Zeitschrift: IABSE reports = Rapports AIPC = IVBH Berichte
Band: 999 (1997)

Artikel: Composite bridges: ductility versus brittleness
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DOI: https://doi.org/10.5169/seals-994

Composite Bridges: Ductility versus Brittleness

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Summary

Current design methods for composite bridges are stress oriented, which includes the risk of in-built brittle behaviour. Starting with an example which shows that the traditional approaches can lead to unsafe structures, this paper presents a strain oriented design method, suitable for ductility evaluations. Since cross-section ductility is governed by the response of steel compression elements, their analysis requires special attention: a strain oriented approach is introduced for the calculation of load shortening curves of unstiffened and stiffened compression plates.

1. Introduction

Current design codes for composite structures generally use a system of classes to define the ductility of cross-sections, the class of a section being a function of its geometry, stress distribution and the steel grade. The class of the section dictates allowable methods of global analysis (including moment redistributions limited to fixed values) and resistance calculations. In most composite bridges the width to thickness ratios of the steel flange in compression or the part of the steel web in compression is such that the corresponding cross-sections belong to the least ductile class according to the classification system of current design codes (class 4, slender cross-sections, in [1]). Following the approach from these codes, structures with slender cross-sections require elastic global analysis and resistance moments are calculated by using the theory of elasticity and by taking into account the effects of local buckling [1]. If a cross-section shows an elastic behaviour as assumed in the analysis method, then its failure mode is brittle: due to instability phenomenons of the steel sections at internal supports an abrupt decrease of the resistance moment following the attainment of the peak value is observed for increasing rotations (curvatures) (Figure 1a). Cross-sections in the span, on the other hand, are able to undergo large rotations maintaining the maximum value of the resistance moment, they are ductile (Figure 1b).

Brittle structures are very sensitive to the uncertainties of action effects, including those due to creep, shrinkage, temperature, settlement and earthquakes [2], which is illustrated for a bridge with the span arrangement and the cross-sections from Figure 1. From the geotechnical study it can be concluded that no settlement is to be expected for the end supports. The load arrangement according to Figure 1 governs the design of the cross-section A-A following the design philosophy of current codes [3, 4] (Figure 2a). However, one year after the completion of the bridge a settlement of δ=0.1 m of one of the end supports is observed (Figure 1). The governing load arrangement including the effect of the settlement results in an interaction between shear force, V, and bending moment, M, such that the support cross-section A-A no longer reaches the required structural safety level according to the codes [3, 4]: the design action effects are bigger than the design resistance (Figure 2b). Due to the expected behaviour of the support cross-section (Figure 1a), the structure is potentially brittle after the settlement, which means that it can fail abruptly without previous warning. In order to avoid the design of brittle structures, the traditionally stress oriented design methods are substituted by a strain oriented reasoning [2].

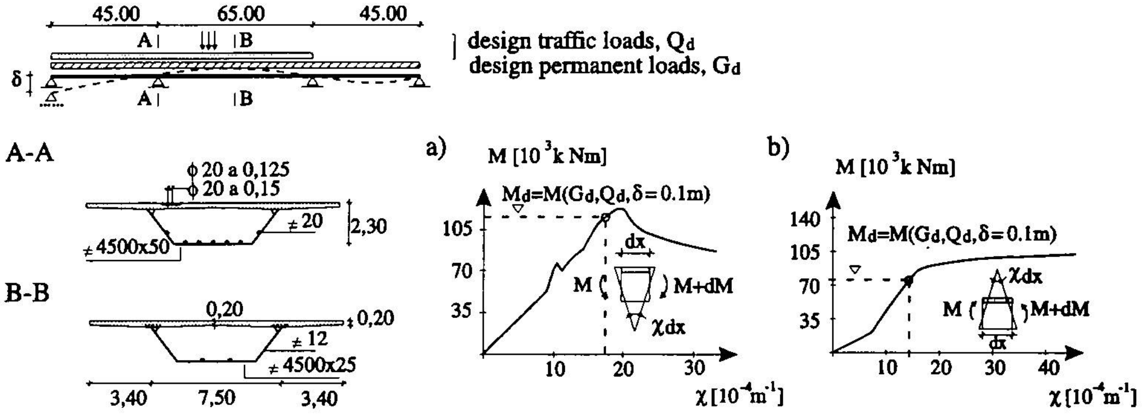

Fig. 1 Composite box girder bridge with typical cross-sections and their behaviour in terms of moment vs. curvature, a) at the support (section A-A), b) in the span (section B-B).

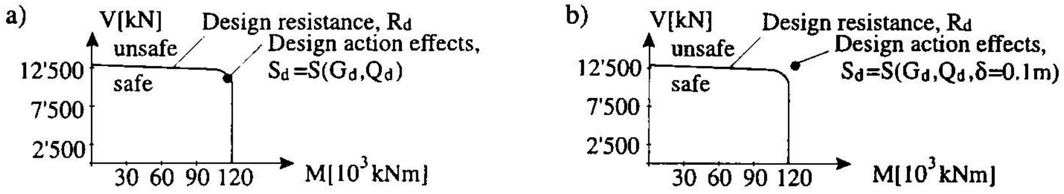

Fig. 2 Sensitivity of brittle cross-sections to the uncertainties of action effects: structural safety of the support cross-section A-A from figure 1, a) without, b) with settlement.

2. Strain oriented reasoning

2.1 Overview

2.1.1 Response of cross-sections

Composite box and plate girder bridges subjected to bending moments are composed of different steel plated elements and a concrete slab. A cross-section at an internal support (hogging bending), for example, is composed of the steel compression flange, the reinforced concrete tension flange and the steel webs subjected to bending and axial forces. It is assumed that the behaviour of the different elements composing the cross-section can be modelled independently in terms of load strain curves in the case of the compression- (chapter 3) and tension flanges, and as moment-curvature diagram in the presence of an axial force, in the case of the webs [2]. If the response of the elements composing a cross-section is known, the response of the whole cross-section can easily be obtained: considering different states of strain, the moment-curvature diagram can be established point by point following an iterative calculation procedure [2, 5].

2.1.2 Non-linear analysis

If the behaviour of the bridge cross-sections can be modelled in terms of moment-curvature diagrams (including falling branches in the M-χ diagram of cross-sections in the hogging bending region), the aforementioned redistributions of bending moments (chapter 1) according to these diagrams can directly be taken into account. For this, a non-linear global analysis must be carried out for the calculation of the internal forces and moments. For failure, there is no need to introduce a first yield criteria as usual in bridge design. Allowance can be made for strains in structural steel elements of beyond those corresponding to the yield strength of the steel, ε_y. This implies necessarily additional serviceability checks [4, 5].

A non-linear analysis carried out for the example from chapter 1 shows that in spite of the settlement of δ=0.1 m of one end support no structural failure is to be expected (Figure 1). For the failure criteria from [4, 5], settlements of up to δ=0.3 m could be justified by a non-linear analysis [6]. However, even though non-linear analysis is possible with modern numerical tools it is only considered viable in special cases. Therefore, an alternative design method which is aimed at the practising engineer for everyday use is proposed in 2.1.3.

2.1.3 Ductile structures
If instability phenomenons of the steel member govern the behaviour of a composite cross-section, its failure mode is brittle (Figure 1a refers to a box girder; in the case of plate girders the falling branch of the M-χ diagram may be steeper due to possible lateral torsional buckling). If, on the other hand, the tension flange yields before the compression flange reaches its ultimate strength, the corresponding cross-section shows a ductile behaviour which is typically the case for composite cross-sections in sagging bending regions (Figure 1b). However, if in the hogging bending region the neutral axis is near to the compression flange, important gains in section ductility are possible [2, 5, 7]. Consequently, the criteria for a ductile behaviour of a cross-section is that the tension flange reaches its maximum strain before the compression flange fails.

If a ductile behaviour of both, the cross-sections in the span and at the support can be guaranteed, then there is no need to carry out a non-linear global analysis (2.1.2): applied moments can be determined by using elastic global analysis which is the common practice in bridge design. Resistance moments may be deduced from the moment-curvature diagrams, which are needed to ensure that the ductility criteria is satisfied and which are calculated according to the strain oriented approach from 2.1.1. In a ductile bridge structure moment redistributions are possible. Therefore, their sensitivity to the uncertainties of the action effects is considerably reduced. If in the example from chapter 1 the cross-section at the support would show a ductile behaviour, settlements of even more than δ=0.3 m (2.1.2) would not reduce the structural safety below the required level according to [3, 4].

2.2 Validity of the method

The proposed strain oriented method, based on the separate modelling of the different elements composing a cross-section only is valid if possible negative reciprocal influences between the elements due to instability effects can be excluded. Furthermore, it is to be ensured that the different elements behave as is implicitly supposed in the design models. The assumed structural behaviour can be guaranteed if a set of geometrical minimum requirements are reached [2, 4, 5].

3. Unstiffened and stiffened steel plates under compression

3.1 Introduction

The behaviour of composite bridge cross-sections depends very strongly on the response of their steel plated elements under compression. Due to different possible instability effects, the analysis of these elements is complex and requires special attention. Most design codes only give rules for the calculation of their ultimate load. The presented strain oriented approach requires, however, an easy-to-use method which allows for an accurate estimation of the load shortening behaviour of steel plated elements. The cases of unstiffened and stiffened plates are considered separately.

3.2 Unstiffened steel plates

3.2.1 Strain oriented formulation of the effective width approach
Due to a membrane effect unstiffened steel plates under compression possess a post-critical resistance which can be taken into account according to the effective width concept [8]. Appropriate buckling curves (e.g. Winter [8]) are introduced for the modelling of geometrical imperfections and residual stresses. The effective width, and consequently the resistance, of such elements is strain dependent: the bowing effect (out-of-plane deflection) increases with increasing strains, which means that the effective width and the resistance decrease after having reached a

maximum value. The strain dependence of the resistance can be taken into account by reformulating the effective width approach for steel compression plates [5]:

$$b_e = \rho \cdot b = \left(\frac{\overline{\lambda}_p - 0.22}{\overline{\lambda}_p^{\,2}} \right) b \tag{1}$$

b_e	effective width
b	total width of the plate
ρ	reduction factor for plate buckling

In the well known expression for the reference slenderness, $\overline{\lambda}_p$, stresses are substituted by strains:

$$\overline{\lambda}_p = \sqrt{\frac{\varepsilon}{\varepsilon_{cr}}} \tag{2}$$

ε	applied strain
ε_{cr}	critical buckling strain of the plate according to linear buckling theory [8]

3.2.2 Results

In the present section, the load shortening curves of unstiffened steel plates which are established by using the proposed strain oriented effective width approach are compared to available results [8] from tests and detailed numerical simulations. Major parameters affecting the behaviour of such elements are discussed. These parameters are: residual stresses, aspect ratio and initial out-of-plane deflections. The obtained results show that the proposed method is applicable for design purposes [5].

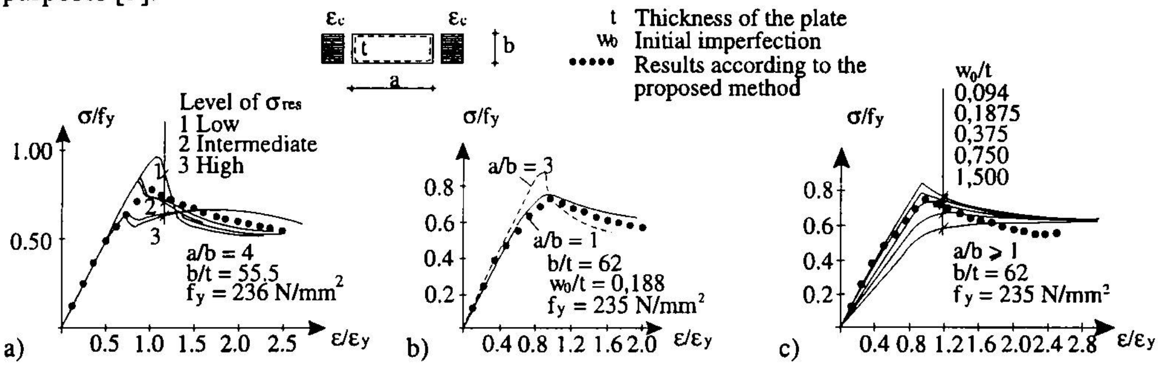

Fig.3 Strain oriented effective width approach - comparison with test results: influence of a) residual stresses, σ_{res}, b) aspect ratio, a/b, c) initial imperfections, w_0.

Moxham [8] tested simply supported compression plates with an aspect ratio of a/b=4, different levels of residual stresses, σ_{res}, and without nominal initial deflections, w_0=0. Even though the proposed method takes simultaneously into account the influence of residual stresses and initial deflections, a direct comparison with the test results is possible: for a plate slenderness of b/t=55.5, as used in the tests, the influence of the residual stresses tends to mask the one of the geometrical imperfections [8]. The proposed method leads to results which almost coincide with the obtained test results for an intermediate residual stress level (Figure 3a).

Frieze [8] showed that minimum plate strength is obtained for plates with an aspect ratio of approximately a/b=1. Longer plates exhibit higher maximum values of the strength, higher prepeak stiffness and less ductility (Figure 3b). The results calculated by using the proposed method are very close to the ones obtained by Frieze for square plates.

Figure 3c) shows the influence of the variation of the initial deflections, w_0, on the behaviour of a plate without residual stresses, σ_{res}=0 [8]. Again, a direct comparison is possible although in the proposed method residual stresses and initial deflections are simultaneously taken into account: for a plate slenderness of b/t=62, the influence of initial deflections dominates over the residual stresses [8]. The results according to the proposed method correspond to an intermediate level of initial deflections, which is in the range of the fabrication tolerances in current codes [4].

3.3 Stiffened steel plates

3.3.1 Strain oriented strut approach

In orthogonally stiffened compression plates different buckling modes are possible: overall buckling, nodal buckling of the plate panels between longitudinal stiffeners, tripping of stiffeners with open cross-sections and all possible combination modes. Practical geometries of compression flanges in composite box girder bridges usually lead to column type failure [8]. This failure mode can be guaranteed by respecting so called minimum requirements (2.2). The load shortening behaviour can therefore be obtained by using a model based on the strut approach (Figure 4). The proposed model treats the stiffened plate as a serie of disconnected struts consisting of a stiffener and an associated plate width [4, 5]. The ultimate load of a stiffened plate, N_{ult}, is the sum of the ultimate load of these struts and the ultimate load carried by the panels at the supports of the longitudinal edges (Figure 4):

$$N_{ult} = \left[n \cdot \chi\left(\rho \cdot b_L \cdot t + A_L\right) + \rho \cdot b_L \cdot t\right] f_y \qquad (3)$$

n	number of longitudinal stiffeners
b_L	width of the panel between two longitudinal stiffeners
t	thickness of the plate
A_L	area of the cross-section of a longitudinal stiffener
f_y	yield strength
ρ	reduction factor for plate buckling of the subpanels
χ	reduction factor for buckling of the struts

The interaction between the buckling of the struts and nodal buckling is taken into account by an iterative calculation of ρ (according to (1)) and χ (according to european column buckling curve c [4]). For this purpose, the applied strain, ε, in the reference slenderness of the subpanels for the calculation of ρ according to (2) is substituted by $(\chi \cdot \varepsilon_y)$, where ε_y is the yield strain of the material [4, 5].

In order to obtaining the load shortening curve, it is assumed that the ultimate load, N_{ult}, is reached for an intermediate strain, ε_{ult}, between the ultimate strains of an unstiffened plate and the one of a steel column, respectively [5]. For the prepeak behaviour, a linear load shortening relation is assumed until first buckling occurs, usually nodal buckling between longitudinal stiffeners (Figure 4). The postpeak behaviour is described by using well known relations between load and deflections of ideal rigid plastic columns.

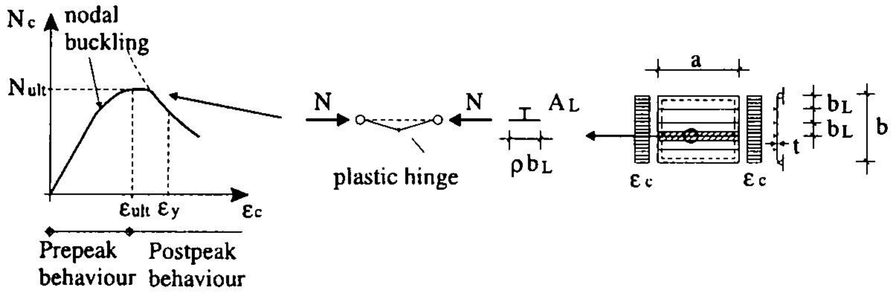

Fig.4 Stiffened compression plates - strain oriented approach.

3.3.2 Results

Ghavami tested longitudinally stiffened steel compression plates with and without transversal stiffeners up to collapse [9]. In [5], the results of 12 of these tests are compared to the results obtained by using the proposed model for the ultimate load, N_{ult}. For comparison, the ratio between the ultimate load according to the model and the collapse load from the corresponding test is established for each of the 12 results. For the analysed sample, a mean value for this ratio of m=0.97 is found, and a standard deviation of s=0.055. These results are very satisfactory: the model produces slightly conservative results (m<1), and the found scatter is very small (coefficient of variation cov=0.056) compared to usual scatters produced by other design approaches for stiffened compression plates [9].

4. Conclusions

The rules given in current design codes for composite bridge structures are simple to use but do not include all relevant parameters for the representation of section ductility. Therefore, the beam load capacity can not be accurately predicted, and in-built brittle behaviour can not be excluded in the design. In order to avoid these drawbacks, ductility or rotation-capacity evaluations for cross-sections are necessary, which is possible if the traditionally stress oriented design philosophy is substituted by a strain oriented reasoning. Benefits of a ductile structural behaviour include that the sensitivity of a structure to the uncertainties of action effects is reduced.

The proposed strain oriented design method is suitable for a uniform treatment of all cross-sections and allows for the evaluation of moment-curvature diagrams in a way which is applicable for design purposes. Ductile structural behaviour can therefore be guaranteed.

Acknowledgements

Some of the presented work has been carried out as a part of the studies for the Spanish Recommendations for the design of steel and composite bridges, promoted by the "Dirección General de Carreteras" of the Ministry of Public Works.

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