we have to wait for the research work to be completed (1998) before drawing the final conclusions.

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Reliability-Based Evaluation Section of the Draft Canadian Highway Bridge Design Code

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Summary

This paper outlines the code provisions of the draft Canadian Highway Bridge Design Code, CHBDC, which is to be published in 1997, for the evaluation of live load capacity of existing bridges. A methodology to assess the adequacy of the existing 3000 provincial bridges, designed to previous codes dating back to the turn of the century, for the current vehicle loads in Ontario based on the probabilistic approach reflected by the draft provisions of CHBDC is discussed.

1. Introduction

The Canadian Highway Bridge design Code, CHBDC, has been under development since 1993 and will be published in 1997. It will replace the current editions of the Ontario Highway Bridge Design Code, OHBDC, [1]; the Canadian Standard, CSA-S6, for Bridge Design, [2]; and the CSA-S6 Supplement No. 1 for Existing Bridge Evaluation [3].

The Evaluation Section of the draft Canadian Highway Bridge Design Code, CHBDC, provides procedures of evaluating an existing bridge to determine if it will carry a particular load or set of loads. The bridge may be evaluated by one of the following methods:

- Ultimate limit state, serviceability limit state, fatigue limit state;
- Mean load method;
- Load testing;
- Other methods approved by the Regulatory Authority.

2. Permanent Loads

The evaluation of the load carrying capacity of existing bridges should consider all permanent loads. Dead load should include the weight of all components of the bridge, fill, utilities and other materials permanently on the bridge. Earth pressure and hydrostatic pressure should be treated as permanent loads.

3. Transitory Loads

3.1 Normal traffic

The traffic load models to be used for the Evaluation Levels 1, 2 and 3 are CL1-W, CL2-W and CL3-W Loading, respectively. They consist of the corresponding Truck given in Figure 1, or the corresponding Lane Load given in Figure 2, whichever gives larger load effects.

The number "W" indicates the gross load in kN of the CL1-W Truck. Corresponding gross loads for the CL2-W Truck and the CL3-W Truck are 0.76 W and 0.48 W, respectively. In general, the value of W is taken as 625 kN.

The uniformly distributed load in a Lane Load occupies a width of 3.0 m in a traffic lane, and is placed transversely concentric with the truck.

3.2 Permit vehicle loads

Vehicles operating under permit are classified as PA, PB, PC or PS.

PA traffic includes the vehicles authorized by permit on an annual basis or for the duration of a specific project to carry an indivisible load, mixed with other traffic without supervision. PB traffic includes bulk haul traffic, or vehicles carrying divisible load authorized by permit programs for many trips, mixed with general traffic. PC traffic includes vehicles authorized by permit to carry an indivisible load on an specified route under supervision and specified travel conditions. PS traffic includes vehicles authorized by permit for a single trip, to carry an indivisible load, mixed with other traffic without supervision.

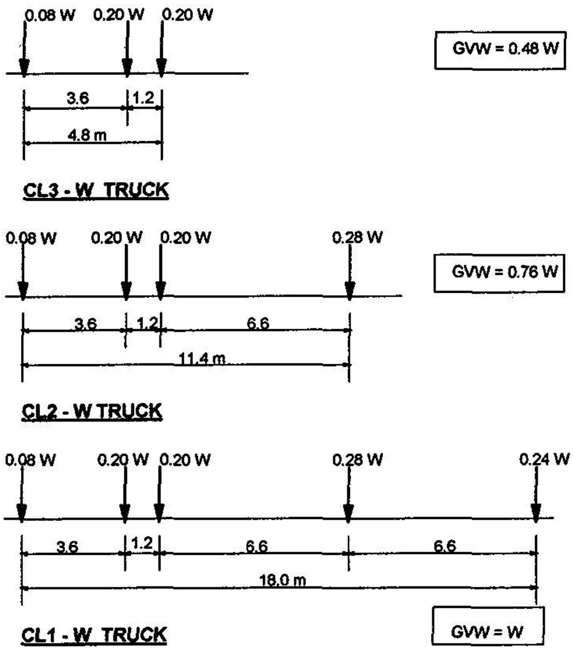
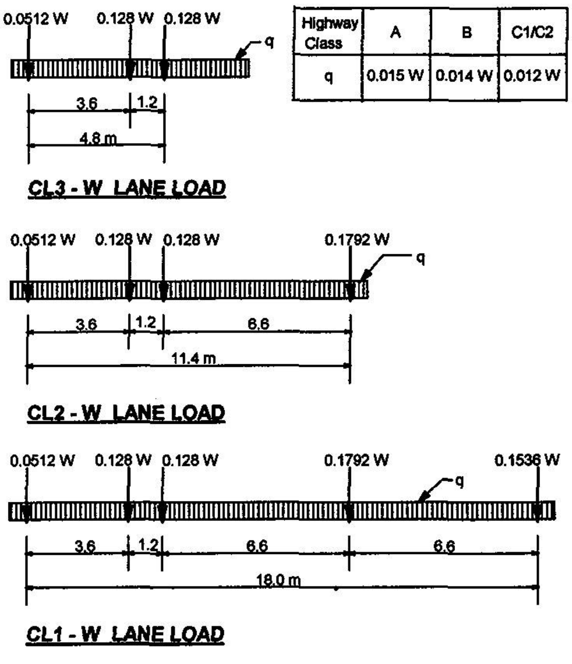


Fig. 1 Truck loading for normal traffic



Highway Class	A	B	C1/C2
q	0.015 W	0.014 W	0.012 W

Fig. 2 Lane loading

4. Target reliability index

The reliability index, β, is taken from Table 1(a) for all evaluation levels for normal traffic and for permit vehicles except PC vehicles for which β is taken from Table 1(b). In both cases, the system behaviour, element behaviour, and inspection level are taken as defined below.

The life safety criteria that forms the basis for the reliability indices presented in this section considers only loss of life resulting directly from the failure of the structure. For structures which indirectly affect life safety or are essential to the local economy or are necessary for the movement of emergency vehicles, a value of β which is 0.25 greater than those given in Table 1 is used.

4.1 System behaviour

- Category S1, where element failure leads to total collapse;
- Category S2, where element failure probably will not lead to total collapse; or
- Category S3, where element failure leads to local failure only.

4.2 Element behaviour

- Category E1, where the element being considered is subject to sudden loss of capacity with little or no warning;
- Category E2, where the element being considered is subject to sudden failure with little or no warning but will retain post-failure capacity; or
- Category E3, where the element being considered is subject to gradual failure with warning of failure probable.

4.3 Inspection level

- Level INSP1, where a component is not inspectable;
- Level INSP2, where inspection is to the satisfaction of the evaluator, with the results of each inspection recorded and available to the evaluator; or
- Level INSP3, where inspection of critical and/or substandard components has been carried out by the evaluator and final evaluation calculations account for all information obtained during this inspection.

System Behaviour	Element Behaviour	Inspection Level		
		INSP1	INSP2	INSP3
S1	E1	3.75	3.5	3.5
	E2	3.5	3.25	3.0
	E3	3.25	3.0	2.75
S2	E1	3.5	3.25	3.25
	E2	3.25	3.0	2.75
	E3	3.0	2.75	2.5
S3	E1	3.25	3.0	3.0
	E2	3.0	2.75	2.5
	E3	2.75	2.5	2.25

Table 1(a) *Target reliability index, β, for CL1-W, CL2-W, CL3-W, PA, PB & PS traffic*

System Behaviour	Element Behaviour	Inspection Level		
		INSP1	INSP2	INSP3
S1	E1 E2 E3	3.25 3.0 2.75	3.0 2.75 2.5	3.0 2.5 2.25
S2	E1 E2 E3	3.0 2.75 2.5	2.75 2.5 2.25	2.75 2.25 2.0
S3	E1 E2 E3	2.75 2.5 2.25	2.5 2.25 2.0	2.5 2.0 2.0

Table 1(b) *Target reliability index, β, for PC vehicles*

5. Load factors

The unfactored loads effects for each element under consideration is multiplied by the appropriate load factors for the value of β determined above.

5.1. Dead loads

When the dead load effect counteracts the effect due to transitory load, the minimum dead load factors are used for all dead load categories at any β value. Otherwise, the maximum dead load factors given in Table 2 are used.

Dead Load Category	Target reliability index, β							
	2.00	2.25	2.50	2.75	3.00	3.25	3.50	3.75
D1	1.04	1.05	1.07	1.08	1.09	1.10	1.12	1.13
D2	1.10	1.13	1.17	1.20	1.22	1.25	1.29	1.33
D3	2.10	2.30	2.50	2.70	2.90	3.10	3.30	3.50
D4	1.08	1.10	1.14	1.16	1.18	1.20	1.24	1.26

Table 2 *Maximum dead load factors, α_D*

Where:

D1 = factory-produced components, excluding wood
D2 = cast-in-place concrete, wood and all non-structural components
D3 = wearing surface
D4 = earth fill, negative skin friction on piles

5.2. Normal traffic

Live load factors for Evaluation Levels 1, 2, and 3 are as given in Table 3.

Type of Analysis	Target reliability index, β						
	2.25	2.50	2.75	3.00	3.25	3.50	3.75
Statically determinate	1.42	1.47	1.51	1.57	1.63	1.69	1.75
Sophisticated	1.48	1.55	1.62	1.69	1.75	1.83	1.90
Simplified	1.36	1.42	1.49	1.55	1.62	1.70	1.78

Table 3 *Live load factors, α_L, for normal traffic*

6. Live load capacity factor

For ultimate limit states the smallest value of the live load capacity factor, F, is calculated using the following equation for all structural components:

$$F = \frac{UR_f - \Sigma\alpha_D D - \Sigma\alpha_A A}{\alpha_L L(1+I)} \qquad (1)$$

7. Mean Load Method

As an alternative to Equation (1) the live load capacity factor, F, at the ultimate limit state may be calculated using the following equation:

$$F = \frac{\overline{R}\exp[-\beta(V_R^2 + V_S^2)^{0.5}] - \Sigma\overline{D}}{\overline{L}} \qquad (2)$$

where:

$$\Sigma\overline{D} = \Sigma\delta_D\delta_{AD}D$$

$$\overline{L} = \delta_L\delta_{AL}L(1+\delta_I I)$$

$$\overline{R} = \delta_R R$$

$$V_S = \frac{(S_{\Sigma\overline{D}}{}^2 + S_{\overline{L}}{}^2)^{0.5}}{(\Sigma\overline{D} + \overline{L})}$$

$$S_{\Sigma\overline{D}} = (V_D{}^2 + V_{AD}{}^2)^{0.5}\delta_D\delta_{AD}D$$

$$S_{\overline{L}} = \left[V_{AL}{}^2 + V_L{}^2 + \frac{(V_I\delta_I I)^2}{(1+\delta_I I)^2}\right]^{0.5}\delta_L\delta_{AL}L(1+\delta_I I)$$

Where:

D nominal (unfactored) dead load effect

$\bar{D}$ mean dead load effect

I nominal (unfactored) dynamic component of the live load, expressed as a percentage of the nominal static live load effect

L nominal (unfactored) static live (traffic) load effect

$\bar{L}$ mean static and dynamic live (traffic) load effect

R nominal unfactored resistance

$\bar{R}$ mean resistance

$S_{\Sigma\bar{D}}$ standard deviation of dead loads force effects

$S_{\bar{L}}$ standard deviation of live load force effects

δ_{AD}, δ_{AL}, δ_D, δ_I, δ_L, δ_R bias coefficients (ratios of mean to nominal effects) for dead load analysis method, live load analysis method, dead load, dynamic load allowance, live load and resistance respectively.

V_{AD}, V_{AL}, V_D, V_I, V_L, V_R, V_S coefficients of variation for dead load analysis method, live load analysis method, dead load, dynamic load allowance, live load, resistance, and total load respectively.

β target reliability index

8. Global assessment of existing bridges in Ontario

Following the principles behind the mean load method described in (8) above, a modified approach was used to assess the adequacy of the existing bridges in Ontario to safely carry the current vehicle loads. The existing bridges in Ontario have been designed for various different design loads, such as H10, H15, H20, HS20, OHBD Loading. The bridges built prior to 1960's were generally designed by the working stress design methods. Bridges built after 1960 have mostly been designed by the load factor design methods or by limit states design approach.

The population was divided in to various families of bridges characterized by the original design load and design method, the structure type and span length. For each family of bridges, the reliability index β was determined for the current vehicle loads. This reliability index was then used to assess the adequacy of the bridges in that family.

To determine the reliability index, the following equation was used,

$$\beta = \frac{\ln(\bar{R}/\bar{S})}{\sqrt{(V_S^2 + V_R^2)}} \qquad (3)$$

where $\bar{S}$ is the real total mean load effect, including the mean dead load effects and the mean largest live load effects including dynamic load allowance. The other notations have the same meanings as given in Section 7.

8.1 Statistics for real loads

For this assessment, only the effects of dead load, traffic loads and the dynamic load allowance were considered. Statistics for various types of dead load effects used in the analysis are given in Table 4.

Dead Load Component	Description	Mean/Nominal Ratio	Coefficient of Variation
D1	Factory-produced components, excluding wood	1.03	0.04
D2	Cast-in-place concrete, wood, and all non-structural components	1.05	0.08
D3	Wearing surfaces	1.10	0.20

Table 4 *Statistics for dead load effects*

The extreme real lifetime live load effects were determined using data for 6287 trucks from the 1995 Commercial Vehicle Survey conducted in Ontario. Real values of mean dynamic load allowance and its coefficient of variation were obtained from the field testing results of a number of bridges in Ontario [4]. Mean value of dynamic load allowance was taken to be 0.20 up to a span length of 10 m with a coefficient of variation of 0.60, and 0.14 for spans greater than 10 m with a coefficient of variation of 0.82.

8.2 Real resistance

Since the evaluation did not address a specific bridge for which actual member sizes could be obtained from the drawings, real resistance for a family of bridges was determined by considering the original design provisions and design method. From the design provisions, minimum required nominal resistance at the ultimate limit states of the critical component was determined. The statistics for the real resistance were obtained by using the bias factors and coefficient of variation for resistance from [5] for structural steel components, and from [6] for the concrete components. These are summarized in Table 5.

Type of Response	Bias Factor, ρ_R	Coefficient of Variation, V_R
Steel Rolled Sections:		
Plastic Moment	1.126	0.081
Moment at First Yield, Axial Tension	1.210	0.077
Steel Welded Section:		
Plastic Moment	1.133	0.096
Moment at First Yield	1.221	0.100
Composite Plastic Moment	1.098	0.096
Reinforced Concrete	1.04	0.090
Prestressed Concrete	1.03	0.080

Table 5 *Bias factor and coefficient of variation for resistance*

8.3 Interpretation of results

The target value of reliability index for main components with ductile behaviour and normal inspection is 3.50. The family of bridges which had a value of reliability index less than 3.50 could be considered somewhat deficient to carry unrestricted traffic. However, with regular inspection and a re-evaluation within five years, reliability index of as low as 2.80 was considered an acceptable threshold for a need to post a bridge. Considering this threshold, it was found that approximately 4 percent of the bridges on the provincial system in Ontario may be potentially deficient and would require a detailed evaluation to establish posting loads. Most of these bridges are old steel trusses or steel girder bridges designed for H 15 or H 20 loads.

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Fatigue Considerations in the Evaluation of Existing Reinforced Concrete Bridge Decks

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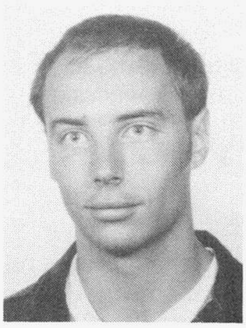

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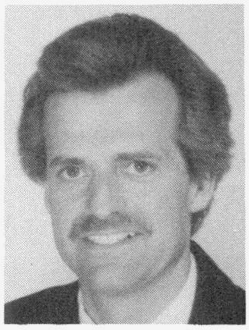

Summary

Despite the fact that deck slabs are the most fatigue loaded elements of composite bridges, they are rarely designed or examined for fatigue. The fatigue safety of existing reinforced concrete deck slabs often cannot be verified if current fatigue design provisions are applied. Thus, there is a strong need to enhance knowledge of the fatigue behaviour of reinforced concrete deck slabs. The initial results of ongoing research with fatigue tests on slab-like reinforced concrete beams indicate that current fatigue design provisions appear to be too conservative and that the fatigue reliability of existing deck slabs is satisfactory if the principles of good fatigue design and construction practice have been respected.

1. Introduction

1.1 Motivation

Deck slabs are the most fatigue loaded elements of composite bridges. The fatigue loading is due to moving wheels and is characterised by a high number of heavy load cycles which may exceed 100 millions over the service life of a bridge. Despite this fact, reinforced concrete deck slabs are commonly not designed for fatigue and most attention is still given to the fatigue of the welded steel structure of composite bridges.

Fatigue loading of deck slabs may lead to progressive damaging of concrete and steel reinforcement, with subsequent failure. Although the fatigue phenomenon has been observed in many tests, few cases of concrete bridge damage in deck slabs due to fatigue are known [1, 2]. This may be explained by the fact that fatigue cracking of concrete cannot be clearly distinguished from cracks due to other concrete deterioration, and fatigue cracking of the steel reinforcement cannot be observed.

The objective of this paper is (1) to outline the question of fatigue of reinforced concrete decks of composite bridges and (2) to present first answers on the basis of results of current research.

Fatigue provisions have been introduced in design codes for concrete structures only during the last few years, for example in Switzerland in 1989 [3] or more recently in the pre-Eurocode 2 dealing with bridge design of concrete bridges [4]. These provisions rely on a narrow knowledge basis when compared to most other domains of concrete structures. They have led to a significant change in the design of railway bridges and deck slabs for road bridges; fatigue often is the determinant design criterion. As a result, considerable additional reinforcement, in particular shear reinforcement in slabs, and larger structural dimensions are necessary when compared to elements designed using former codes. This situation raises two questions: Are these fatigue design rules too conservative? Is the fatigue reliability of existing bridges in jeopardy?

1.2 Fatigue safety of existing concrete bridges

To investigate the second question, the fatigue safety of various concrete railway bridges and one composite road bridge has been examined on the basis of current code provisions [5]. The ratio $n_{fat} = R_{fat}/S_{fat}$ between fatigue resistance R_{fat} and fatigue action effect S_{fat} has been determined and a ranked list identifying fatigue critical structural elements for $n_{fat} < 1$ has been established. These structural elements do not fulfill the requirements of current codes regarding fatigue safety and thus the fatigue life may be shorter than the design service life of the bridge. The most fatigue critical elements of these bridges turned out to be the deck slabs and in particular when subjected to shear stress in the concrete.

In addition, examination of several existing concrete bridges in Switzerland allowed to make the following observations related to deck slabs :

- Typical calculated maximum stress values in the traverse direction of deck slabs due to the traffic model for fatigue in the Swiss code (4 concentrated wheel loads of 75 kN each in distance of 1.8 m and 1.4 m and a dynamic factor of 1.8) are 14 MPa for concrete normal stress, 0.45 MPa for concrete shear stress and 225 MPa for steel reinforcement. The minimum stress level is between 10% and 30% of the maximum stress.
- Due to the different load positions for fatigue safety and structural safety verification, fatigue safety can be insufficient in cross sections where structural safety is fulfilled and vice-versa.
- Thickness of pavement and deck have an important influence on load distribution and the determination of stresses in deck slabs. Distribution of concentrated loads through the pavement and deck under a 45 degree angle [4] may double the wheel print from a square of 0.4 m to 0.8 m which smoothes significantly the amplitude of local shear stresses.
- Verification of decks in the longitudinal direction gave values n_{fat} for shear stress reversals (Fig. 3) significantly smaller than 1.
- Normal stress reversals in the longitudinal direction lead to transversal cracking of deck slabs. This affects serviceability more than safety.

Whether the fatigue reliability of the investigated deck slabs really is in jeopardy cannot be answered now. It is speculated that sufficient fatigue safety may be determined on the basis of improved knowledge and a more realistic examination. Hence, a strong need to enhance knowledge in fatigue behaviour of concrete bridges is identified. Additionally, the steady increase of traffic loads demands a keener alertness to the fatigue phenomenon.

2. Fatigue of concrete bridge elements

The three most important fatigue relevant parameters are (1) the magnitude of stresses, (2) the number of load cycles and (3) discontinuities both in the cross section and the layout of the steel reinforcement resulting in stress concentration at possible fatigue damage locations. The stress magnitude due to fatigue loading and the number of load cycles determine whether fatigue damage occurs in these locations. Fatigue reliability of a structural element over its design service life is verified if the fatigue resistance R_{fat} is larger than the effect of fatigue loading S_{fat}. The fatigue safety of steel reinforcement and concrete are determined separately :

2.1 Fatigue of steel reinforcement

Similar to steel structures, the fatigue resistance of mild and prestressing steel reinforcement may be represented by the detail category $\Delta\sigma_{fat}$ which is defined as the fatigue strength at $2x10^6$ cycles (Fig. 1). Regarding the fatigue action effect, the stress range is the most important parameter and a correction factor is used to account for the cumulative fatigue damage caused by the stress spectrum of traffic models for a lifetime of a hundred years. This correction factor as calculated for steel bridges using the Palmgren-Miner damage accumulation rule is also applicable with sufficient accuracy to steel reinforcement in concrete bridges [6].

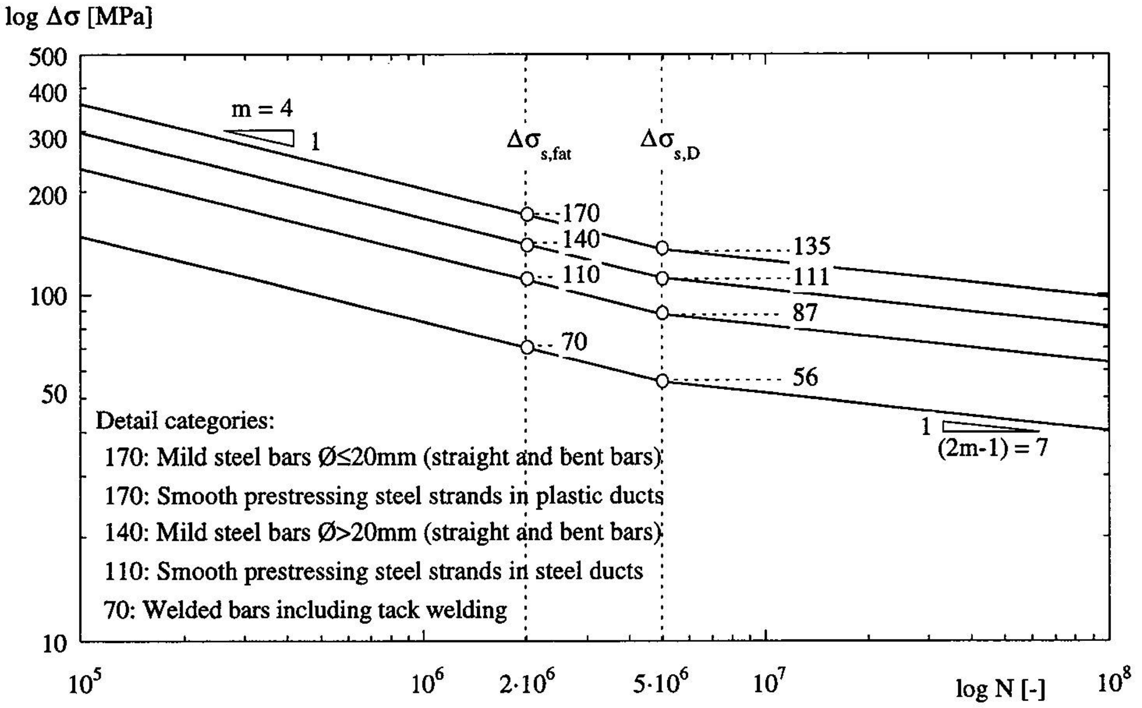

Fig. 1 Fatigue strength of steel reinforcement according to [6].

2.2 Fatigue of concrete

The fatigue resistance of concrete is in principle defined by a pair of stresses, i.e. the maximum and minimum stress values as the most important fatigue relevant parameters. The effect of this pair of stresses on the fatigue strength as a function of the number of load cycles is best represented by a Goodman diagram (Fig. 2). Other fatigue relevant parameters include the concrete strength and the structural size effect which are taken into account by the nominal design values f_c and τ_c for static compressive and shear strength respectively. The fatigue action effect in the concrete is described by the maximum and minimum stress values due to the fatigue loading and the dead load of the structure including permanent loads. No correction factor is introduced because of a lack of proven models for fatigue damage accumulation in concrete.

In the longitudinal direction of the deck slab, stress reversals may occur in a given section due to the passage of a moving load (Fig. 3). The effect of these stress reversals for shear is not conclusively known; the literature [7, 8] indicates a strong reduction of shear fatigue resistance which is accounted for in Fig. 2b. Since the fatigue strength of the rebars is not expected to be sensitive to stress reversals, the failure mode of the slab under normal stress reversals is likely to be characterised by concrete fracture alone.

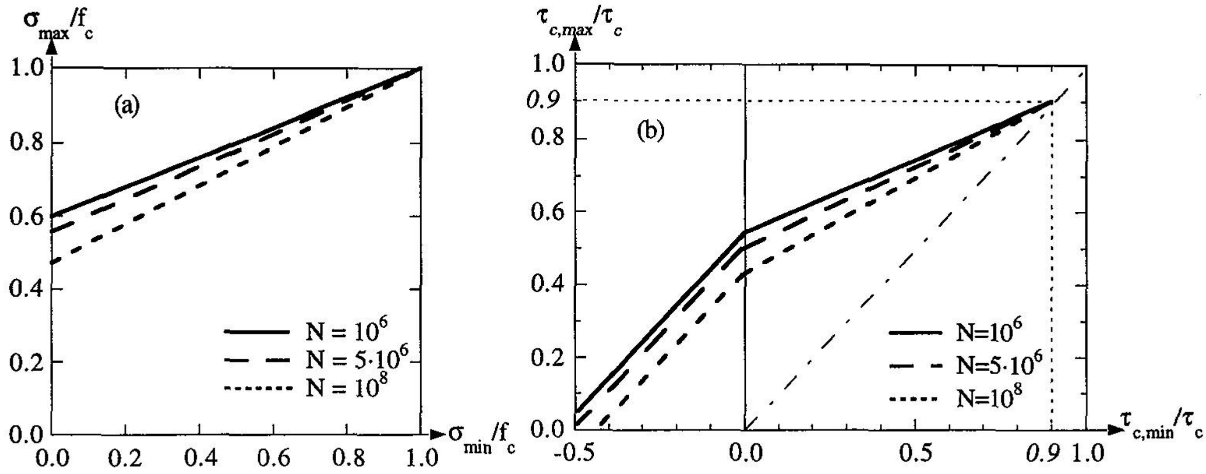

Fig. 2 Fatigue strength of concrete as represented by a Goodman diagram for (a) compressive and (b) shear stresses according to [6].

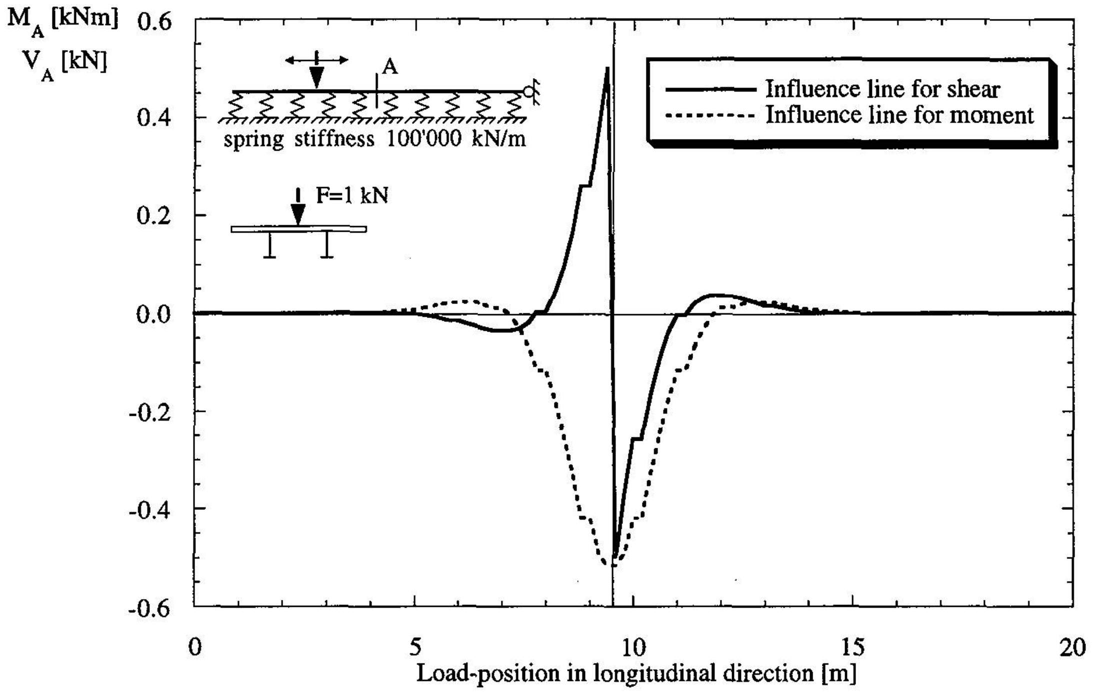

Fig. 3 Influence line of a moving load in the longitudinal direction of a deck slab.

3. Concept for the examination of existing bridge decks

In the examination of existing bridges, a stepwise procedure is normally adopted.

The *first step* is a verification of the structural safety, fatigue safety and serviceability of the existing structure based on the requirements of current codes. The objective of this first step is to identify determinant structural elements.

In the *second step*, the existence of the structure allows load and resistance models to be updated. Updating the information on a given bridge includes :

- more detailed assessment of past and future traffic loads (including their locations on the slab)
- determination of actual structural dimensions

- determination of in-situ material properties

If safety cannot be verified during the second step, stresses can be determined in a *third step* using more detailed methods, for example in-situ load testing or a refined FE model, and more advanced methods such as those based on probabilistic concepts.

4. Current research at the Swiss Federal Institute of Technology

4.1 Specimens and experiments

Research focusing on the remaining service life of existing concrete bridges has recently begun. Firstly, the fatigue behaviour of concrete slabs without shear reinforcement is investigated. Seventeen slab-like beams without shear reinforcement have been made (Fig. 4) using ordinary concrete with an average compressive cylinder strength f_{cc} of 35 MPa at the age of 28 days, which corresponds to a nominal compression design strength f_c of 19.5 MPa and a nominal shear design strength τ_c of 1.0 MPa. Three reinforcement ratios were chosen; the yield and ultimate strength of the mild steel bars is 490 MPa and 585 MPa respectively. All beams were older than 90 days when tested.

Before conducting the fatigue tests, two specimens have been tested under static loading to observe the crack pattern and to determine the ultimate static strength. Fatigue loading was applied by hydraulic actuators providing a sinusoidal load history at a frequency of 4.5 Hz. The first six specimens were tested with a minimum load equal to 10% of the maximum load. Since no concrete damage was detected, the minimum load was increased to 30 % of the maximum load. One specimen was even tested at a minimum load level of 54 % of maximum load but it was found that the maximum load must be less than 75-85% of the ultimate static strength to avoid yielding of the rebars with subsequent failure due to low cycle fatigue.

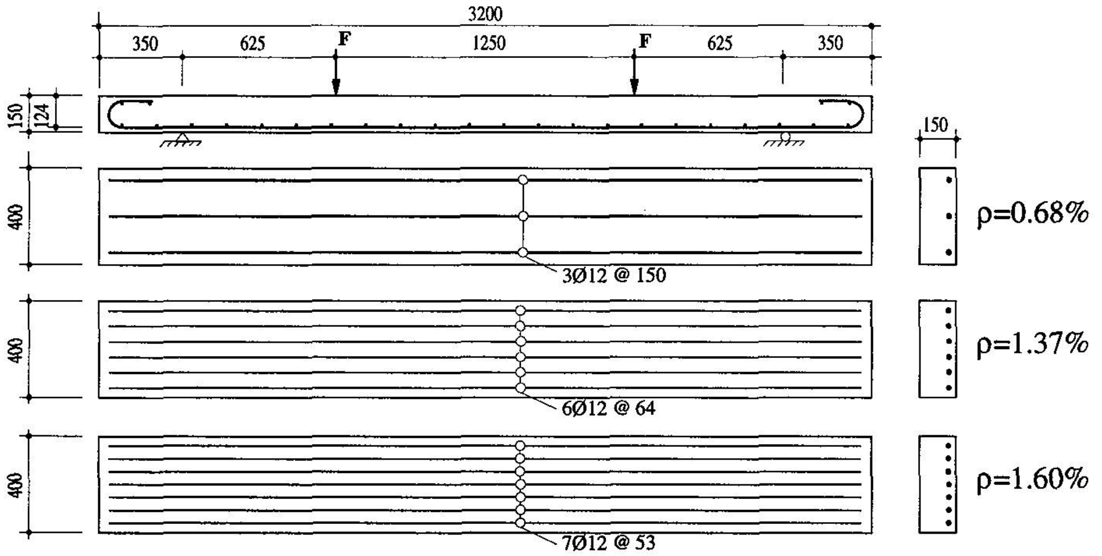

Fig. 4 *Specimen dimensions and static system (all dimensions in mm).*

The force, the axial strains in one rebar and the axial strains on the compressed concrete surface in the constant moment region were monitored during the test. Deflection was measured at mid-span. Two strain gauges with a length of 20 mm were glued to the lower side of one rebar of each specimen after casting the specimens. For this, the rebar surfaces were made accessible for the glueing of the strain gauges by an opening which was blocked out in the formwork. This hole created stress concentration so that cracks appeared without any exception in the cross sections with the strain measurements. In the same sections, strain gauges with a length of 100 mm were glued on the concrete top surface. Despite the long strain gauges, measurements varied strongly and consequently measurements using a mechanical dial-gauge are planned for the future experiments. A typical strain versus number of load cycles curve for concrete and reinforcement is shown in Fig. 5. To observe the development of cracks, the crack pattern of the concrete surface was mapped several times during each test.

4.2 Test results

For all 9 specimens tested under fatigue loading failure was always fatigue fracture of rebars.

The strains and the deflection strongly increased within the first thousand cycles followed by a sequence of constantly increasing strains and deflection at a much lower rate (Fig. 5). Increasing strain and deflection was accompanied by crack propagation (Fig. 6): during the first thousand cycles, the visible crack propagation at the surface was strong, i.e. about 1-2 cm. New cracks developed in particular in the region with shear loading. In the final phase of the tests, cracks often changed direction and propagated parallel to the neutral axis. One specimen was subjected to static loading after 10 million cycles on two load levels each; no further crack propagation was observed before the specimen started yielding, the yielding and ultimate load of this specimen was equal to the corresponding values obtained from the static experiments.

A reinforcement ratio of 0.69% is considered to be a low value for decks of two girder bridges. Four fatigue tests have been performed using specimens with this reinforcement ratio. Failure always occurred by fatigue fracture of rebars and concrete under compression showed no spalling at the surface under fatigue loading. Thus, because of the low reinforcement ratio, the rebars are significantly more loaded than the concrete which, as a result, does not show any distress due to fatigue loading.

Specimens with the high reinforcement ratio showed no or only minor local fatigue damaging of concrete under compressive stresses. On two specimens, compression peaks such as those at location A in Fig. 6 formed cracks which were closed when the specimens were loaded and open when unloaded.

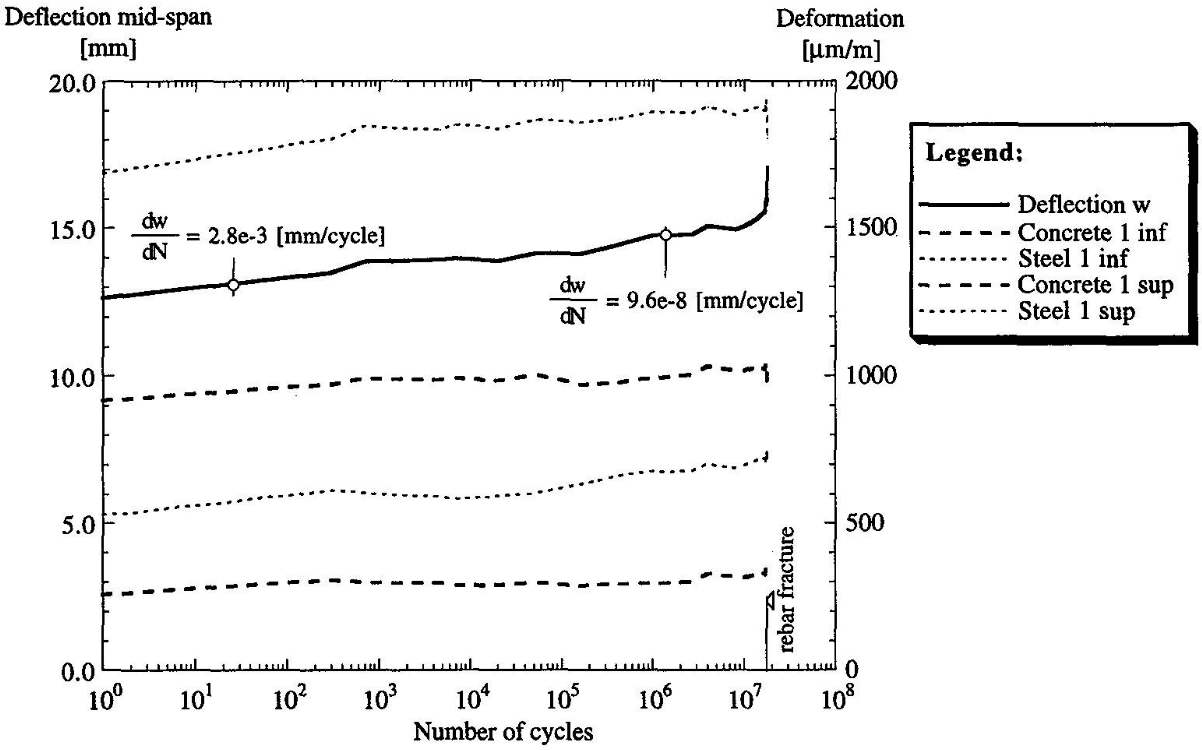

***Fig. 5** Fatigue behaviour of a specimen with 1.6% reinforcement.*

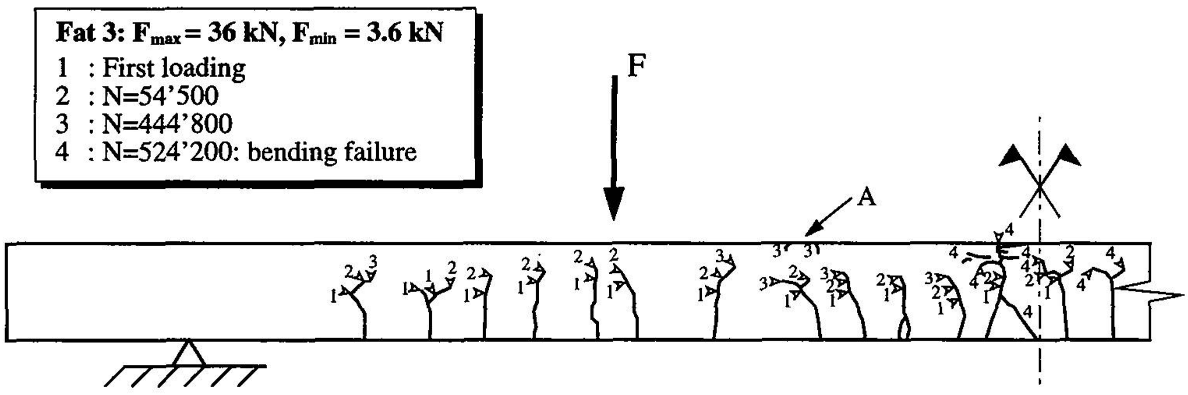

Fig. 6 Crack pattern of a specimen with 1.37% reinforcement as observed by visual inspection.

Although this research is still in its initial phase, the following initial observations can be made:

- Fatigue failure of concrete under compression is to be expected only if the stresses are extremely high, i.e. above 60% of cylinder strength f_{cc}. Concrete subjected to lower stresses may only fail after an extremely high number of cycles exceeding the amount a bridge deck may be subjected to during its service life. Also, the present results are considerably above the fatigue strength as suggested by present codes (Fig. 7). Recent research on plain concrete shows that fatigue failure under axial compression only occurs at a stress level above 60% of the cylinder strength f_{cc} [9]. These stresses are larger than the maximum possible stress level implied by the requirements of codes for structural safety and serviceability limit states.
- The detail category of 170 MPa for non-welded mild steel reinforcement is confirmed by the tests in the domain of high stress ranges, but appears to be conservative for stress ranges smaller than about 230 MPa (Fig. 7). There are indications that the ratio of minimum to maximum stress may influence the fatigue resistance of rebars in slab-like elements.
- In the case of concrete subjected to shear stresses, the test results suggest a high fatigue strength; e.g. no fatigue failure was observed for $\tau_{c,max}$ = 0.6 MPa and after more than 20 million load cycles. However, bridge slabs are also subjected to shear stress reversals; the fatigue strength may be significantly lower when compared to shear without stress reversals (Fig. 2b).
- Fatigue damage was only observed when the maximum fatigue load was greater than 60% of the ultimate static load F_u. The requirements for structural safety and serviceability limit states are by far not fulfilled at this high load level. For example, maximum deflection of one specimen was 1/170 of the span or about 6 times larger than the limit value of 1/1000 suggested by codes.
- Fatigue loading which results in stresses below the fatigue limit of steel and concrete affects serviceability (deflection, crack opening) but not fatigue safety.
- The tested beams have been designed and made according to principles of good fatigue design practice. If these principles are not respected, the fatigue strength may be significantly reduced. Factors affecting the fatigue strength of reinforced concrete include :
  - welded rebars including tack welding (Fig. 1)
  - strong corrosion and pitting corrosion of rebars resulting in defects and reduction in cross section.
  - deteriorating concrete (microcracking due to corroding rebars and freeze-thaw cycles)

Also, the fatigue action effect (including dynamic impact) may be amplified by higher axle loads and bad condition of the bridge deck surface.

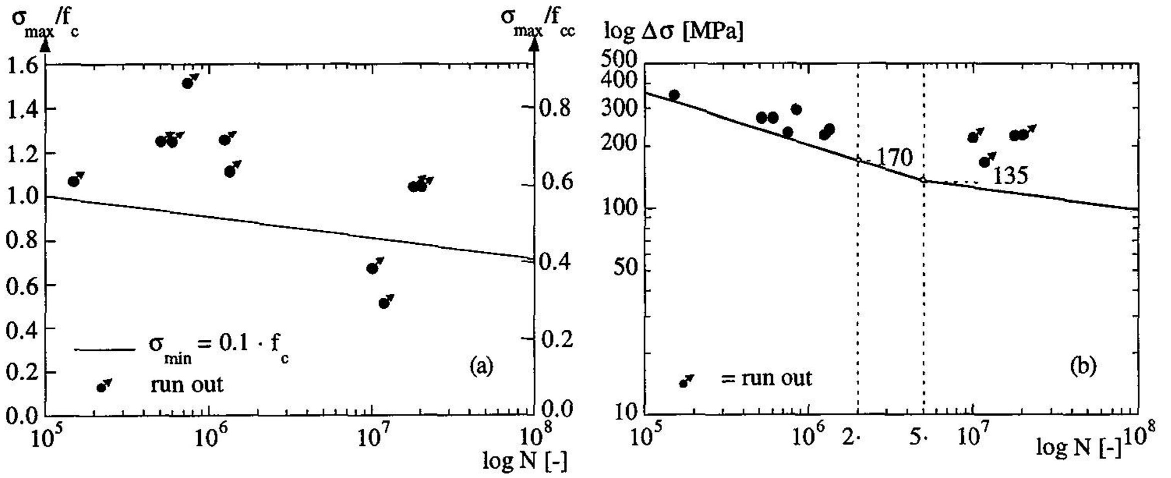

Fig. 7 Test results in comparison with fatigue strength of (a) concrete under compression and of (b) mild steel reinforcement according to [6] (f_c : nominal design compressive strength, f_{cc} : static cylinder strength).

5. Future research

To improve knowledge about concrete fatigue behaviour, additional slab-like beams will be tested. Beams with a span to depth ratio of about 8 will be subjected to eccentric 3-point-bending to enhance shear fatigue failure of concrete. In addition, the fatigue damage mechanism of plain concrete under compression is being investigated using microscopy and the effect of shear stress reversals due to moving wheel load will be studied.

Other research topics will include the fatigue behaviour of structural elements with welded reinforcement and deteriorating concrete as well as methods to determine the remaining fatigue life of deck slabs.

6. Conclusions

1. Based on current knowledge, the fatigue safety of existing reinforced concrete bridge decks may be a problem.

2. The fatigue safety of existing bridge deck slabs appears to be satisfactory if the principles of good fatigue design practice have been respected and the deck slab is in good condition.

3. Research into the fatigue behaviour of reinforced concrete slabs needs to be intensified and suitable methods to examine the remaining fatigue life must be developed.

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