Zeitschrift: IABSE reports = Rapports AIPC = IVBH Berichte

Band: 76 (1997)

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DOI: https://doi.org/10.5169/seals-57475

Resistance of Open Stiffener Orthotropic Bridge Deck Plates according to the Eurocodes

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Summary

This paper presents the structural safety and fatigue safety verification of an existing orthotropic steel bridge deck, thus enabling a comparison of Belgian code and Eurocode requirements. Using the Eurocodes to verify the structural safety of a typical open structure orthotropic plated bridge deck, with non-continuous open stiffeners, is straight forward and is rarely determinant for this type of structure. The high concentrated loads of the traffic load model do not seem to introduce overestimated stresses. However, fatigue verification is more complex and requires extensive calculation. The paper compares results of the fatigue evaluation using the four Eurocode fatigue load models and shows that non-continuous stiffeners are highly fatigue-sensitive.

1. Introduction

The load models, as defined by Eurocode 1 (EC 1) part 3 for road bridges [1], are characterised by the absence of impact factors and the high magnitude of the concentrated loads. Both of these characteristics have a negative effect on the design of orthotropic steel plated bridge decks. In addition, recent studies [2] and [3] have demonstrated the high fatigue sensitivity of typical orthotropic plates. This sensitivity arises mainly at strip-butt welds and stress concentrations around cope holes in cross stiffeners. Furthermore, orthotropic deck plates can be analysed using either simple methods or with sophisticated computer programs. In any case, even current finite element models are unable to account for all possible structural details, for example cope holes. These must be analysed separately, thus overlooking their behaviour as a part of the whole structure.

For the above reasons, the Universities of Liège and Ghent, together with the national and regional authorities in Belgium, have started a research programme in order to study the EC 1.3 load models [4] and to evaluate the consequences of their introduction for existing bridges. Orthotropic steel plate bridge decks are the part of this programme being conducted by Ghent University.

2. Open stiffener deck plates

Bridge designers and owners are often alarmed by the complex behaviour of orthotropic plated bridges as well as results of tests on isolated cross beams which have shown that cope holes cause high stress concentrations and thus low fatigue strength. As a result, orthotropic plated bridges are rarely adopted. Therefore, within the joint research programme on the effects of EC 1.3, attention is given to various types of orthotropic plates. As a first step, a selection of existing bridges with orthotropic plate has been made. All structures from this selection will be verified according to EC 1.3 and EC 2. This verification, together with thorough inspection, will eventually reveal if there is any problem at all. The existing structures were selected by considering the following important design parameters of orthotropic plates :

- open cross-section of bridge deck, with lateral girders or closed box section having shear diaphragms ;
- open or closed section longitudinal stiffeners ;
- longitudinal stiffeners welded between cross beam webs or continuing through cross beam webs with or without cope holes ;
- type of road surface, consisting either of several hydrocarbonic layers (no special requirements) or of an extremely thin sheet of appropriate material.

The continuous box section girder of the overpass at Vilvoorde, having continuous closed section stiffeners, built in 1976 and supporting heavy road traffic, is presently being examined. The verification of the Gentpoort-bridge at Bruges, a small movable bridge deck of 16.15 m span built in 1977, has been completed. It consists of an open structure, having two main lateral girders, closely spaced cross-beams with depth about half that of the main girders and many light, non-continuous longitudinal I-stiffeners. The latter are connected to the cross beams by 5 mm fillet welds. This arrangement is shown in the cross-section of Figure 1. Figure 2 shows a photograph of the lower side of the structure. The structure was designed using the loading scheme of the Belgian code, approved in 1993. From [5] this code is certainly not severe, compared to the design loads of EC 1.3.

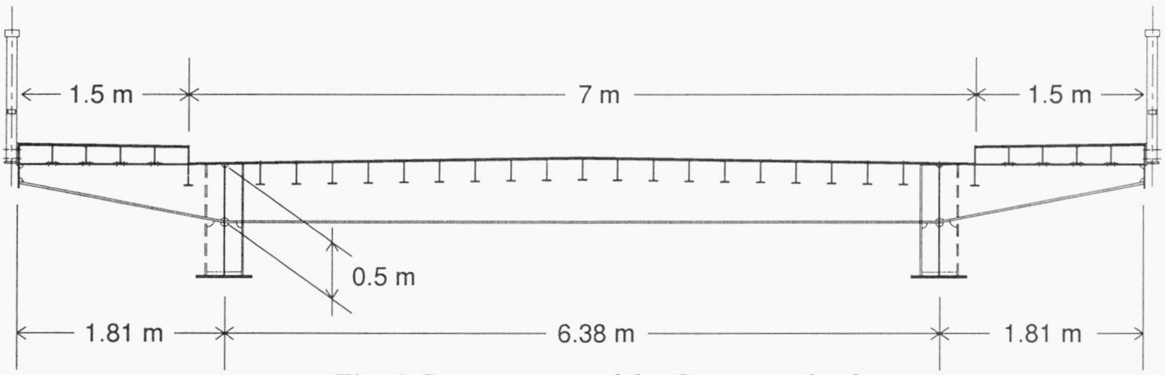


Fig. 1 *Cross-section of the Gentpoort-bridge*

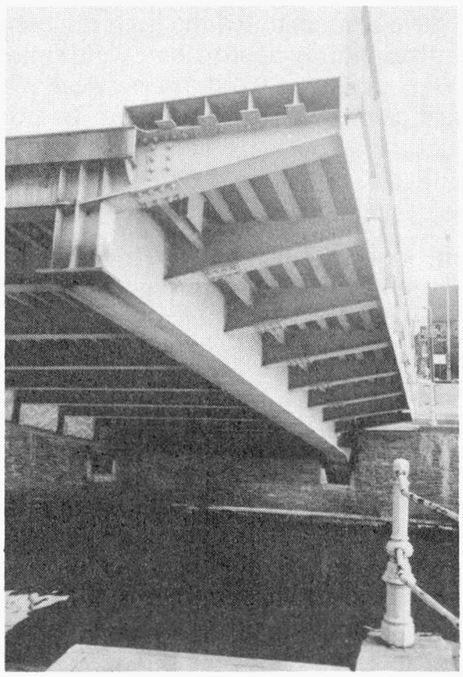


Fig. 2 *Photograph of the lower side of the Gentpoort-bridge*

The use of closely spaced open longitudinal stiffeners has a number of advantages. The deck plate is extremely light and is supported homogeneously. Compared to closed section stiffeners the number of longitudinal welds is double. However, thanks to modern automatic welding technique this does not affect the cost extremely. In addition the fatigue-sensitive cope holes in cross beam girders are left out. This has been adopted recently in important bridge constructions (Erasmus-bridge Rotterdam - Kronprinzen-bridge Berlin).

The road surface of the bridge being considered, consists of a thin sheet, glued to the steel structure. Hence there is no dispersal of concentrated loads through the pavement. This constitutes an unfavourable condition with respect to the effect of concentrated loads.

3. Structural safety

A full analysis, complying with the LM 1-scheme of EC 1.3, of the bridge deck being considered was carried out, using a fine mesh sophisticated FE-code (with Mindlin-elements) as well as by a current plane grid computer programme. The results of these calculations showed a rather good agreement. Figure 3 shows the deformation of the FE-model.

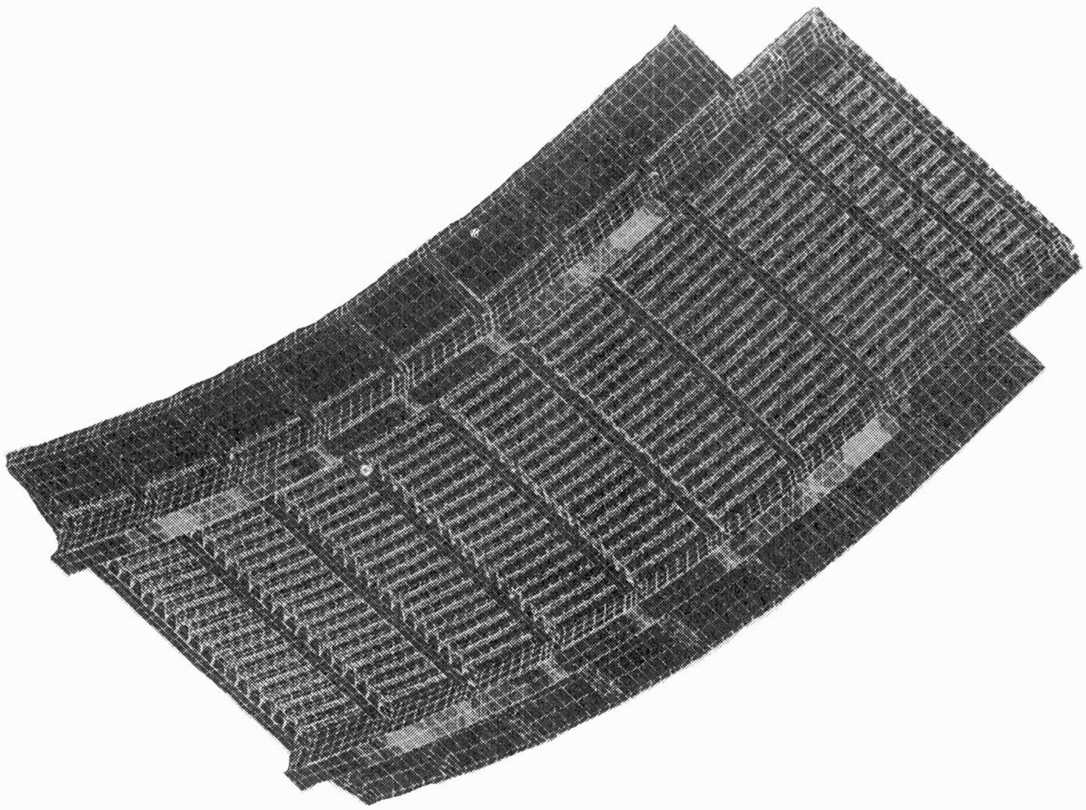
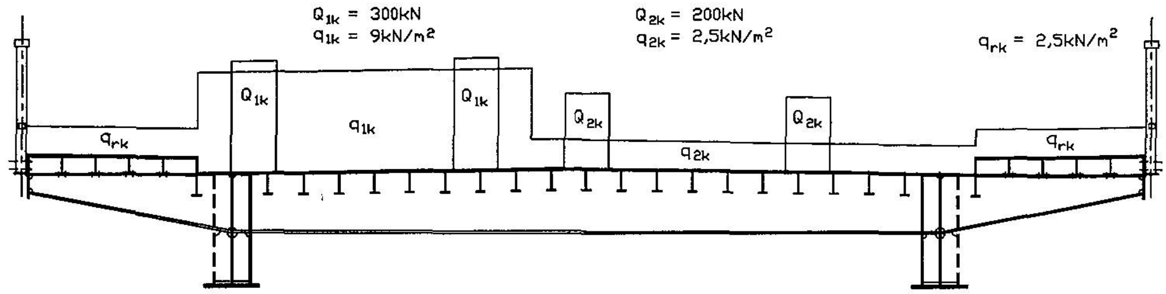


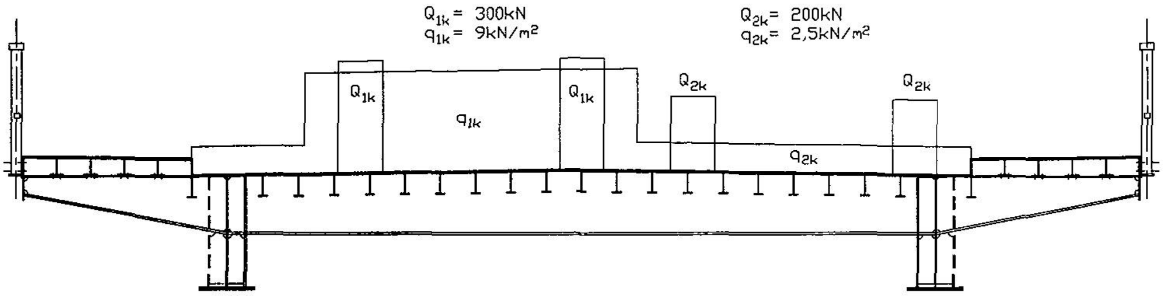
***Fig. 3** Deformation of the FE-model*

According to EC 1.3 there are 2 notional lanes, which must be located eccentrically on the bridge deck. Thus one main girder is loaded more heavily than the remaining one. Also, by placing the load model eccentrically, the cross beams and the longitudinal stiffeners can be loaded more heavily. Consequently the 5 positions, according to fig 4, across the bridge deck plate have been considered, whereas in fact real traffic is circulating in a less aggressive position. Similar positions were adopted for the LM 2-scheme.

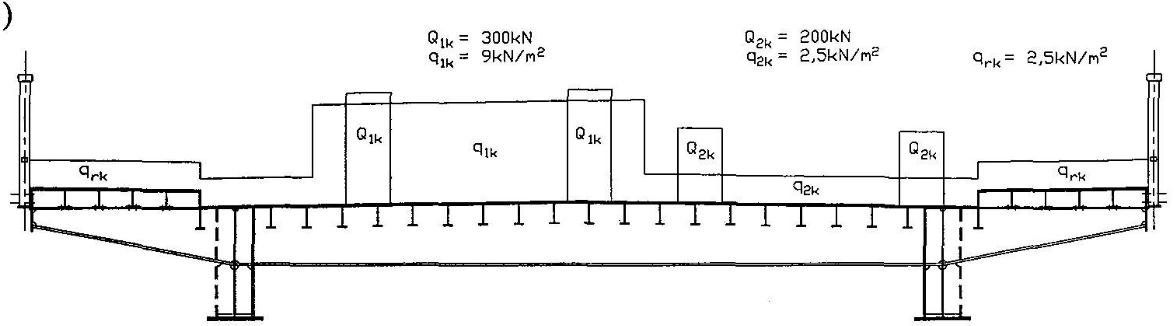
1)



2)



3)



4) : 2) with loading in 2 adjacent fields
5) : 2) with loading in 3 non-adjacent fields

***Fig. 4** The 5 positions of traffic considered across the bridge deck*

The verification of total stress at ULS, being far more easy than the use of actual strength criteria, Table 1 summarises the stresses which are the most relevant to the load carrying capacity. They include partial safety factors and dead load as well as the effect of the LM 1 and of LM 2.

stress ULS LM 1(Mpa)	stress ULS LM 2(Mpa)	location	direction to bridge deck axis	at cross section
$f_1 = 297$	$f_1 = 144$	lower flange lateral main girder	parallel	near mid-span
$f_2 = 201$	$f_2 = 134$	lower flange cross beam	perpendicular	near mid-span
$f_3 = 241$	$f_3 = 183$	lower flange long stiffener	parallel	near mid-span
$f_4 = 214$	$f_4 = 165$	principal stress long stiffener to cross beam	parallel	near 1/4 of bridge length

***Table 1** Total stress at ULS*

As expected the bending stresses in the different elements - main girders, cross beams and stiffeners - are the representative figures, together with the principal stress at the joint of stiffener and cross beam. In Figure 5 a detail of this joint is given. The reader will take notice of the cut-outs avoiding triaxial welds. These cut-outs introduce stress concentrations, which generate the altogether not too large stresses f_4. Anyway, in spite of the low value of the initial design loads, this construction satisfies all criteria for structural safety, the most relevant being the main girder stresses.

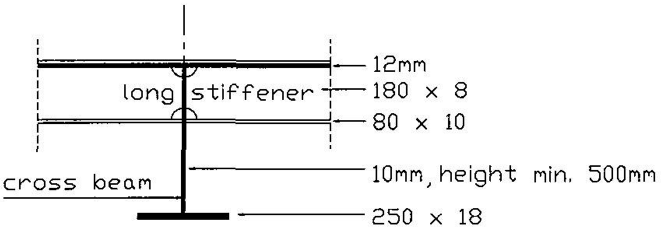


Fig. 5 Joint of stiffener and cross beam

4. Fatigue resistance

The tools for verifying fatigue resistance are similar to those for checking structural safety. However, this verification is far more complex. Instead of comparing total stresses to a single value of f_d, stress ranges at various fatigue-sensitive points must be compared to category-values defined in the tables of EC 3.1-9 [6]. Furthermore, 4 load models, some consisting of many vehicles must be considered. Taking into account the various fatigue details, more than 9 locations were analysed, as shown in Figure 6 and listed in Table 2.

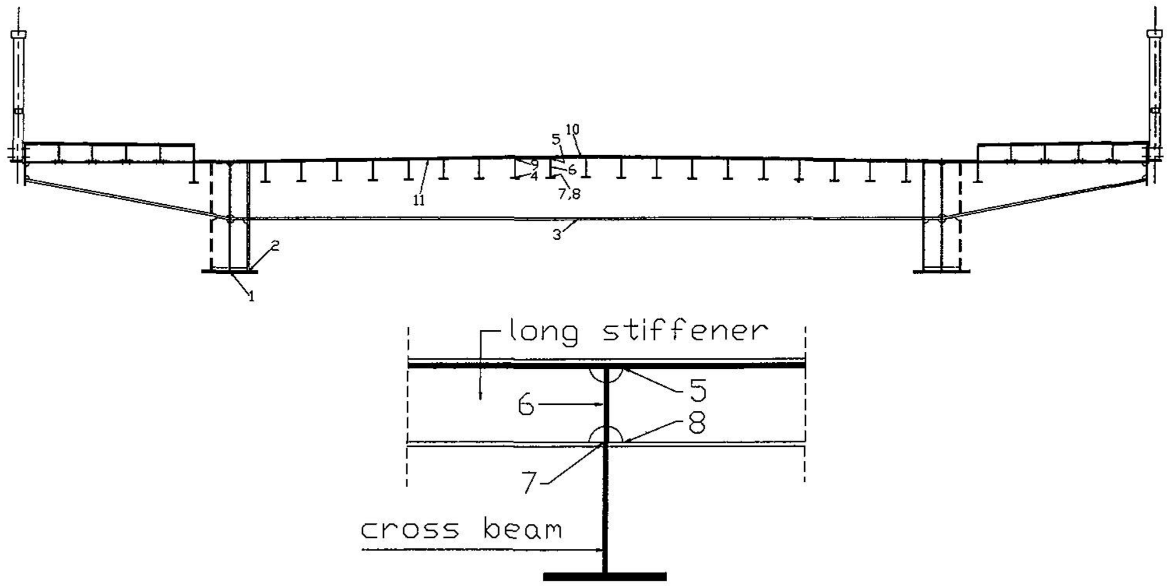


Fig. 6 *Points considered for verifying fatigue resistance*

Location	Fatigue detail	Category
1	lower flange welded main girder	112
2	main girder where connected to vertical stiffener	71
3	lower flange welded cross beam	112
4	lower flange welded stiffener in between cross beams	112
5	weld cutout stiffener to cross beam web near upper flange	71
6	fillet weld stiffener web to cross beam web (shear)	80
7	fillet weld lower flange stiffener to cross beam	45
8	weld cutout stiffener to cross beam web near lower flange	71
9	stiffener web welding to deck plate in between cross beams	112
10	cross beam web welding to deck plate	80
11	bending in between stiffeners of butt weld in deck plate	71

***Table 2** Points considered for verifying fatigue resistance*

The verification of fatigue resistance first requires a comparison of the constant amplitude (5 10^6 cycles) category value $\Delta\sigma_D$ to the stress ranges due to fatigue load models 1 and 2 (FLM1 and FLM2). Then, fatigue safety is represented by the following inequality :

$$\Delta\sigma_{FLM1\ or\ FLM2} \leq \frac{\Delta\sigma_D}{1.15} \tag{1}$$

If Equation (1) is satisfied, there is no fatigue damage for the given detail. The use of FLM1 has no special requirements. FLM2 however, consists of 5 lorries. It is not evident which lorry is determinant and the effects of the lorries are only slightly different. Since the main girders have a span of less than 20 m lorry 3 can be expected be determinant for these elements, whereas lorries 2 and 4 are determinant for the 1.9 m spaced cross beams and stiffeners. Table 3 summarises the effects of FLM1 and FLM2 and also indicates which lorry is determinant. The value of $\Delta\sigma_D$ is given for each case and the stress ranges which do not meet the criterion of Equation (1) are boxed.

Point	1	2	3	4	5	6 ($\Delta\tau$)	7	8	9	10	11
$\Delta\sigma_{FLM1}$	120.1	107.5	88.4	126.8	64.6	33.3	82.7	106.9	82.7	35.7	92.9
$\Delta\sigma_{FLM2}$	62.2	59.7	42.0	111.4	34.6	22.3	64.1	75.1	42.3	15.0	31.5
Lorry	3	3	2	1	2	4	2	2	4	4	4
$\Delta\sigma_D$	82.5	52.3	82.5	82.5	52.3	66.6	33.2	52.3	82.5	58.9	52.3

***Table 3** Stress ranges introduced by FLM1 and FLM2 ($\Delta\sigma$ in MPa)*

This results agree in general with those found in the literature. FLM1 is less aggressive than FLM2. In addition, for this particular case the results of FLM2 require closer verification on the main girders as well as on the longitudinal stiffeners, the latter both at its ends and at mid-span. The next step is the use of FLM3 and FLM4 for determining the structure's life-time. Stress ranges
$\Delta\sigma_{FLM3}$ due to FLM3 are easily found, whereas $\Delta\sigma_{FLM4}$ must be determined from the effects $\Delta\sigma_{i,i=1,5}$ of 5 lorries, each occurring with a percentage f_i according to the traffic type. Consequently $\Delta\sigma_{FLM4}$ is found from [7]

$$\Delta\sigma_{FLM4} = \left(\sum_{i=1}^{5} Ds_i^5 f_i\right)^{0.2} \tag{2}$$

whereas the structure's life-time, according to the fatigue detail being considered, is found from

$$\text{life-time} = 100 \text{ years} \frac{5\ 10^6 \left(\dfrac{\Delta\sigma_D}{\Delta\sigma_{LM}}\right)^5}{N_{obs}\ 0.67\ k_2} \tag{3}$$

N_{obs} (number of lorries per 100 year per slow lane) equals 200 10^6 for road category 1, 50 10^6 for category 2, 12.5 10^6 for category 3, although the denominator of this expression must be lower than 100 10^6. The aim is to find the road category which corresponds to a life-time of at least 100 years. The calculation results with FLM3 and FLM4 are summarised in Table 4.

Point	1	2	3	4	5	6	7	8	9	10	11
$\Delta\sigma_{FLM3}$	59.6	51.7	37.1	82.7	41.2	24.9	52.95	67.8	30.9	11.0	25.3
life FLM3 cat 1	25.3	5.3	272	5	16.5	684	0.5	1.4	682	22008	189
life FLM3 cat 2	75.7	15.8	661	5.5	18.2	755	0.5	1.5	752	53542	563
life FLM3 cat 3	303	63	2646	21.9	72.8	3021	2.1	6	3009	∝	2254
$\Delta\sigma_{FLM4}$ type1	43.5	42.2	29.1	79.7	24.3	16.2	37.4	44.7	30.0	10.7	21.2
$\Delta\sigma_{FLM4}$ type 2	39.9	38.8	26.7	79.1	23.7	16.0	35.8	43.3	29.1	9.7	21.3
$\Delta\sigma_{FLM4}$ type 3	35.5	34.6	24.1	78.8	23.1	15.9	34.4	42.1	28.3	8.4	21.4
life FLM4 type1 cat 1	122	14.6	923	6	230	5825	2.7	11	781	25755	459
life FLM4 type2 cat 2	562	66.5	3405	6.8	289	6845	3.8	14.2	1003	∝	1333
life FLM4 type3 cat 3	4057	471	23090	28	1312	28726	18.5	65	4681	∝	5224

***Table 4** Calculation results with FLM3 and FLM4 ($\Delta\sigma$ in MPa)*

Again in Table 4 the boxed values do not comply with the requirement of 100 years life-time. FLM3 appears too conservative for details 2 and 5. Hence in this case the use of FLM4 is more accurate for the main girder. From the design loading of this bridge, the structure should match the conditions for local traffic and road category 3. However due to stress ranges in details 4, 7 and 8 the structure doesn't even comply for FLM4 and road category 3. Clearly this is due to the severity of the fatigue category-values of these details.

After observing the number of lorries crossing the bridge, which consists of only one lane, the daily average N_{obsj} is 60 lorries, so N_{obs} equals

$$0{,}9 \times 365\ N_{obsj} \times 100 \text{ years} = 1.971\ 10^6 \text{ for observed category 4} \tag{4}$$

In these conditions, the life-time for FLM3 and FLM4 for road category 4 take the values from Table 5.

Point	1	2	3	4	5	6	7	8	9	10	11
life FLM3 cat 4	1919	400	16779	139	461	19161	13.6	38.2	19082	∝	14293
life FLM4 type3 cat 4	25728	2988	∝	176	8318	∝	117.5	412	29685	∝	33132

***Table 5** Calculation results with FLM3 and FLM4 for observed category 4*

We can see that FLM3 is too conservative for details 7 and 8. For FLM4 all details comply with the requirement of 100 years life.

According to FLM2 detail 2 has a limited life-time if intensive lorry traffic is considered, where FLM4 is more accurate. FLM2 as well as FLM4 predict the reduced fatigue resistance of the points 4, 7 and 8. Thus the calculation for all lorries of the bridge is necessary. This is a tedious operation, requiring special effort of the designer and high calculation cost. In addition, from this example it is shown that fatigue assessment is no longer a verification, but becomes a determining criterion. Concerning the bridge being verified, it satisfies all requirements for local traffic and the observed category 4. Inspection should be concentrated on the joints of longitudinal and transverse stiffeners and on the lower flange of the longitudinal stiffener. As expected, the use of non-continuous longitudinal stiffeners, welded in-between cross beams, reduces fatigue resistance.

5. Conclusions

From the detailed verification of the Gentpoort-bridge it was implied that structural safety, according to Eurocodes 1 and 2 is not determinant for open stiffener orthotropic bridge deck plates. On the other hand, the application of the load models for fatigue resistance and assessment shows that extensive calculations are needed. Furthermore, details showing fatigue damage according to FLM2, cause no problems according to FLM4. This proves that the complete application of EC 1.3 is imperative, thus increasing design cost.
For a small bridge FLM2 seems more accurate than FLM1. While verifying the structure's life-time it was found that fatigue assessment by FLM4 is more accurate than FLM3.

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