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Composite Plate Girder Bridges: Safety and Serviceability

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Summary

The structural behaviour of composite bridge decks, made of two plate girders and a reinforced concrete slab, is considered on the basis of two design examples and some parametric studies developed with a nonlinear numerical model. The effects of the bracing system, external prestressing, cracking of the concrete slab and the postbuckling behaviour of the webs are discussed for ultimate (ULS) and serviceability limit states (SLS).

1. Introduction

In the last few years, composite bridge decks made of two plate girders only with thin webs and thick flanges (up to 150mm at supports) have been extensively adopted [1,2] for both highway and railway bridges. The webs have very often transverse stiffeners only and its postbuckling strength is taken into consideration in design. Class 4 sections, according to the new eurocodes EC3 and EC4 [3,4] are generally adopted for medium and long span bridges. Special problems at ULS should be considered, namely bending moment redistributions due to effects of cracking of the concrete slab or due to web buckling. The first problem is considered, under different approaches, by present design codes [4 to 8]. The effect of local plate buckling on cross section properties is considered by most of the codes for the section analysis only and not for the structural (global) analysis. Also the effects of the bracing system on the stresses induced on the girders by vertical loads (permanent and live), are currently neglected. Some results for these effects are presented in this paper, and the possible advantages of adopting externally anchored prestressing schemes in composite bridge decks is discussed.

2. Design cases

Two railway composite bridge decks designed by the authors have been selected to highlight present design criteria at ULS and SLS. The first design case (Fig. 1) consist on a 3 span deck (main span 55 meters) one track railroad. Transversely, the deck is asymmetric due to specific site constraints; however, the girders where located to optimize the transverse load distribution under rail loading.

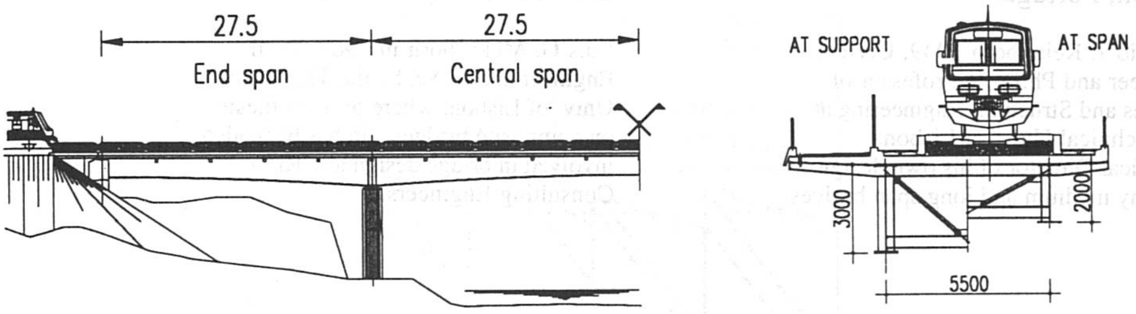

Fig.1 Longitudinal and cross section of a composite railway bridge over river Ave, Portugal

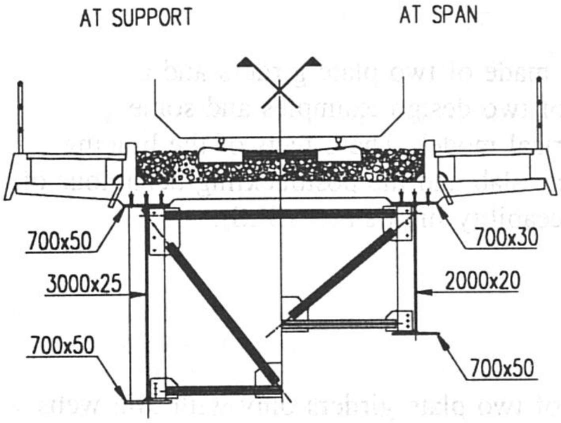

Fig.2 Simplified section

For the parametric studies, the section was simplified (Fig. 2). The bridge deck was designed to be erected in 3 stages, first the end spans plus 1/5 of the mid span at each side, and finally the central part (33.0m) is lifted from the river.

The second design case, refers to an incrementally launched railway deck - the Northern Approach Viaduct to the Tagus Suspension Bridge in Lisbon. A composite bridge deck (Fig.3), almost 1000m long, was designed to be incrementaly launched from both ends in two parts (Fig. 4).

The superstructure is a continuous deck, with typical 76m spans from the northern abutment to pier P14 (392.3m), where an expansion joint exists, and from the transition pier to the suspension bridge to P14 (526.6m). Details of the conceptual design and of the erection scheme are presented in [9]. For the parametric study a 3 span continous beam model (60+80+60) was considered.

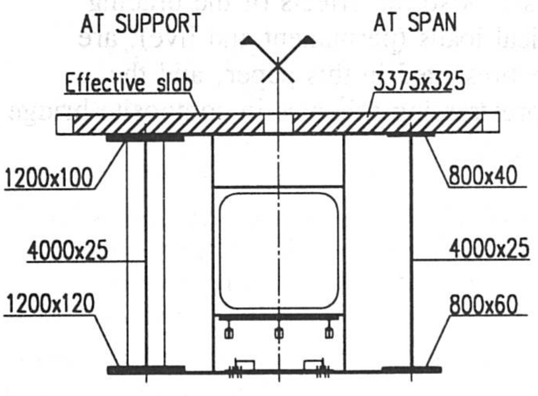

Fig.3 Viaduct cross section

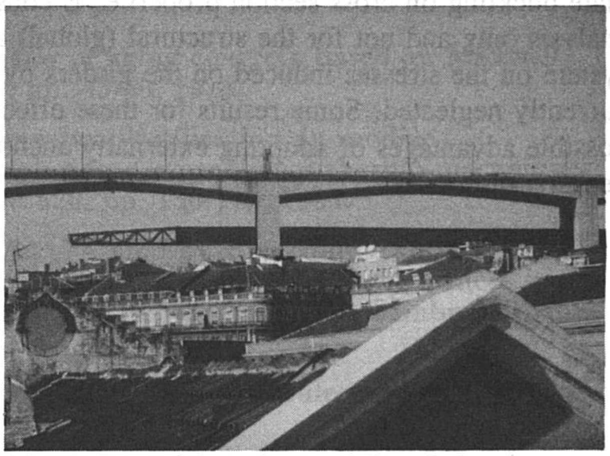

Fig.4 Launching phase of the viaduct

3. Effects of the bracing system

The main girders in a composite bridge deck are connected at the lower flange level by a bracing system (Fig. 6). If a 3D model is adopted for the structural analysis, the part of the overall bending (even under symmetric loading) taken by the bracing system may be evaluated. In Fig. 5 one shows the bracing system of the composite bridge deck shown in Fig. 3 and a typical result of the induced stresses. For the diagonal bracing, horizontal transverse loads are introduced at the "gussets". These loads produce transverse bending stresses at each flange; of course the two transverse bending moments are self equilibrated at the overall section. For the section analysis at ULS of the deck, it is acceptable in design pratice to assume some redistribution of the stresses at the lower flange.

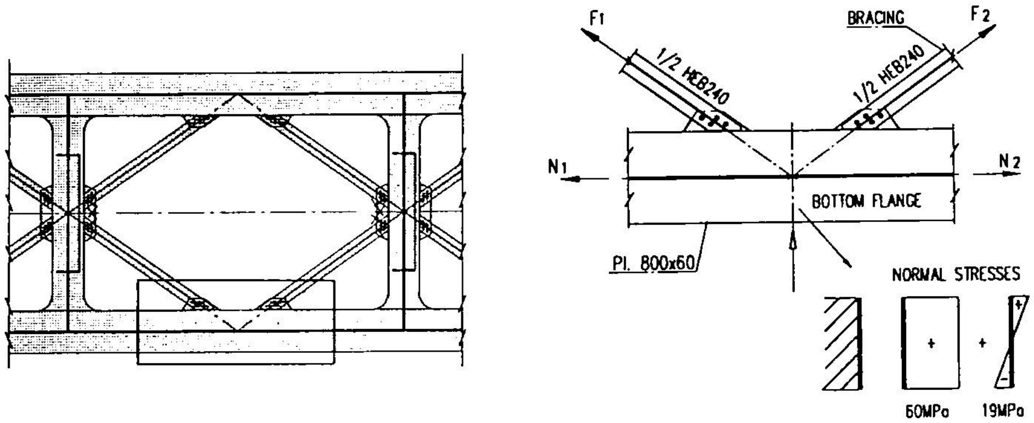

Fig.5 Normal stresses at lower flange due to horizontal bracing

4. External prestressing

External prestressing in composite plate girder bridges, has been adopted in several design cases. The main advantage is a reduction of the amount of steelwork required to achieve the resistant capacity at ULS. External prestressing shall be considered as an external force and the beneficial effect due to the increase of the prestressing force during the loading stages may be neglected. That compensates for some reductions of the prestressing effects due to geometrical nonlinearities. The main disadvantage of the classical external prestressing scheme, where the cables are attached to the steel structure, are the high compression forces induced at the girders requiring complex details at the anchorages. This disadvantage is eliminated when an externally anchored prestressing scheme is possible. The example in Fig. 6 shows an external prestressing scheme solution developed for the railway deck of Fig. 3. The cables were designed to be stressed at two stages: immediatly after erecting the steel structure, by incremental launching, and after casting the deck slab. In Fig.7 one shows the benefitial effect of the external prestressing in reducing the maximum stresses at the flanges. With this scheme (case B) it would be possible to adopt 80mm thick plates at the flanges at the support sections instead of 120mm plates required by the conventional composite girder (case A). A reduction of about 25% in the total amount of the steelwork could be achieved by the externally anchored prestressing scheme. Of course, part of these savings are canceled by the cost of the prestressing cables, anchorages and deviators. For the present design case the conventional solution (without external prestressing) was prefered mainly because the execution time was shorter.

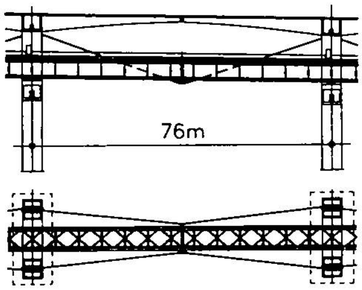

Fig.6 External prestressed solution

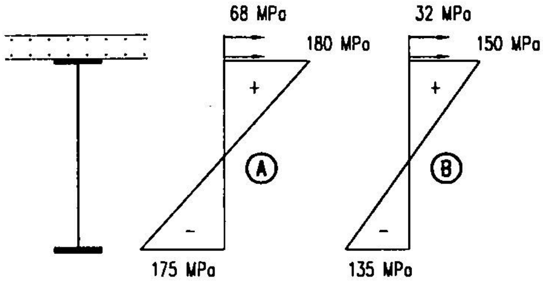

Fig.7 Effects of the prestressing scheme

5. Nonlinear numerical model

For the model developed, the nonlinearities taken into consideration are restricted to cracking effects in the slab and local buckling at the steel section. The slab is considered as an axially loaded bar element under pure tension or compression. This assumption is acceptable for medium to large spans, where deep girders are adopted. The constitutive relationship [10] for the slab under tension is shown in Fig. 8. The tension branch is similar to the one proposed in Jan. 1997 Draft of Part 2 of EC4 - Annex L, for the "Effects of tension stiffening in composite bridges".The finite element considered is a bar element with only two degrees of freedom at each node (vertical displacement and rotation), and the linear system of equations is solved by the Gauss Substitution Method. At each step of the loading process, a new stiffness matrix is evaluated, according to an iterative procedure, based on the Newton-Rapson technique. This new matrix is updated taking into consideration the effective steel section in the postbuckling range (based on EC3 formulae), and cracked or uncracked properties of the concrete slab. The numerical model was developed for continuous plate girder bridges, with a composite deck, taking into consideration the evolution of the static scheeme during construction.Span by span, incremental launching or other execution methods, may be considered. At each step, steel or composite (full interaction) section properties are evaluated depending on the phase at which the slab is casted.

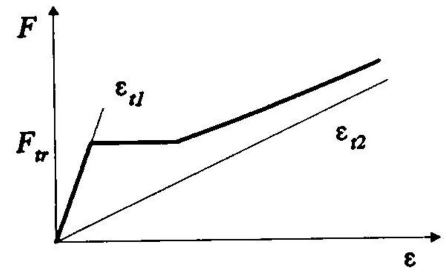

Fig.8 Constitutive relationship for concrete slab under tension

6. Cracking and time dependent effects

Most of the present design codes for composite bridge decks adopt a limit state format and allow a linear uncracked elastic model for the structural analysis. If so, bending moment redistributions at ULS are acceptable (up to 10% in BS 5400 for class 4 sections). The main differences between present design codes concerns the requirements for evaluation of cross section properties for the global analysis. In BS 5400 [5] it may be assumed the slab is cracked at 15% distance of the span each side of an internal support. Then, no redistribution is allowed. In other codes, like the French instruction [6] and the Swiss code [7] uncracked sections are assumed for the global analysis. If concrete is fully neglected near support sections, tension stiffening effects are not taken into account. In both EC4 [4] and the new Spanish recommendations for the design of composite bridges [8], an elastic analysis is

allowed but with reduced moments of inertia to account for cracking effects.
Typical results are presented in the following Table for moments of inertia and bending moments evaluated according to the codes mentioned above. It may be concluded from Table 1, that the influence of the different approaches is much greater for I than for M.

Code	Section	Approach	I(m^4) (for n=6)	M(kNm) Live Load
EC4	Support Span	cracked uncracked	2.644 2.039	-130942 69057
BS5400	Support Span	cracked uncracked	2.424 2.039	-128675 71324
Circ n° 81-63,1981 SIA 161	Support Span	uncracked uncracked	3.858 2.039	-140825 59174

Tab.1 Comparative results for I and M (for live load) according to several design codes

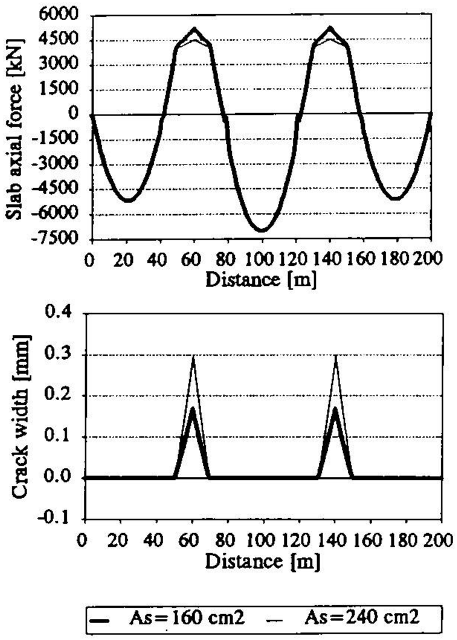

Fig.9 Influence of reinforcement on the slab behaviour

When evaluating cross section properties, like I, the modular ratio $n=E_s/E_c$ between the elastic modulus of steel and concrete, time dependent effects are generally taken into consideration in the present codes [4 to 9] by multiplying the modular ratio for short term actions n_o, by a coeficient $[1 + \psi\varphi(t, t_0)]$. Here φ is the concrete creep coefficient and ψ is dependent on the type of action involved. For example, ψ =0, 0.55, 1 or 1.6 in EC4 [4] respectively for short term, permanent, shrinkage effects and prestressing by imposed deformations. In the French instructions $\psi\varphi=2$ for all permanent actions and in the new Spanish recommendations a more sophisticated age adjusted effective modulus method is proposed. The nonlinear model described was used to develope some parametric studies for ultimate load of composite class 4 sections, and to investigate bending moment redistributions due to cracking and local buckling effects.

By taking the design case of Fig. 3, the influence of the concrete strenght class, thickness of the slab, amount (density) of reinforcement, bond action of the reinforcement and the sequence of the erection scheme were investigated. As a general conclusion, for the ULS of the composite sections, most of the parameters refered to above have very little influence. In what concerns SLS, namely crack widths, the most relevant parameter is the amount of reinforcement (Fig. 9) as also the sequence of the erection scheme in what concerns limitation of crack widths and deformability. In what concerns bending moments, the results obtained by elastic uncracked models are about +5% at support and -10% at span sections compared to the ones obtained by the nonlinear model described in section 5. The results obtained for ULS by the elastic cracked model, as proposed by EC4, are less than 1% different for bending moments and +5% for displacements compared to the ones obtained by the nonlinear model.

7. Local buckling effects

Another aspect is related to code requirements concerning the effects of local buckling in composite bridge decks. These effects are only considered when evaluating cross section resistance (i.e at section level) and not for the structural (global) analysis. In composite plate girder bridges with Class 4 sections, the webs are very often in the postbuckling range at ULS. So, a redistribution of bending moments shall be expected. In Fig. 10 one compares results obtained by the nonlinear model and from EC4 model, for the support cross section of the bridge deck shown in Fig. 2.The difference in the effective width of the web is +11% and at the ultimate moment is only +2%. The force at the slab is 22% lower by EC4 mainly due to cracking effects. In Fig.11 the influence of the phase at which composite action is considered is highlighted for the cross section of Fig. 2, under increasing imposed curvature.

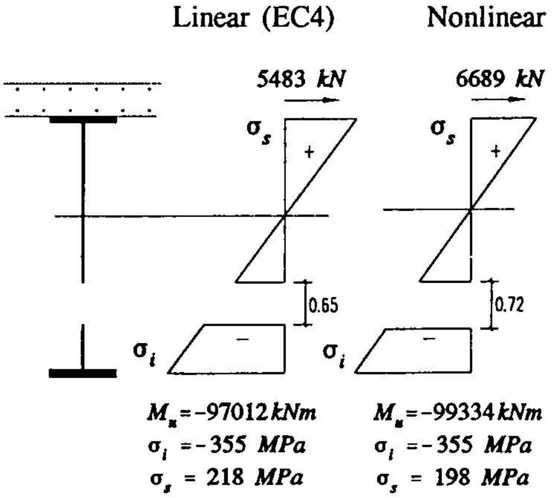

Fig.10 Ultimate moment obtained from linear (EC4) and nonlinear analyses

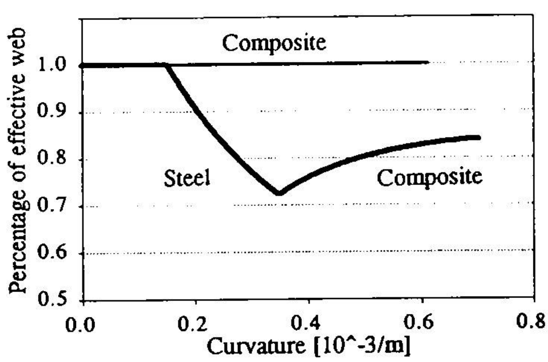

Fig.11 Influence of the composite action on the effective width of the web

8. Conclusions

Safety and serviceability problems for composite bridge decks were discussed on the basis of two design cases. A nonlinear model was adopted for parametric studies and results were compared with the ones obtained by a draft for EC4 (Part 2).

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