CETRA[2016]

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23–25 May 2016, Šibenik, Croatia

# Road and Rail Infrastructure IV

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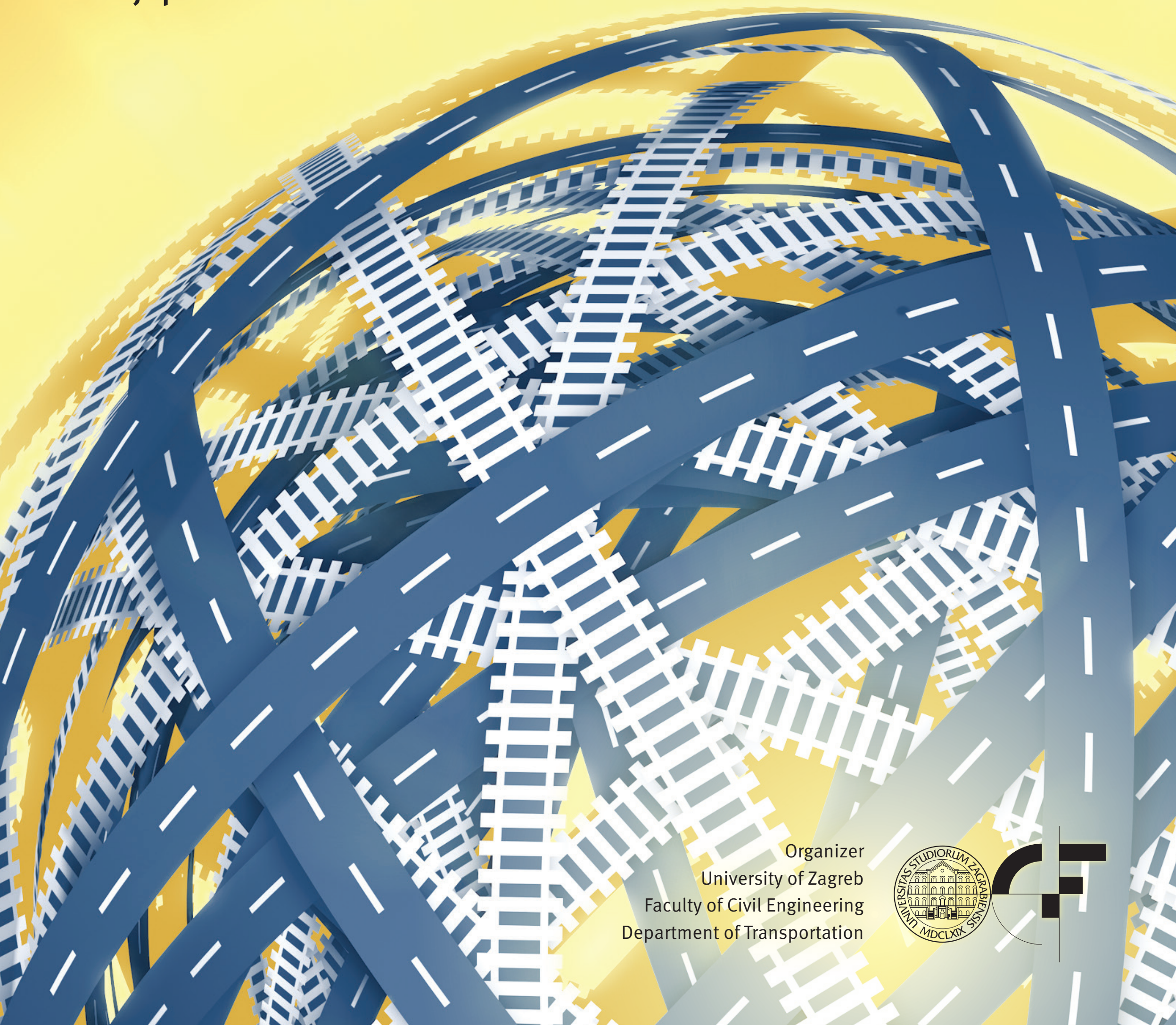

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# Road and Rail Infrastructure IV

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# UNDERSTANDING AND PREDICTING GLOBAL BUCKLING DURING CONSTRUCTION OF STEEL BRIDGES

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## Abstract

Collapse of steel bridges during construction can occur as a result of a global buckling behaviour which may be overlooked when using member resistance checks such as those in the Eurocodes. Furthermore, buckling can require careful study when working with existing structures that were not constructed to modern tolerances and which consequently cannot be safely assessed using modern design codes. It is an issue of design, construction and sustainability. This paper describes how finite element analysis can be used to predict buckling modes. It draws on recommendations in the recently published NCHRP Report 725 [1], exploring the problem of global buckling modes and considering how to identify when these should be of concern to the designer. Use of FE analysis in the determination of member resistances is also discussed, with reference to current design standards and to alternative approaches which may be appropriate for historical structures, drawing out the key considerations and necessary checks when undertaking such analyses.

*Keywords: steel bridges, buckling, construction, finite element analysis*

## 1 Introduction

Buckling analyses performed using Finite Elements (FE) may be elastic or nonlinear. Elastic buckling analyses give results which may be used in member resistance calculations in codes of practice, and – crucially – can be used to identify 'global' buckling modes not routinely identified when carrying out such checks (see Figure 1 below). Nonlinear buckling analyses may also be useful in certain cases, such as when considering existing structures which have details and tolerances that fall outside modern standards. This paper explores the practical applications for both of these buckling analysis options.

## 2 Elastic buckling analyses using FE

In FE analyses, the real or potentially real object is idealised as a series of 'elements', connected at nodes. For analysis of bridge structures in 3D, the most commonly used elements are beam elements – suitable when a member is long in comparison to its cross-sectional dimensions; and shell elements – suitable when a member has plan dimensions which are large in comparison to its thickness. FE models may use mixed elements and can be used to analyse members or whole structures, considering non-standard details, support conditions and load arrangements as necessary.
In the FE solution, a stiffness matrix is constructed, based upon the member dimensions, material properties and support conditions. When combined with the loading, a linear static analysis can be performed. Alternatively, an elastic buckling analysis can be carried out, determining the eigenvalues of the stress-stiffness matrix and the corresponding eigenvectors.

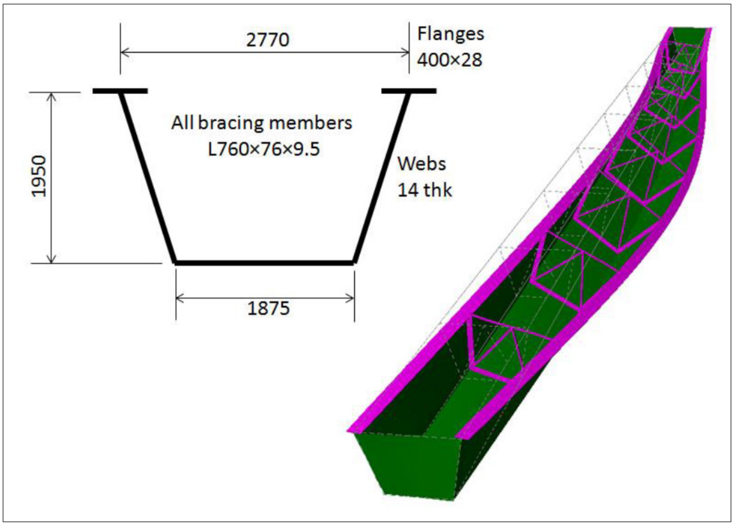


**Figure 1** Global buckling mode causing collapse of Marcy pedestrian bridge

The eigenvalues obtained from an elastic buckling analysis each give the factor, $\alpha_{cr}$, by which the applied loading would have to be increased to cause elastic instability in the corresponding mode (determined from the relevant eigenvector).

Due to material plasticity, initial imperfections (including out-of straightness and residual stresses) and second-order effects (large displacement theory), the experimental buckling resistance of a member is generally less than might be inferred from an eigenvalue 'load factor', although post-buckling behaviour allows certain classes of member to achieve a higher resistance (see Galambos [2]).

Nonetheless, elastic buckling – and therefore eigenvalues – are of immense use to practicing engineers, as described in the following sections.

## 3 Global buckling phenomenon

The Marcy pedestrian bridge in Figure 1 collapsed during construction in New York State, 2002, when the concrete deck pour was about 60% complete (Yura & Widianto, [3]). It was a straight, single span trapezoidal box of 52m, with a design complying with the appropriate member resistance clauses in the US standards of the time. The Marcy collapse was caused by a 'global' buckling mode which may equally arise in other girder systems. Member resistance checks considering buckling between bracing locations may indicate adequacy, but the braced system can buckle in a lower mode over an effective span-length.

### 3.1 Identifying susceptible structures

A 'global' mode such as the one which caused the Marcy collapse can be identified with an elastic buckling analysis and the corresponding load factor, $\alpha_{cr}$, allows an assessment of whether the mode could occur under the design loads.

For the reasons already described, $\alpha_{cr}$ cannot be thought of as a factor of safety against buckling. Instead it is more helpful to use it in the calculation of an 'amplification factor', $AF_G$:

$$AF_G = \frac{1}{1 - \frac{1}{\alpha_{cr}}} \tag{1}$$

Expression 1 above is based on NCHRP Report 725 [1] (Eqn 2) and conceptually might be used to factor up responses obtained from a linear static analysis in lieu of a second-order (geometrically nonlinear) analysis. Hence, the report indicates, where $AF_G < 1.1$, the influence of second-order effects may be neglected whereas where $AF_G > 1.25$ then the adequacy of the structure must be justified and a comprehensive nonlinear analysis is recommended.
Calculation of $AF_G$ is recommended because the value gives engineers a sense of the inaccuracy associated with the results obtained from their linear static analysis. In fact EN1993-1-1 [4] clause 5.2.1(3) also deems second-order effects negligible when $\alpha_{cr} \geq 10$ (corresponding to $AF_G > 1.1$), but experience suggests that engineers find it difficult to regard a factor of 10 as entirely necessary.
In either case, a global buckling mode and corresponding value for $\alpha_{cr}$ needs to be determined. Yura et al [5] propose some simplified expressions which can be used for this purpose, based upon a pair of identical, prismatic, doubly-symmetric I-section girders with effective bracing, subject to a uniform moment. They offer adjustments for singly symmetric sections, consideration of up to 4 girders and moment gradients but even so, the limitations are such that for most practical cases, an eigenvalue buckling analysis using FE can be used to give a greater level of accuracy and confidence.

### 3.2 Example – Marcy Pedestrian bridge

For the purposes of this paper, the Marcy Pedestrian bridge was retrospectively modelled using LUSAS (Figure 1). The webs, bottom flange and diaphragms were represented using quadratic order thick shell elements (QTS8), while the compression flanges and bracing were represented using quadratic order thick isoparametric beam elements (BMI31) ref. LUSAS [6]. Such a model is capable of identifying a variety of buckling modes including local modes – and the critical global mode – and is rapid both in generation and solution, therefore an efficient model for this instance. Values for $AF_G$ in Table 1 indicate significant second-order effects under merely the self-weight of the girder, considerably increased when the deck pour reached 60% completion – to the extent that the collapse would have been readily predicted by engineers using such data.

**Table 1** Key results from LUSAS analysis of Marcy pedestrian bridge

| | Units | Self-weight of girder only | Added load of wet concrete |
|---|---|---|---|
| Eigenvalue load factor, $\alpha_{cr}$ | none | 3.32 | 1.07 |
| Amplification factor, $AF_G$ | none | 1.43 | 15.28 |
| Peak elastic stress in compression flange from linear static analysis, $\sigma_{x,Ed}$ | MPa | 50.7 | 136.9 |
| Elastic critical stress, $\sigma_{cr} = \alpha_{cr} \times \sigma_{x,Ed}$ | MPa | 168.5 | 146.5 |
| Amplified stress, $\sigma_{AF} = A_{FG} \times \sigma_{x,Ed}$ | MPa | 72.5 | 2091.8 |
| Real world outcome | | No collapse | Collapse |

## 4 Member resistance calculations

The design resistances in the Eurocode, in common with other international codes, are based upon slenderness which is in principal defined from the critical elastic buckling load or moment – see $N_{cr}$ in EN1993-1-1 [4] clauses 6.3.1.2(1) and 6.3.1.4(2) and $M_{cr}$ in clause 6.3.2.2(1) respectively.
Values for $N_{cr}$ and $M_{cr}$ may be determined by any suitable means. Where members are of prismatic cross-section and within the standard section shapes, end restraints and loading conditions and moment distribution, the values are typically determined using closed form

expressions such as those available in SN001a [7] and SN003b [8]. For sections which fall outside such criteria, for example tapering sections – or indeed for any section – $N_{cr}$ and $M_{cr}$ may be determined using an eigenvalue extraction from an FE analysis, i.e.

$$N_{cr} = \alpha_{cr} N_{Ed} \text{ or } M_{cr} = \alpha_{cr} M_{Ed} \text{ as appropriate} \tag{2}$$

Importantly, buckling modes identified using FE may be visualised, potentially resulting in a better understanding of structural behaviour than when formulae are used 'blindly'.
These critical elastic values ($N_{cr}$ and $M_{cr}$) are not themselves used as design resistance values because they do not take into account material plasticity, initial imperfections or second-order effects. The expressions for member resistance in the Eurocode make the necessary allowances leading to a safe design.
The ability to use eigenvalue analysis to determine slenderness for member resistance calculations makes it possible to use the codified expressions to determine resistances for quite non-standard structures, such as existing structures.

### 4.1 Example – existing U-frame bridge

The assessment of existing railway structures in the UK is to NR-GN-CIV-025 [9], which requires member resistances to be calculated using BS5400-3 [10]. The bending resistance of the main girders in the U-frame bridge of Figure 2 below was assessed using the manual method of clause 9.6.4.1.3, and found to be inadequate for the Client requirements.

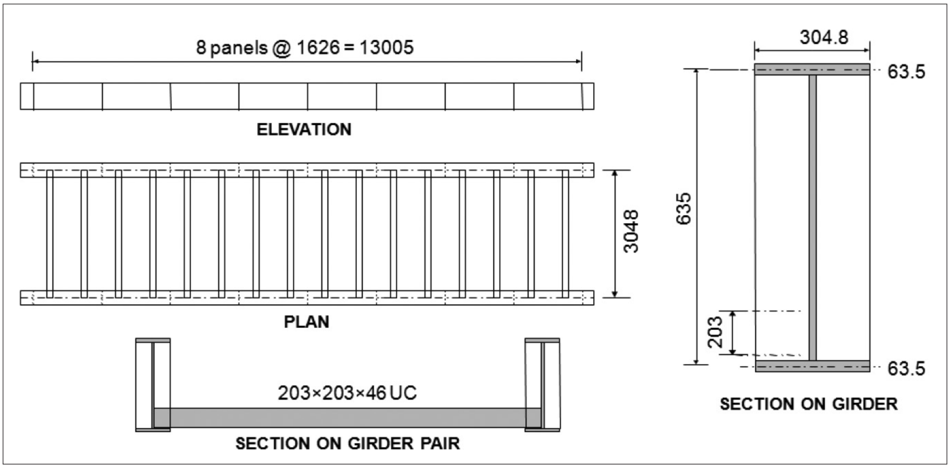


**Figure 2** Example U-frame bridge deck

The member resistance for the main girders was then re-calculated, using an eigenvalue analysis of an FE model as illustrated in Figure 3. Using the load factor obtained to derive a slenderness in accordance with BS5400-3 [10] clause 9.7.5 (similar to the Eurocode approach); the calculated resistance was improved by 41% over the manual method of clause 9.6.4.1.3. In the FE model of Figure 3, shell elements are used to represent the main girders (webs, flanges and stiffeners) and the cross-members (webs and flanges). Using shell elements throughout ensures that buckling modes, global and local, are identified and can indeed contribute, together, to the predicted failure of the member in a full nonlinear analysis, if this is later required. Thus the small additional overhead in modelling and solution time associated with using shells rather than beams to represent flanges, stiffeners and cross-members often pays back.

In this example, the manual calculations to [10] clause 9.6.4.1.3 had revealed that the connection between cross-members and main girders was effectively rigid by comparison to other parts of the U-frame. In other cases, that approach would result in an over-estimate of buckling loads, and so the flexibility should then be included in the model, by inclusion of suitable joint elements.

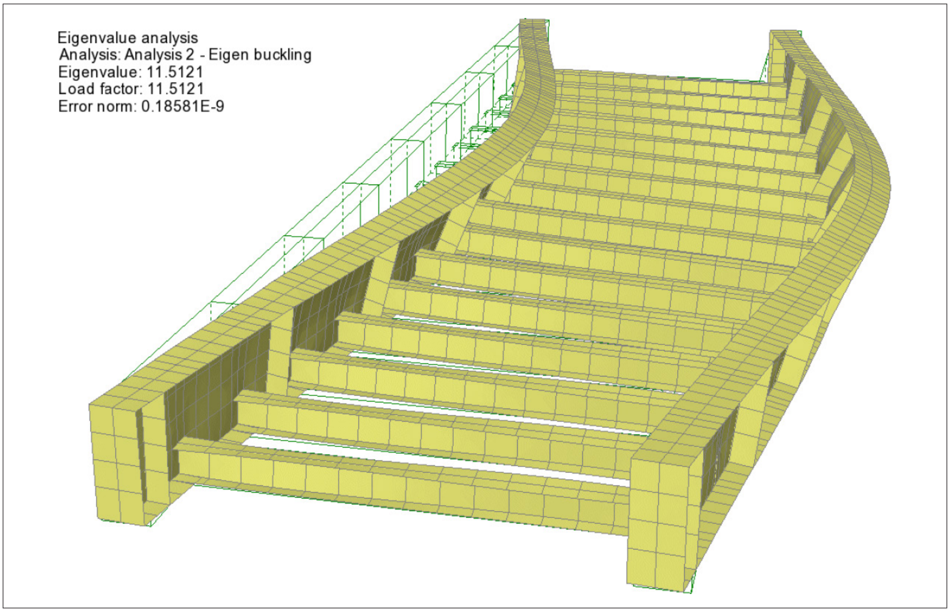


**Figure 3** Lowest buckling mode from eigenvalue analysis of U-frame deck

## 5 Nonlinear buckling analyses using FE

Moving beyond Eigenvalue analyses and codified approaches, a nonlinear analysis can provide an alternative means for assessing failure loads. This may be appropriate when unexpected behaviour has been highlighted by a prior Eigenvalue buckling analysis, when the structure or details are outside the scope of the code, or when the importance of the structure warrants further investigation, for example, a heavily trafficked existing structure where remedial works would be very costly and disruptive.

Models such as those in Figure 3 allow both local and global buckling modes to arise and therefore are a suitable starting point for a full nonlinear analysis. Crucially, such an analysis must take into account material plasticity, initial imperfections and second-order effects. It may also incorporate lift-off at bearings in skew structures, slack connections and other structure-specific issues such as corrosion, as necessary.

The results from a full nonlinear analysis for the example U-frame deck of Figure 2 gave a buckled shape as shown in Figure 4, and indicated a failure load 70% greater than that determined using the manual method of calculation of BS5400-3 [10] clause 9.6.4.1.3. Similar improvement in capacity was found by Hendy et al [11].

Material plasticity is readily added to FE models when using software with the appropriate facilities such as LUSAS. Steel materials have significant strain hardening beyond yield, and ideally this should be incorporated in order to give represent the behaviour of the structure well. However, post-yielding behaviour is "sensitive to gauge length type effects" (CIRIA C664 [12] section 7.7.5) and so a conservative strain hardening slope should be adopted.

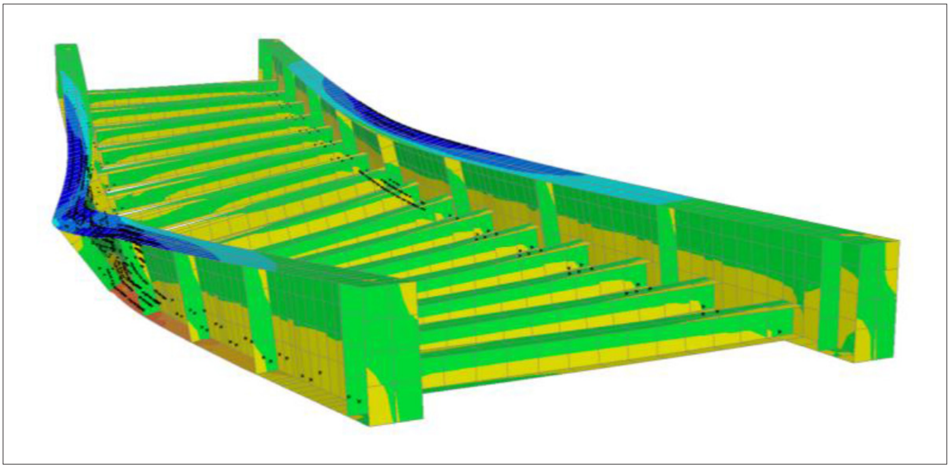

**Figure 4** Buckling with yielding, from nonlinear analysis of U-frame deck

Regarding initial imperfections, these are also readily incorporated and can have a significant effect on the analysis results – this underlines the limitation of codified buckling rules to members fabricated and erected to modern tolerances, and the possible need for nonlinear analysis to be used for members not meeting such standards. EN1993-1-1 [4] suggests using the shape of the elastic critical buckling mode as an imperfection when second-order analysis is used (see clause 5.3.4) with the amplitude based on the section in question (see Table 6.2 and Table 5.1 in conjunction). Broadly speaking, the imperfections are of order span/150, or span/300 for heavy bridge sections if LTB is concerned. These values – significantly greater than expected fabrication tolerances – incorporate an allowance in lieu of residual stresses, locked-in during fabrication. For existing structures NR-GN-CIV-025 [9] clause 9.12.1A requires initial imperfections corresponding to 1.2 times the measured out-of-straightness of the compression flange – the 1.2 factor allows for residual stresses. If the bow in the flange is not measured, but instead the construction and fabrication tolerances from Codes or drawings are used, the imperfection should be based on a larger factor, perhaps twice the stated tolerance (CIRIA C664 [12] section 7.7.3).
Use of an incremental-iterative approach with geometric nonlinearity should ensure that second order effects are fully incorporated (LUSAS [13], Chapter 3).
Partial factors can be conveniently applied to the results from nonlinear analyses (rather than within nonlinear material properties, for example). This is over-conservative for buckling analyses, being equivalent to applying the partial factors to the elastic modulus of the material as well as the strength. However, it is noted by CIRIA C664 [12] to be "rigorously safe and simple".

## 6 Conclusions

The elements used in FE buckling analyses might comprise shells, beams or a mixture. In general, shell elements are recommended. In all cases, mesh refinement should be checked. Elastic critical buckling loads may be obtained from eigenvalue buckling analyses and used for member resistance calculations using codes of practice. This has particular application for non-standard members such as existing structures.
Nonlinear analysis may also be used to assess member resistances. Initial imperfections need to be included in such analyses and eigenvalue buckling mode shapes typically provide a suitable imperfect shape.

Eigenvalue buckling analyses can also be used to investigate the susceptibility of any girder system to second-order effects or stability issues.
Nonlinear analysis is recommended by NCHRP Report 725 [1] and EN1993-1-1 [4] clause 5.2.1(3) for girder systems with a large amplification factor ($AF_G$), or for which lift-off may occur. Such analyses can be readily undertaken, utilising the same analytical models constructed for a prior eigenvalue buckling analyses.

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