Rating and Upgrading of Steel Bridges using Finite Element Modeling

T. Cakebread.
LUSAS, New York, USA

S. Rhodes
LUSAS, Kingston upon Thames, UK

ABSTRACT: This paper describes how Finite Element (FE) modeling techniques can assist in the assessment (rating) and upgrading of steel bridges of various types. Global and local modeling options are considered with reference to several projects, in particular the West Gate Bridge in Melbourne, Australia. The use of different analysis assumptions, element types, eigenvalue and nonlinear analysis functions to achieve greater load rating capacity is outlined, identifying examples of good practice and drawing on international codes and literature for recommendations.

1 INTRODUCTION

In assessment, rating, rehabilitation or retrofit of bridges, it is often necessary to consider issues outside the scope of design codes, or analysis approaches which are not routinely used in design of new structures. Assessments progress in stages of increasing sophistication (AASHTO/NSBA 2011), (COST 345:2004) since bridges which are deemed substandard by use of simplified approaches can sometimes be shown to be adequate when more advanced approaches are employed.

Since the cost and disruption caused by interventions can be very significant, approaches which would not perhaps be regarded as cost-effective for new designs can be justified.

This paper considers several aspects of FE modeling in the context of the assessment and upgrading of steel bridges, giving relevant project references and recommendations.

2 GLOBAL MODELING APPROACHES

Steel bridges come in a wide range of structural arrangements such as trusses, half-through girders, steel girders composite with Reinforced Concrete (RC) decks and so on. The method of analysis used must be appropriate to the structural behavior of the bridge in question, and more highly developed models - better representing the likely real behavior of the structure - become appropriate as a multi-stage assessment reaches its final phases.

Assessments almost universally commence with a linear elastic analysis, used to evaluate the load-carrying capacity of the structure at Ultimate Limit State (ULS). Although hand calculations are used, often FE software is involved early on, when it is typically not known if nonlinear analysis will be required later. However, software which has nonlinear capabilities should be selected at the outset, enabling analysis models to be modified and developed, rather than re-created from scratch, as the assessment progresses from stage to stage. Switching software late in the process often leads to conflicting results from models which are only somewhat similar in their set-up, and these are time-consuming to resolve since they might arise from a range of sources.

Even without recourse to nonlinear analysis techniques, the idealization adopted can significantly affect the peak load effects identified in the structure. Therefore modeling assumptions should be challenged early in the assessment process and the analysis approach modified to give more realistic results. The following subsections consider some analysis approaches which have been adopted for specific projects.

Assumptions concerning boundary conditions can have a profound influence on calculated internal force and on member resistances. The assumption that translations and rotations are either rigidity restrained or free at support locations is as crude as it is commonplace. While the attention of the bridge engineer is quite reasonably drawn to the deck structure, and issues described in the following sections are principally concerned with deck analysis, it is important to devote as much care to the representation of the boundary conditions as to the deck structure (HAMBLEY:1991), (Guideline for Load and Resistance Assessment of Existing European Railway Bridges:2007). Substructures which transfer loads by direct compression can be quite stiff under vertical loads. However, piers with bending elements such as cross heads can have significant flexibility. Foundations and substructures may be included explicitly in a finite element analysis - alternatively simpler formulations may be adopted, such as springs, perhaps calibrated using a more comprehensive analysis (the considerations detailed in section 4 below apply). Even the inclusion of some representation for elastomeric bearings, substructure and subgrade using a spring stiffness can be sufficient to alleviate perceived problems with lift-off in many instances. For bridges where lateral soil pressures are a consideration, such as integral bridges, clearly, the treatment of the soil-structure interaction is fundamental to the structural behavior and special attention must be afforded.

2.1 *Truss Bridges*

Truss bridges would generally be analysed using global models made up of bar or truss elements. However it is important to note that while truss members might be assumed pin-ended for ULS assessment, when considering serviceability and fatigue criterion, the likely real moment fixity at connections may require consideration (Guideline for Load and Resistance Assessment of Existing European Railway Bridges:2007). This makes the use of beam (rather than bar) elements preferable. Gusset plates may require local modeling, as discussed in section 4 below.

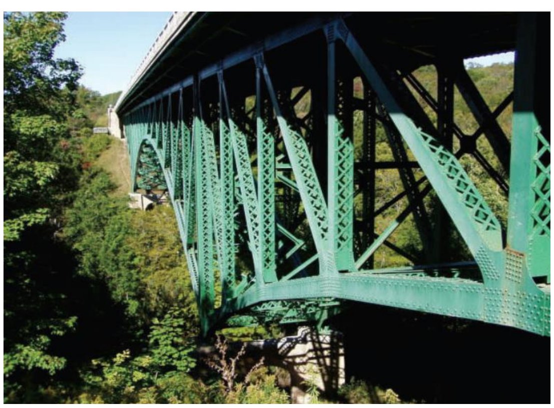

Figure 1. Cut River Bridge.

Considering trusses to be generally of low redundancy, the stress distribution given by simplified (2D beam-type) models may be considered difficult to improve upon. However, somewhat more elaborate models have been shown to be cost-effective in assessment work, for example, including shell elements to represent the deck and joint elements to allow for partial fixity at connections. Such an approach was used in the load rating of the 183m span Cut River Bridge (Fig. 1) by Benesch for Michigan Department of Transportation. In this bridge, the truss supports a concrete deck with non-composite action via stringers and floor beams. A 3D FE analysis was found to distribute loads more evenly than might have been assumed when using a simplified method - with concentrated vehicle loads being dispersed longitudinally as well as appropriately divided between the two trusses. The reduced live load forces in truss members eliminated the significant cost and disruption which would have arisen from a strengthening project.

2.2 *Half-through Girder Bridges*

In half-through girder bridges, cross-girders connect to the main girders near the bottom flange (i.e. remote from the centroid), and this arrangement has significant effect on the behavior of the structure, particularly as regards stability (buckling).

The resistance of girders connected in this “U-frame” arrangement is covered by several codes of practice (EN 1993-2:2006), (BD 56/10: 2010), (BS5400-3:2000), (CAN/CSA S6-06: 2006) However, many existing bridges have details which fall outside the scope of the codes, and in any case, generalized rules can be significantly over-conservative for certain cases. One study (MEHRKAR-ASL: 2005) showed that using analyses which included the restraint associated with the in-plane shear stiffness of the bridge deck (not incorporated in codified rules) led to significant improvement in load rating values.

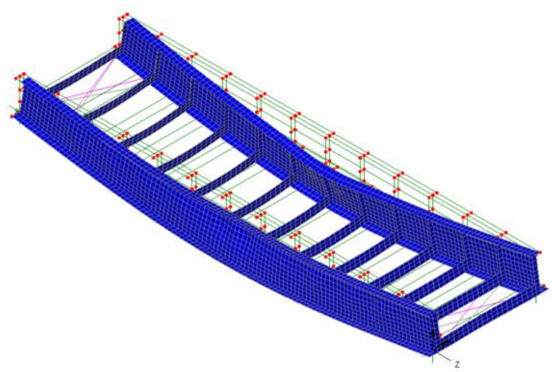

Figure 2. Half-through girder modeling.

Use of beam elements to represent the main girders in a half-through bridge is not generally appropriate, as the connection - remote from the centroid - is not easily represented well in such a model. Shell element models (Fig. 2) can better represent the structural arrangement. The cross members might be represented using either shell elements or beam elements and joint elements can be used at the connections. Guidance on the flexibility in the connection may be found in codes (EN1993-1-8: 2004) although the information may not be sufficient (Guideline for Load and Resistance Assessment of Existing European Railway Bridges: 2007).

In the assessment of the edge-girder span of the railway over-bridge at London Road, Hackbridge (Fig. 3), Robert West Consulting and their Category III checkers used shell and beam models of this sort to demonstrate the adequacy of the structure for 40 tonnes assessment live loading when earlier assessments had indicated that a weight restriction would be required.
For buckling behavior to be well represented in a nonlinear analysis, it will be necessary to use shell elements and to apply initial imperfections. See 5.1 below.

Figure 3. Over-bridge at Hackbridge, UK.

2.3 *Beam and slab bridges*

A variety of spaced beam and slab construction formats are used in bridge engineering, but for the purposes of selecting an analysis approach, they can be generally rationalized to three groups:

1. Girders having two or more webs (i.e. box beams). Generally grillage models or 3D shell element models are used. Such structures are considered in section 2.4 below.
2. Girders having a single web and where cross-bracing is not structurally significant in-service. Such structures are typically analyzed using grillage models or shell-on-beam models with an offset (sometimes termed "plate-eccentric-beam" PEB models) (Fig. 4)
3. Girders having a single web with cross-bracing which is structurally significant in-service. A 3D mixed element model may be appropriate (Fig. 5).

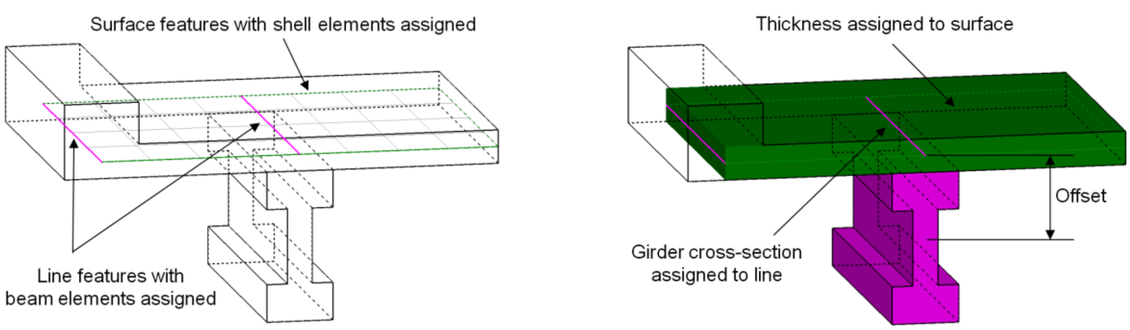

Figure 4. Shell-on beam model, showing offset for main girder (where in-plane loads are not applicable).

Certainly, for curved or heavily skewed girder structures, bracing can have a significant effect on load distribution from the construction stages forward. Traditional grillage or PEB models cannot readily incorporate bracing in a rigorous way. Furthermore, warping stresses may be significant in the girders of such structures.

As described in clause 4.6.3.3.2 of AASHTO (AASHTO LRFD: 2012): "Frequently, the torsional warping degree of freedom is not available in beam elements. The finite element method may be applied to a three-dimensional model of the superstructure. A variety of elements may be used in this type of model. The three-dimensional model may be made capable of recognizing warping torsion by modeling each girder cross-section with a series of elements."

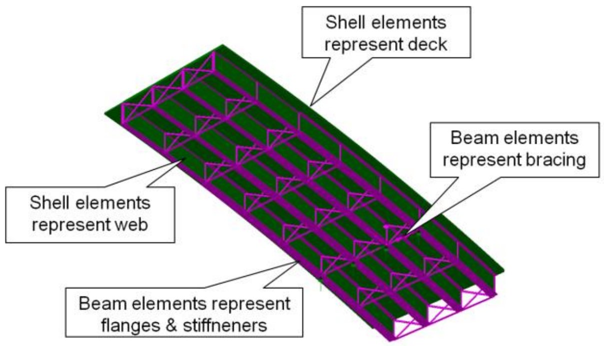

Figure 5. Mixed beam and shell element modeling.

The model illustrated in Fig. 5 answers to this description. In this illustration, shell elements are used to represent the slab and the girder webs, with the top and bottom flanges represented using beam elements. Stiffeners and cross-bracing are also incorporated using beam elements.

An alternative model might use shell elements to represent the flanges also, however, for many structures the inaccuracy arising from assuming that plane sections remain plane for each flange would be acceptable.

2.4 *Box girder bridges*

For box girder bridges, global analysis approaches include the use of simple beam models, grillages, or models largely made up of shell elements.
On the West Gate Bridge Upgrade project (West Gate Bridge Upgrade: 2012) the designers, Flint and Neill, used a global model primarily constructed from shell elements, with beam elements used for some stiffening, bracing and some other members (Fig. 6). The project required the assessment and analysis of the existing 5-span cable-supported structure, with proposed strengthening in place, considering an increased number of traffic lanes.

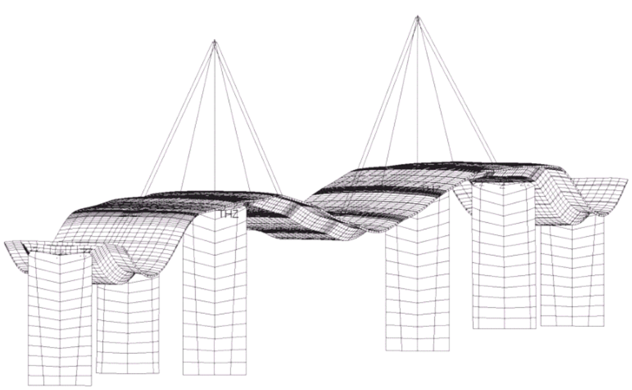

Figure 6. Shell and beam model of West Gate Bridge, showing exaggerated permanent load deflection and cable creep.

The deck of the West Gate Bridge is a continuous steel box girder, heavily stiffened, divided into three cells longitudinally, and having plated diaphragms every 16m. A grillage analysis of such a deck would notionally cut the deck longitudinally between webs into large I-beams. This would place longitudinal members more than 6m apart, considerably wider than traffic lanes, making it necessary to consider transverse behavior with separate models. Taking into account torsional and distortional behavior in such stiffened boxes is difficult, with inclined webs presenting a further challenge (HAMBLEY: 1991). Added to these issues, in-plane effects would be expected to be significant and these would create misleading local in-plane distortions of grillage members in a 3D analysis (O'BRIEN and KEOGH: 1999).

The shell element model used captured the global behavior of the deck, with all these issues handled by virtue of the robust mathematical formulation of the shell elements, rather than with assumption upon assumption. The approach offered other advantages, some of which are described in section 4.2 below.

3 CORROSION

The assessment of a steel bridge must make due allowance for section loss arising from corrosion (or other damage). Although guidance exists which helps to classify corrosion in terms of a severity number (Inspection Manual for Highway Structures: 2007), measurement of corroded sections on-site is often difficult, leading to uncertainty in the effective sections which should be used in subsequent assessment calculations.

Generally, section loss due to corrosion does not lead to a significant change in the load distribution in steel girder bridges (AASHTO/NSBA: 2011). This means that models based on uncorroded sections are often sufficient, although section loss can result in a high-stress zone which requires special consideration. In truss bridges, loss of section at a connection or in an isolated member can lead to disproportionate collapse owing to low redundancy and great care must be taken. Closed hollow sections which can corrode from the inside out present a particular concern.

On the resistance side, corrosion should be taken into account. However, initial calculations for resistance of steel members based on uncorroded sections are often instructive since where the uncorroded sections are found to be inadequate, discussions about the severity of the corrosion can be truncated. Instead, other assumptions must be challenged – other assumptions in the derivation of assessment load effects (i.e. in the analysis) - or in the member resistance calculation.

4 LOCALISED MODELING

For non-standard structural details or structures with significant discontinuities, the conventional P/A, My/I, or VAy/Ib relationships are not able to sufficiently describe the state of stress, and it is unsafe to use member resistances based on similar assumptions. In such cases a detailed stress analysis can be carried out using some form of localized FE model. Bearings, connections and cable anchorages are amongst the details most often requiring localized FE models, which may make use of a variety of element types including beam, shell and 3D continuum (volume) elements.

Is important to recognize that stress limits given in codes will not always be compatible with the results of such analyses; a first-principles approach to stress distribution needs to be reflected with a first-principles approach to material resistance. Furthermore, the considerations concerning boundary conditions, as described in section 2 above, apply equally, or more, to local models (AASHTO/NSBA: 2011).

4.1 *Standalone local models*

Traditionally, standalone local models have been adopted, requiring the correct application of appropriate boundary conditions derived from a prior global analysis. This approach is computationally inexpensive but relies on the engineer updating two models between which there is some interaction, increasing the risk of error.

4.2 *Embedded local models*

Alternatively local models can be embedded within a global model, obviating the need for boundary conditions to be derived from the global model and applied as described above.

The West Gate bridge project described in 2.4 above again provides a good example. The deck is cable-supported along its centerline with two sets of cables which pass over saddles at the tower heads and splay out at deck level into cable anchorages within the steel deck box. Detailed stress analysis of the cable anchorages was carried out with local models embedded in the global model (Fig. 7), as well as detailed consideration of stresses in the webs, diaphragms and various stiffened plates. The stress results obtained were considered in light of current standards and the Interim Design and Workmanship Rules (IDWR) produced by the Merrison Committee in February 1973, contemporary at the time of construction of the bridge (HAYWOOD, SADLER. and TORDOFF: 2002).

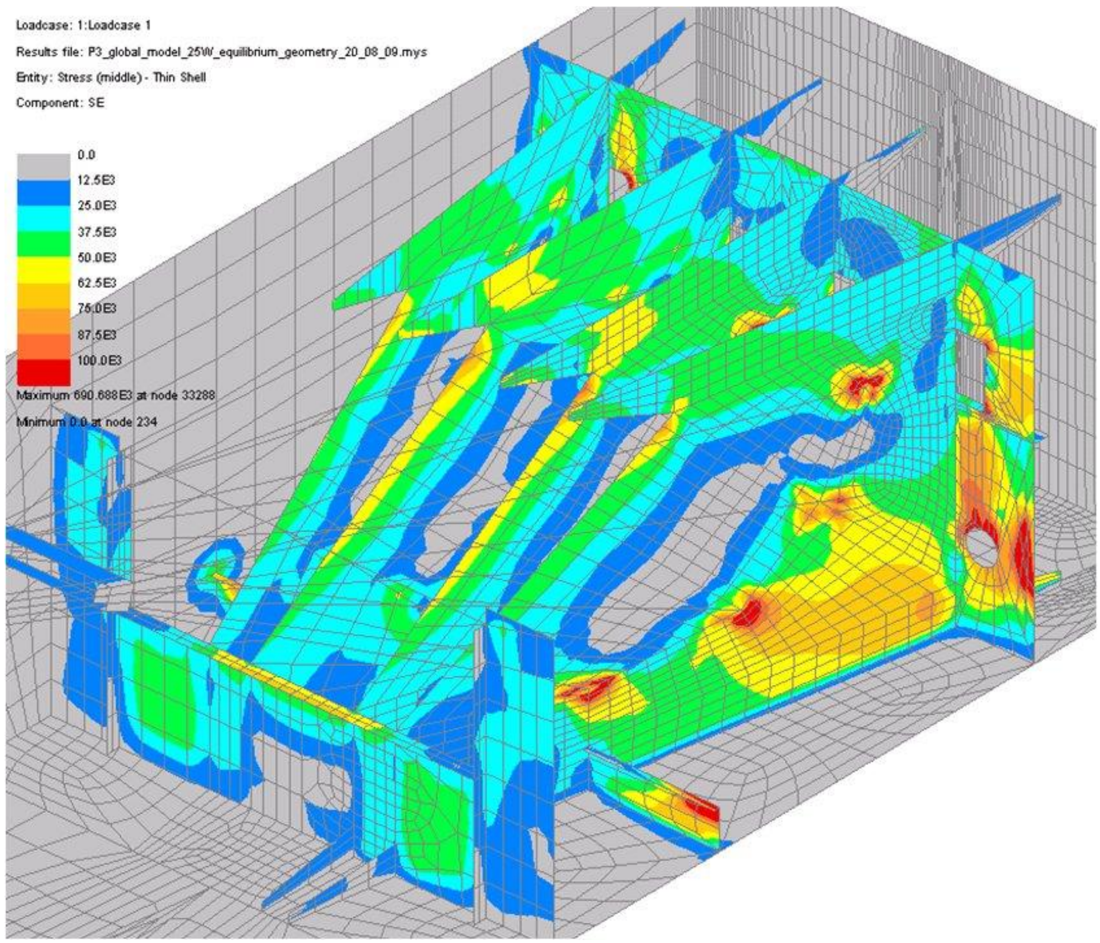

Figure 7. Stresses in cable anchorage plates.

A similar approach can be used to study a range of other details, such as selected gusset plates in a truss structure, perhaps to investigate the buckling susceptibility of the connections (see 5.1 below).

5 USE OF NONLINEAR ANALYSIS

While elastic analysis may be used for Serviceability Limit State (SLS) and fatigue assessments - and for initial assessments at ULS, nonlinear FE analysis is recommended for advanced assessment, since it is likely that allowing for plastic deformations will give a more favorable result (Guideline for Load and Resistance Assessment of Existing European Railway Bridges: 2007).

A nonlinear analysis should always be preceded by a linear analysis, which forms a baseline. It is also prudent to try out nonlinear software functions using test models of simple structures to ensure that a thorough understanding of the function is assimilated before it is applied to the structure in question (AASHTO/NSBA: 2011).With this dual approach, the engineer builds up an understanding of the behavior of the structure and of the numerical methods in use so that confidence grows, rather than diminishes, as the engineer moves towards a final conclusion.

The approach may need to take account of material nonlinearity (e.g. cracking of concrete, yielding of steel), boundary nonlinearity (e.g. lift-off) and geometric nonlinearity (e.g. cable slackening or buckling). General recommendations for nonlinear FE analyses are given in Annex C of the Eurocode (EN1993-1-5: 2006). In the context of assessment of steel structures it is usefully added that residual stresses are likely to be small in riveted structures and they can usually be ignored. (Guideline for Load and Resistance Assessment of Existing European Railway Bridges: 2007).

In consideration of existing steel structures, buckling presents a particular concern since the mode of failure is sudden, with little or no warning, and can be catastrophic. The collapse of the I35-W bridge in 2007 provided a graphic example, initiated by the failure of an undersized gusset plate (Load-carrying Capacity Considerations of Gusset Plates in Non-load-path-redundant Steel Truss Bridges:2008), (HOA: 2011)

5.1 *Buckling analyses*

Buckling of slender compressive members is caused by the amplification of initial imperfections and of deflections due to lateral loads, and results in a sudden loss of stiffness.

In principle, buckling resistance as determined using a design code of practice curve is conservatively below the “real” buckling resistance according to some reliability index. The real buckling resistance is in turn below the theoretical elastic (Euler) buckling resistance (the point of unstable equilibrium in a theoretical member with no imperfections, no yielding and so on). Yielding in stocky members should also be considered, and the situation described is shown illustratively in Fig. 8. Codes (AASHTO LRFD: 2012), (EN1993-1-1: 2005) use classifications such as compact (class 2), slender (class 4) or non-compact (class 3) to help the engineer identify behavior of a member as being dominated by yielding, elastic buckling, or in the range where the two interact (respectively).

In some codes of practice (BS5400-6: 1999), (AASHTO LRFD: 2012), the design resistance can be derived from generalized slenderness parameters.

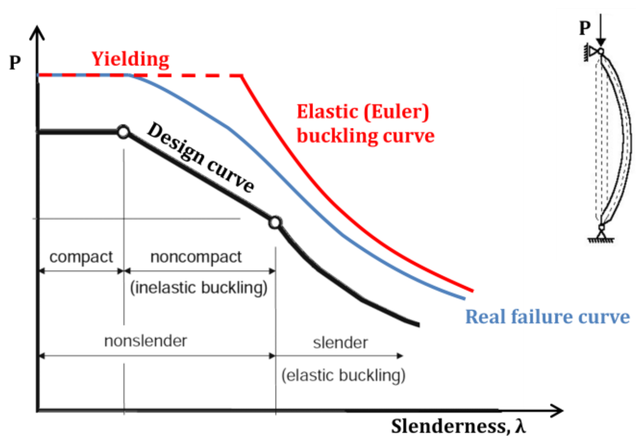

Figure 8. Theoretical, real and codified buckling curves.

Other available rules use the elastic critical buckling force or moment – that is the Euler value –to determine a slenderness parameter which is ultimately used to define a safe design resistance

(EN1993-1-1: 2005), (CAN/CSA S6-06: 2006), (BS5400-3:2000). The elastic critical buckling stress and the mode shape required for such a calculation can be obtained from an eigenvalue analysis using FE software.

All such formulae from codes of practice are intended to be conservative. Being applicable to a wide range of structural arrangements, it may be expected that they are significantly conservative in some cases and less so in others. It should also be considered that they were generally written with design of new structures, rather than rating of existing structures in mind. Therefore it can be appropriate to consider obtaining a more realistic assessment of the buckling resistance of members in an existing structure by use of nonlinear analysis.

As described above, such a nonlinear analysis might need to include boundary, material and geometric nonlinearity. It will generally require initial imperfections to be included. EN1993-1-1:2005 suggests using the shape of the elastic critical buckling mode as an imperfection (see clause 5.3.4) with the amplitude based on the section in question (see Table 6.2 and Table 5.1). Broadly speaking, imperfections are of order span/300 for heavy bridge sections (BS5400-3:2000), (EN1993-1-1: 2005), (BS5400-6: 1999), (AISC 303-10)

Table 1.
Calculated buckling resistance for a braced pair of girders, expressed as a % of the nonlinear prediction.

Calculation method	Buckling resistance
BS 5400 Part 3	50%
Elastic analysis and EN 1993-1-1	78%
Nonlinear analysis using LUSAS	100%

The critical buckling force or moment from the eigenvalue analysis provides a helpful upper bound for the nonlinear results, and the resistance from a code of practice a helpful lower bound (as in Fig. 8).

In a study of bridge girders with restraint to the compression flange provided through bracing in pairs (Fig. 9), UK consultant Atkins compared the buckling resistance predicted using the generalized formulae (BS5400-3: 2000),Eurocode formulae (EN 1993-2: 2006) and a full nonlinear analysis. The results are summarized in Table 1 and indicate that for the girders studied, the nonlinear analysis gave a significantly less conservative assessment of the buckling strength of the girders (HENDY, DENTON, MACKENZIE, and ILES: 2011). The implication for assessment work is clear.

Figure 9. A pair of braced beams.

6 SUMMARY

Assessments progress through several rounds of calculations, employing progressively more advanced analysis approaches. This is cost effective and brings the benefit that the engineer gains a better understanding of the structural behavior and the parameters of most significance to the structure capacity as the process moves on. This understanding must be brought to bear as the analysis increases in sophistication with critical considerations being:

1. Beam models and grillage models are no longer the only available options. Shell element and mixed element models can be more appropriate.
2. Boundary conditions (support stiffness etc), and inclusion of corrosion must be considered.
3. Local models may be required, with some benefits arising from embedding them within global models
4. Nonlinear analyses can be useful. Efficient approaches build on prior linear analysis, add sophistication one step at a time and confidence it built by bounding the results (e.g. using Eigenvalue results as an upper bound code of practice resistances as a lower bound)
5. Specific recommendations for nonlinear analyses should be consulted, such as those relating to initial imperfections when undertaking buckling analyses.

Since the cost and disruption caused by interventions can be very significant, analysis approaches which would not perhaps be regarded as cost-effective for new designs can be justified.

REFERENCES

AASHTO LRFD Bridge Design Specifications, American Association of State Highway and Transportation Officials, 6th Edition, 2012, p. 4-72, p. 4-13.

AASHTO/NSBA Steel Bridge Collaboration, 1st Edition, 2011, G13.1 Guidelines for Steel Girder Bridge Analysis, p. 3-82., p3-10., p. 3-84.

AISC 303-10 Code of Standard Practice for Steel Buildings and Bridges, American Institute of Steel Construction, April 2010, pp. 30-32.

BD 56/10: The Assessment of Steel Highway Bridges and Structures, Design Manual for Roads and Bridges, Volume 3, Section 4, Part 11, Highways Agency, June 2010. p. A/34.

BS5400-3:2000 Steel, concrete and composite bridges – Part 3: Code of practice for design of steel bridges, British Standards Institute, Oct 2000, p. 40., p. 47., p. 74.

BS5400-6:1999 Steel, concrete and composite bridges — Part 6: Specification for materials and workmanship, steel. British Standards Institute, March 1999, p. 24.

CAN/CSA S6-06 Canadian Highway Bridge Design Code, Canadian Standards Association, Nov 2006, p. 484., pp. 450-457.

COST 345: Procedures Required for the Assessment of Highway Structures: Numerical techniques for Safety and Serviceability Assessment. European Commission Directorate General Transport and Energy, Sept 2004, p. 5.

EN 1993-2 Eurocode 3: Design of steel structures – Part 2: Steel bridges, European Committee for Standardization (CEN), Oct 2006, p. 26.

EN1993-1-1: Eurocode 3: Design of steel structures — Part 1-1: General rules and rules for buildings, European Committee for Standardization (CEN), May 2005, pp. 56-68., p. 58. p. 38., p. 34.

EN1993-1-5: Eurocode 3: Eurocode 3 - Design of steel structures - Part 1-5: Plated structural elements, European Committee for Standardization (CEN), Oct 2006

EN1993-1-8 Eurocode 3: Design of steel structures - Part 1-8: Design of joints, European Committee for Standardization (CEN), May 2005, p. 92-98.

Guideline for Load and Resistance Assessment of Existing European Railway Bridges: Advices on the use of advanced methods, Sustainable Bridges, European Commission within the Sixth Framework Programme, 2007, p. 119., p. 200.

HAMBLY, E. C. Bridge Deck Behavior. E C. Taylor and Francis, 2nd Edition, 1991. p. 281. p. 108-127.

HAYWOOD A., SADLER N. and TORDOFF D., "Steel Bridges: A Practical Approach to Design for Efficient Fabrication and Construction" BCSA Publication no 34/02, Dec 2002, p. 37.

HENDY C., DENTON S., MACKENZIE D., and ILES D., EN1993 Practice Paper: Buckling Analysis of Steel Bridges, Bridge Design to Eurocodes – UK Implementation, ICE Publishing, Oct 2011

HOA S., "I35W Bridge Collapse: Lessons Learned and Challenges Revealed" Structures Congress 2011, ASCE, 2011.

Inspection Manual for Highway Structures: Volume 2: Inspector's Handbook, The Stationary Office, London, May 2007

Load-carrying Capacity Considerations of Gusset Plates in Non-load-path-redundant Steel Truss Bridges, Technical Advisory 5140.29, U. S. Department of Transportation Federal Highway Administration, Jan 2008

MEHRKAR-ASL S, et al, "Saving Half Through Girder Bridges using Non-Linear Finite Element Analysis", International Bridge Management Conference 2005.

O'BRIEN and KEOGH. Bridge Deck Analysis. E and FN Spon, London, 1999, pp. 245-246.

West Gate Bridge Upgrade, http://www.lusas.com/case/bridge/west_gate_bridge.html Retrieved 10 October 2012