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Fatigue Resistance of a Steel Railway Bridge

Résistance à la fatigue d'un ponts-rail

Ermüdungswiderstand einer Eisenbahnbrücke

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SUMMARY

The paper describes different designs and optimization of a tied arch railway bridge. The design criterium for fatigue damage due to the several loading spectra, was chosen to be the spectrum corresponding to the largest local stress variation. In the case of a completely optimized structure, fatigue resistance becomes more limiting than ultimate limit state of load-carrying capacity.

RÉSUMÉ

Différents projets, ainsi que l'optimisation d'un pont-rail en bowstring sont décrits. Le critère du dommage causé par la fatigue due aux différents spectres de chargement, est choisi en fonction de la variation de contrainte locale maximale. Dans le cas d'une structure totalement optimisée, la résistance à la fatigue est un critère plus contraignant que l'état limite ultime de résistance.

ZUSAMMENFASSUNG

Dieser Beitrag beschreibt verschiedene Entwürfe und die Optimierung einer Stabbogen-Eisenbahnbrücke. Als Bemessungsbedingung wurde aus verschiedenen Ermüdungslastfällen derjenige mit dem grössten lokalen Spannungsunterschied ausgewählt. In einem vollständig optimierten Tragwerk wird die Ermüdungsfestigkeit und nicht die Tragsicherheit massgebend.

1. INTRODUCTION

Prevention of fissuration by fatigue is important, especially in the design of steel bridges that are to support large live loads. The numerous fatigue design codes [1] [2] are concerned with the analysis of fatigue details, which are mostly due to the welding of the bridge-elements. Therefore these details are only known at the actual construction stage. Hence, the designer has to estimate fatigue damage, aiming to avoid, but not being fully able to exclude all fatigue-sensitive construction details.

While designing the railway bridges at Landegem (Belgium), it seemed possible, with moderate success, to account for fatigue, and to draw some conclusions concerning stress limits to adopt. The tied arch bridges that are discussed further were built to suppress a local narrowing in the deviation canal of the river Lys. First two single track bridge decks are constructed on both sides of an existing bridge, the latter being replaced afterwards by a double track bridge.

2. BRIDGE DESIGNS

Different solutions were examined for the superstructure. Among them, the first classical solution consisted of the construction of two pillars on both future canal shores, supporting a central steel bridge deck with 4 m high plane web girders. Two side spans of steel-concrete girders would complete the bridge. For various reasons, such as the presence of existing butress-walls at the location of the piles, it was decided that three 86 m one-span steel decks were to be preferred. From experience-exchange with Deutsche Bundesbahn [3], the construction of tied arches with vertical suspension hangers was examined (fig.1). A prelimi-

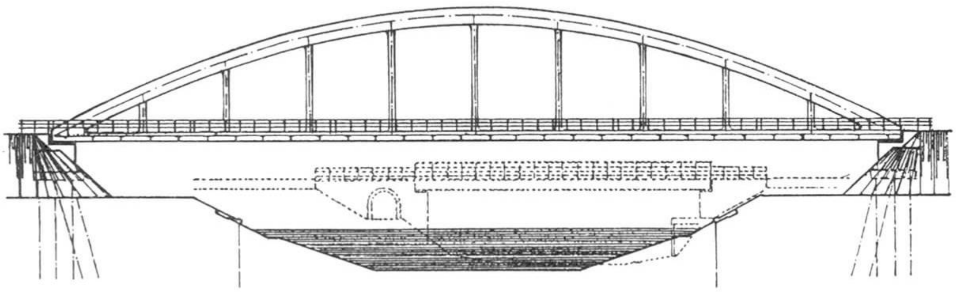

Fig 1 Tied arch with vertical suspension hangers

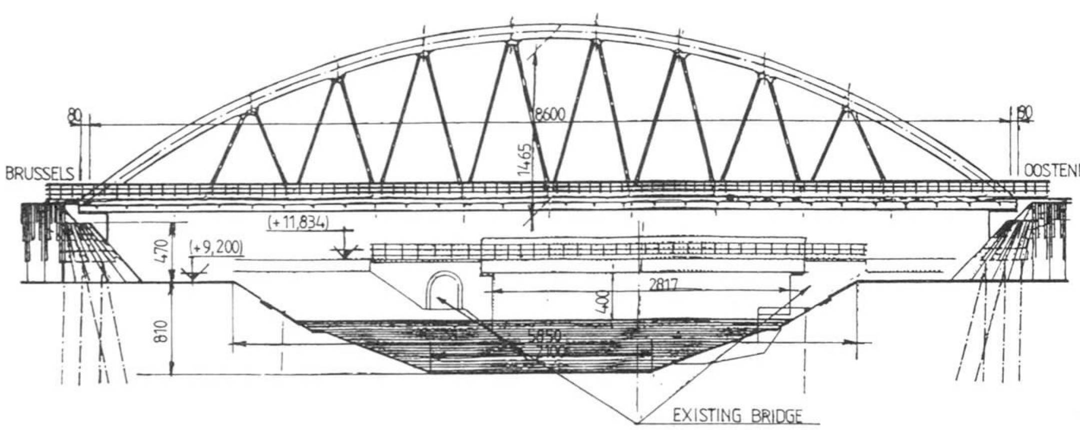

Fig 2 Triangulated hangers

nary analysis, based on the principles that are emphasized further, showed that total weight of construction steel for the three bridges would be about 1455 tons the deflections caused by the loading by the scheme UIC [4] being 1/995 of the span. An important improvement (fig.2) was made by disposing the suspension hangers between arches and tie-deck as triangles. Mainly due to the reduction of the bending moment-sum to be distributed between arch and deck [5], total weight of construction steel was now found to be 1275 tons, the deflections being 1/1260 of the span. However, certain loading cases show that, as the live loads represent some 70% of total loading, and as they induce compression forces in the hangers, the latter are submitted to a maximum residual compression of 221 kN.

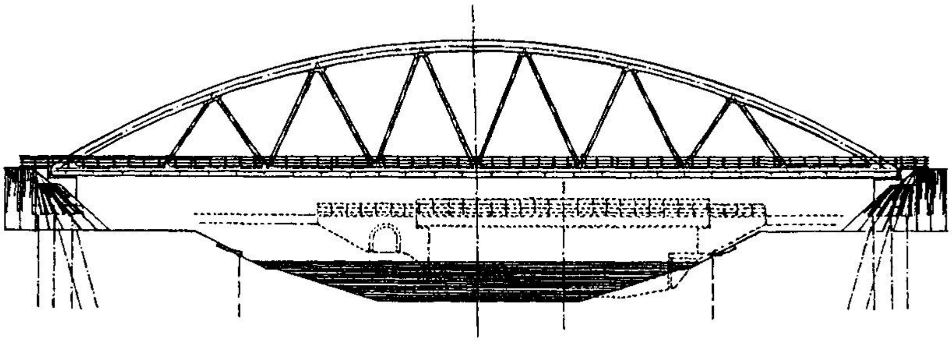

Fig 3 Further improvement of triangulated hangers tied arch

Eventually, the shape that was constructed (fig.3), a further improvement, had a total weight of 1120 tons, maximum deflection of 1/1824 of the span, and maximum residual compression in its hangers of 291 kN. However, it appeared that stress variations, as well as total safety to ultimate limit state, are distributed quite unequally along the bridge's span, the differences being about 35%.

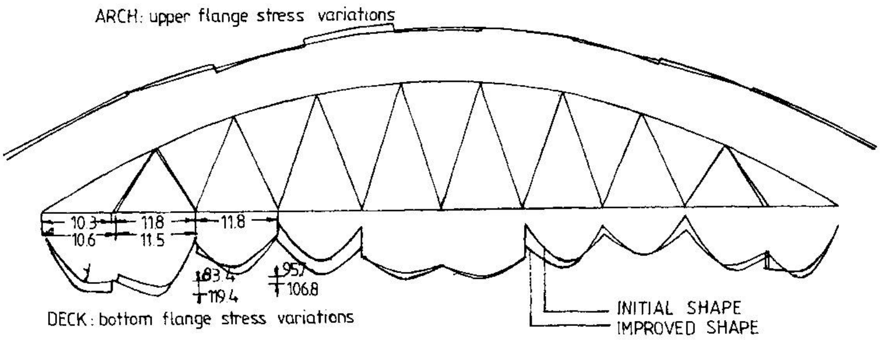

Fig 4 Reducing of stress fluctuations by optimizing

A closer optimizing of the case of a 115 m span bridge, showed that little modifications enabled to reduce stress variations and total safety fluctuations along the bridge's deck and arch's axis, to only 16% (fig.4).

3. FATIGUE STRESSES

All designs were calculated with the UIC loading scheme. The loading spectra to be adopted for fatigue verification, meaning the number of loading cycles, that guarantee a safety of 2.5 for a 50 years lifetime, depend on the influence length of each element, and were determined from UIC-leaflet 778 [2]. In this method, all spectra are reduced to the reference number of $2\ 10^6$ cycles, by modifying the allowable amplitude of stress variations, applying Wöhler's curve with an exponent of 3.75. The procedure is equivalent to ECCS's, the sequence of calculations being reversed. For design purpose, it was assumed that the determining spectrum for the deck's main girders, corresponds to an influence length equal to the distance between suspension nodes. These girders consist of flange plates welded to webs, to which are again welded the webs of transverse stiffeners. Consequently they belong to ECCS-class 112. Using Wöhler's curve, the allowable stress variation for the adapted design spectrum becomes $\Delta\sigma_{AD}$= 125 N/mm^2. Hence, during design the additionnal local bending between transverse deck stiffeners and the overall extension of main girders were disreagarded.

In a similar way, it was assumed that the arch's box section (ECCS-class 100) spectrum corresponds to the total arch length, thus disreagarding any local variation due to suspension node's reactions. Both assumptions were found to be

design	1	2	3
arch $\Delta\sigma$ (dam.)	151.2 (1.345)	136.1 (0.899)	78.67 (0.115)
deck $\Delta\sigma$ (dam.)	123.2 (0.947)	102.3 (0.472)	114.9 (0.729)

Table 1 Stress variations and damage

accurate since, after complete analysis of each structure the stress variations (in N/mm^2) from table 1 were found, the numbers between brackets being the total fatigue damage due to all stress spectra.

Table 2 summarizes the values of γ_m, the material's partial safety factor for ultimate limit state, that was determined for both cases of steel quality Fe37 and Fe42.

design		1	2	3
Fe37	deck	1.050	0.942	0.914
	arch	1.017	0.868	1.319
Fe42	deck	1.290	1.183	1.147
	arch	1.249	1.090	1.655

Table 2 Safety factor u.l.s.

As can be seen from table 2 Fe42 had to be used in the final design. However as it was emphasized in member 2, a more accurate choice of the deck's sections lengths would have permitted the use of Fe37, thus making predominant the criterium of fatigue.

More important fatigue details are present in the shorter elements of the bridge deck, namely the orthotropic plate with closed section stiffeners. Evidently, obtaining the same safety for an equal lifetime, will submit these elements to a many larger number of loading cycles. The stiffeners have delicate welding details, such as the joining of their webs with the deck plate (joint I), and the continuous passage through the transverse stiffener's web (joint II). The latter was shaped as a cardiod [6] to avoid stress concentration. The welding length of joint II is determined by the allowable stress variation amplitude. Both joints I and II belong to ECCS-class 80, the local spectrum allowing a stress variation of 60 N/mm^2. During design it was assumed that the local bending stress variation spectrum is predominant. A full analysis showed the damages summarized in table 3. Hence, allthough there appears some greater importance of the local spectrum, the contribution of damage due to general effects cannot be neglected.

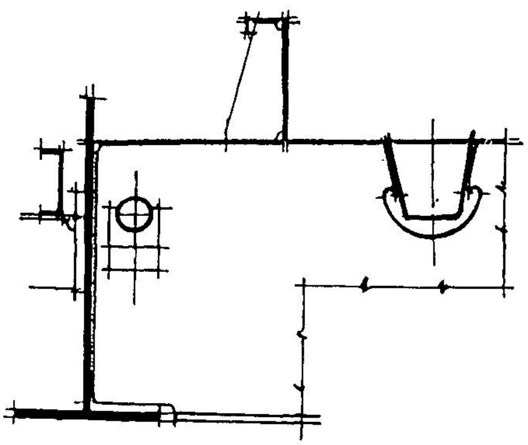

Fig 5 Main girder and continuous closed section stiffener

joint	local bending	general bending	total damage
I	64%	24%	88%
II	31%	21%	52%

Table 3 total damage closed stiffener joints

Consequently, the design values for the local spectrum have to be choosen very low. In the case that was discussed 60 N/mm^2 proved to be an accurate choice.

4.FATIGUE TEST

In order to verify the welding quality and the accordance with theory, a 1/1 scale specimen of the orthotropic deck plate was tested on fatigue. A sample, shown in fig.6, of 3.6 m length, consisting of a deck plate with 2 closed section stiffeners, was submitted to pulsating bending. In all critical details strain gages were attached. A stress variation of 60 N/mm^2 was realized in these points. Transverse weldings were present and the passage through the transverse girder's web was simulated by the welding of stiffening plates 8 mm of thickness. Stresses' variations went up to 160 N/mm^2 at the mid-section's bottom. As the test was performed at constant amplitude, and since the critical spectrum of an element of 3.6m is far beyond the damage limit, no fissuration whatsoever might appear. Since, according to ECCS, beyond 5 10^6 cycles no damage does occur, a safety of 1.5 was adopted, the test was stopped after 7.5 10^6 loading cycles. No damage or fissuration was detected after this test.

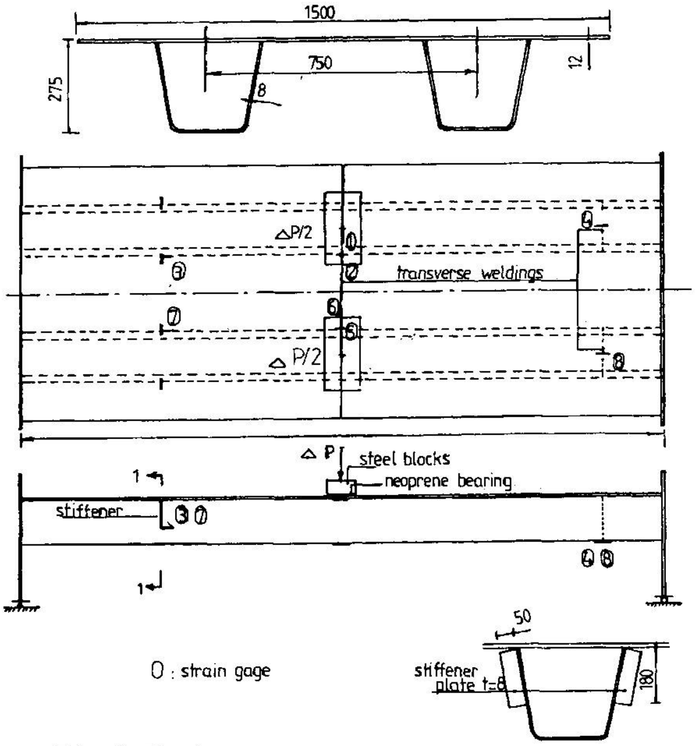

Fig 6 Fatigue test

5. CONSTRUCTION

This paper does not deal with the actual construction of the three bridges. However, it should be mentioned that the bridges were assemled by placing first the decks, followed by the arches, to which the suspension hangers were allready bolted. The gaps between the deck's nodes and hangers, due to the deflection caused by self-weight, were compensated by regulating the level of temporary bearings on construction towers.

One of the completed single-track bridges can be seen from photo 1. As yet, a testing programme for the completed bridges has not taken place. In the future some interesting data may be expected from it.

6. CONCLUSIONS

It was implied that, while designing even moderate sized steel railway bridges,

Photograph 1 Completed single-track bridge

fatigue resistance plays an important role.

Accurate use of verification codes, based on the most wide spectra enables to choose the allowable stress variations for different bridge elements. In those cases were optimized structure geometry has been achieved, prevention of fissuration caused by fatigue, becomes a more strict criterium than ultimate carrying capacity at limit state.

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