



**David Baird**

Olympic Delivery Authority



**Chris Hendy**

**Head of Bridge Design  
and Technology**

Highways & Transportation



**Patrick Wong**

**Senior Engineer**

Highways & Transportation



**Rachel Jones**

**Senior Engineer**

Highways & Transportation



**Tony Sollis**

**Senior Engineer**

Highways & Transportation



**Hayden Nuttall**

**Structural Design  
Director**

Design & Engineering  
Solutions

## Abstract

In preparation for the 2012 Olympic and Paralympic Games, many bridges were required crossing over waterways, highways and railways. This paper describes the technical challenges on two of the Atkins designed bridges - H01 and L01.

Bridge H01 is a single-span integral highway bridge of composite steel box-girder construction that carries a local distributor road over the River Lea in the north of the Olympic Park. It was the first box girder bridge to be erected in the Olympic Park and was opened to construction traffic in October 2009. It is planned to be opened for public use in December 2010 for access to Stratford City. Box girders are torsionally stiff in their completed state but are less so during construction if they are "open top", as here, and require restraint to the compression flange during deck slab construction to prevent lateral buckling. The design of Bridge H01 utilised vertical cross-bracing at regular centres for this purpose, but detailed analysis indicated that this alone was insufficient to prevent significant second order twisting effects under torsional loading. The paper discusses how the designers addressed these issues and how updated national box girder design guidance was produced as a result.

Footbridge L01 crosses Ruckholt Road and provides the vital northern link to the Olympic Park for pedestrians to and from the Northern Spectator Transport Mall. Its construction is expected to be completed in June 2011. The single span structure is a unique form of construction whose behaviour is a hybrid between that of a tied arch and Vierendeel girder. The steel bridge deck itself is 42m long and 5.5m wide, while the arch rib is made exceptionally slender by exploiting the overall Vierendeel girder behaviour. The closely spaced solid plate hangers provide both lateral and in-plane stability while creating a striking effect in elevation. The complex geometry required elastic critical buckling analysis to determine the resistance of the arch and non-linear analysis to assess the effects of arch deformation on the rigid hangers and deck. The main learning points from the analysis and design of this unique structure are presented in this paper.

## Introduction

The Olympic Park bridges in Stratford include 11 highway bridges, 13 pedestrian bridges, 6 underpasses and many other temporary bridges with spans up to 56.5 m. Many of them carry major utilities over waterways or railways. The bridge and associated wingwall designs, including finishes, are co-ordinated to fit seamlessly with the Park's topography and landscape during the Games period and in Legacy. Minor modifications to some bridges will be required after the Games for Legacy transformation. This paper describes the particular technical challenges encountered on two of the bridges, H01 and L01.

## Bridge H01

Bridge H01 is a 52 m single-span integral highway bridge of steel-concrete composite "open top" box-girder construction. It is situated in the north of the Olympic Park and carries a local distributor road over the River Lea. It was the first box girder bridge to be erected in the Olympic Park and was opened to construction traffic in October 2009. It is planned to be opened for public use in December 2010 for access to the Stratford City development. The general arrangement is shown in Figure 1.

An open top box girder solution was adopted for Bridge H01 for a number of reasons:

- The absence of outstand flanges give a sleek appearance and reduce opportunity for bird roosting.

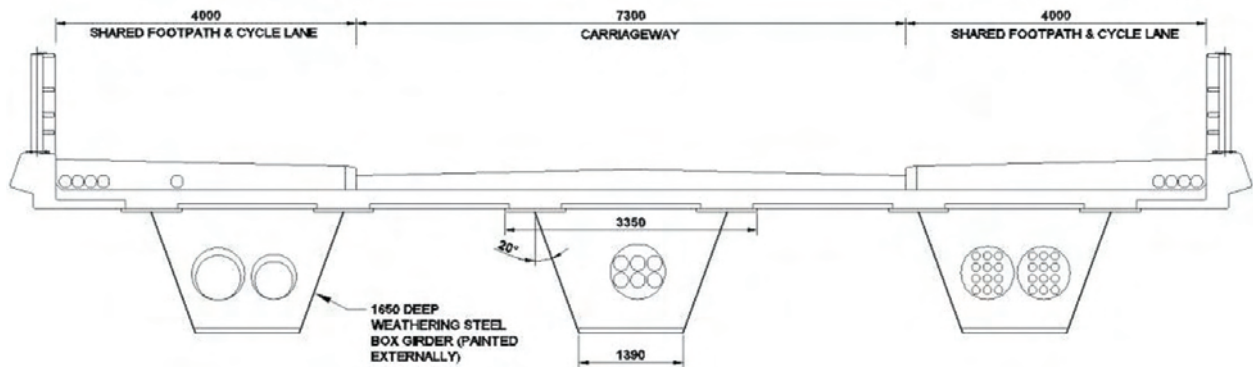


Figure 1 - Cross section of bridge H01

- The inclined webs increase the amount of light under the bridge and reduce the apparent depth of the structure when viewed in elevation
- The services running over the bridge can be concealed within the box (Figure 2), thus improving the aesthetics
- The absence of a steel top flange give material savings compared to closed box solutions and reduced the health and safety risks associated with fabricating closed boxes.

These advantages had to be balanced against a number of disadvantages, however, including:

- Higher fabrication costs and heavier lift weights than for a plate girder
- Greater difficulty in providing the corrosion protection, inspection and maintenance compared to a plate girder due to the internal surfaces. This led to the use of weathering steel to prevent the need for painting the inside of the box

- The need to restrain the outstand top flanges against buckling under wet concrete loading, which would not be required with closed top boxes. This proved to be quite a significant consideration for the particular geometry of bridge H01.

#### Structural analysis

The structural analysis for the in-service case of bridge H01 was straightforward. The bridge was analysed by grillage analysis following the recommendations of the SCI



Figure 2 - Bridge H01 prior to deck slab construction



Design Guide for Composite Box Girder Bridges<sup>1</sup>. A single spine member with outriggers to the positions of the webs was initially used and the distortional flexibility of the boxes was modelled by artificially softening the torsional stiffness of the box; this was an approximation since the distortional flexibility depends on the particular loading applied but the relatively short span, thick plates and small distance between cross braces on Bridge H01 meant that distortional flexibility was very small.

The design for the wet concrete construction case was more challenging and its treatment is much less well documented. During construction, when the concrete slab is not in place, the St. Venant torsional inertia is that of an open section and is therefore several orders of magnitude lower than that of the closed box in the final situation above. However the single spine beam approach with a reduced torsion constant is inappropriate because the open box behaves more like a pair of plate girders in this situation and there is an increase in effective torsional stiffness provided by in-plane bending of the webs and flanges as the cross section twists (the warping stiffness).

Suitable vertical (torsional) bracings or diaphragms are obviously required in this erection condition to give stiffness to the cross-section. Consideration was given to providing top flange plan bracing at this stage to create a pseudo box and eliminate the torsional flexibility. This was ruled out since the plan bracing would have needed to be attached to the underside of the flanges to avoid interfering with deck slab construction and connection to the underside of the flange would have been awkward to construct.

It was clear that the top flanges in compression would need to be checked for buckling. It was initially considered that the open top box could be modelled in the same way as a pair of I-girders with a common flange and that the common flange would effectively provide lateral restraint to overall lateral buckling in the same as does a deck slab in the completed condition of a steel-concrete composite I-girder deck. This is effectively the approach in reference 1 where it is assumed that buckling of the top flange in compression will occur between cross braces and there is no overall instability of the box. There was however concern as to whether the bottom flange was wide enough to provide this lateral stability and whether a global mode might occur.

Boxes with sloping webs, like Bridge H01, are particularly likely to produce this situation. The reason for this is that the elastic critical buckling moment for lateral torsional buckling has the generic form:

$$M_{cr} = \left[ \frac{\pi^2 EI_z}{L^2} \left( GI_T + \frac{\pi^2 EI_w}{L^2} \right) \right]^{0.5} \quad (1)$$

$I_w$  is the warping constant;

$I_z$  is the minor axis second moment of area;

$I_T$  is the St. Venant torsional inertia;

$L$  is the length of the beam between points of torsional restraint.

Expression (1) only applies to bisymmetric sections (not open top boxes) with uniform end moments but it illustrates the resisting components which mainly comprise St Venant torsional stiffness,  $GI_T$ , and warping stiffness  $EI_w/L^2$ . Open top boxes have very low St Venant torsional stiffness so the first term cannot be relied upon to provide any significant resistance (For closed boxes, this term is very large and effectively eliminates lateral torsional buckling for all but the most extreme aspect ratios of tall and narrow boxes). The second term therefore provides the majority of the resistance to lateral torsional buckling.

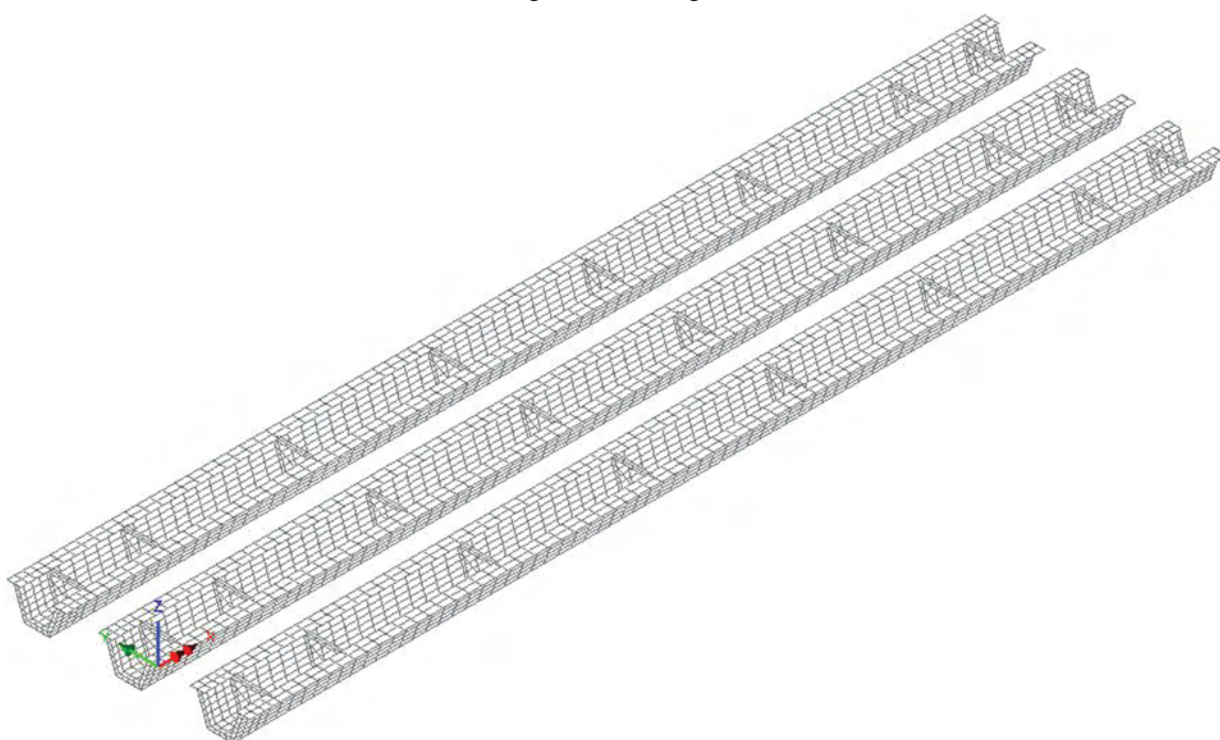


Figure 3 – Shell finite element model for the open top case

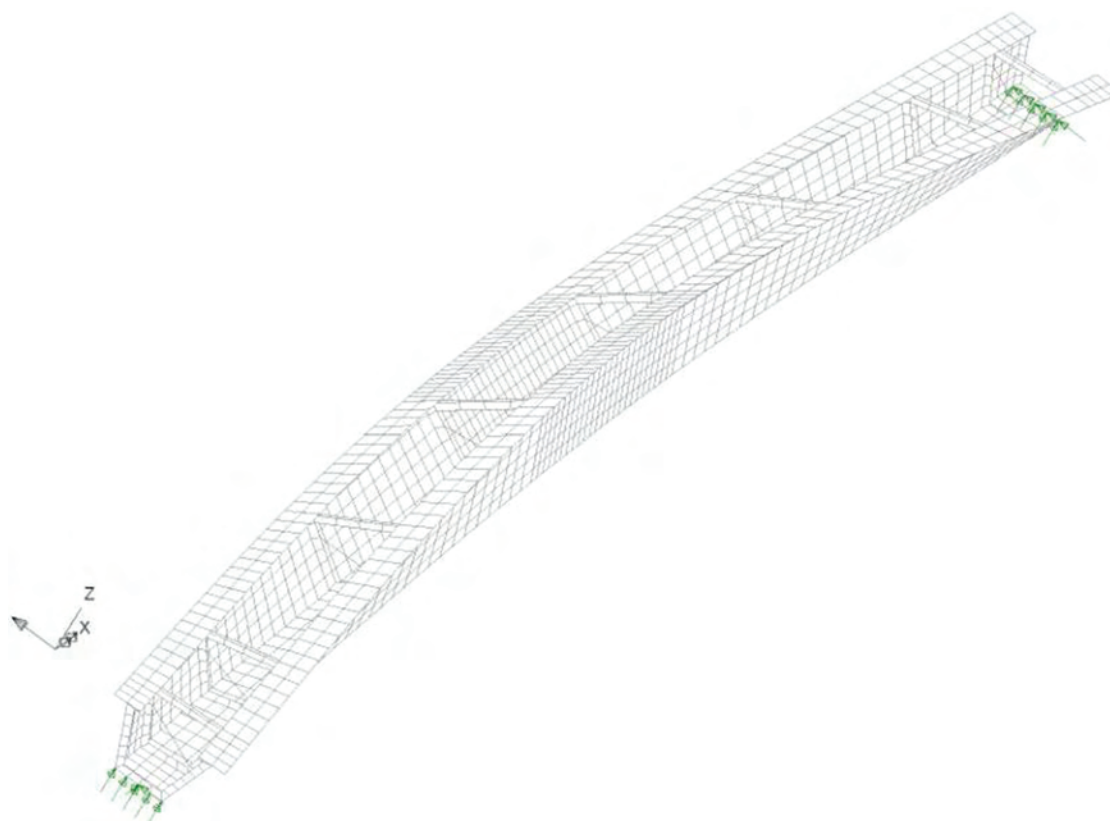


Figure 4 - Lowest buckling mode under application of wet concrete and self weight

The stiffness here depends on how much stiffness the individual plates can provide by in-plane bending when twisted about the cross section's shear centre. Narrow boxes with sloping webs provide less stiffness because of the reduced flange width (and therefore reduced transverse bending stiffness) and the reduced lever arm between the webs. In the limit, a V shaped section with no bottom flange will have a shear centre at the junction between the webs and therefore essentially no warping stiffness. Torsional stiffness is then provided only by the GIT term and the lateral torsional buckling resistance of the overall section will be very low.

There was again consideration given to the provision of plan bracing to the top flange to ensure that the buckling length was restricted to the length between cross braces but it was finally decided to analyse the open top box with a 3-D shell finite element model (Figure 3) to determine  $M_{cr}$  directly and hence obtain the slenderness for buckling following the approach in Eurocode 3.

This analysis was completed and a global mode was indeed obtained as seen in Figure 4. The slenderness for this mode was very much lower than that considering only buckling of the top flange between the cross braces and, although adequacy of the boxes under concreting was demonstrated, there was little reserve of strength available in this condition.

### Construction and lessons learnt

The erection of the steelwork went ahead as planned in October 2008. It soon became clear that the girders were more torsionally flexible than had been imagined (Figure 5a), with significant rotation of the girders having occurred under the relatively light loading of empty service sleeves which had not been thought to be significant. Indeed linear elastic analysis with the shell model shown in Figure 4 had predicted an expected rotation causing only a few millimetres difference between the top flange levels. The real rotations produced created a problem with the final levels of the box top flanges and left much reduced tolerance for the slab construction.

Further recourse to the shell model revealed why the twist was so much greater than expected. The ratio of elastic critical buckling load to applied load (the critical load factor,  $\alpha_{cr}$ ) determined for use in the slenderness calculation was approaching unity for the worst load case. Whilst this ratio had been used to calculate the ULS resistance and demonstrate adequate strength, the second order deflections arising from both initial torsional imperfections and minor eccentric loading had not been calculated directly. Eurocode 3 provides a magnification factor to apply to first order deflections to calculate the resulting deflections including second order effects. This factor is  $1/[1-(1/\alpha_{cr})]$ . Applying this to the Bridge H01, the rotations predicted by a linear elastic analysis multiplied by the magnification factor gave figures of a similar order of magnitude to those observed on site.



Figure 5 - (a) Twisted box under eccentric load; (b) Addition of plan bracing

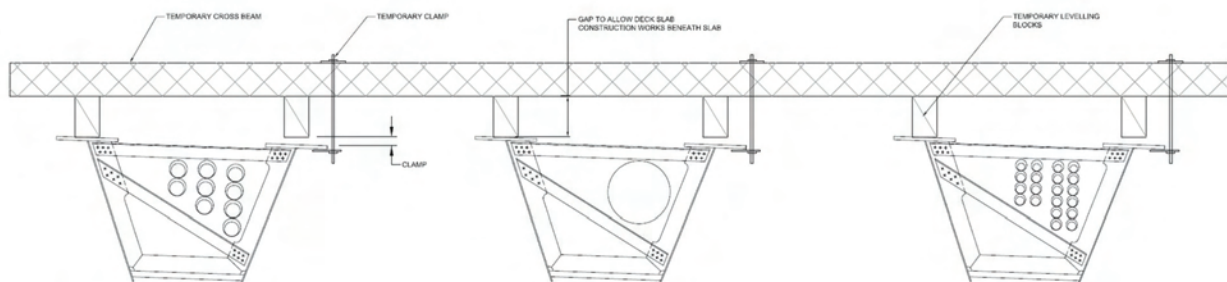


Figure 6 - Temporary cross bracing to reduce box twist

To remedy the situation on site, temporary cross-beams were used to straighten the twist (Figure 6). Initially the option was considered to pour a phased sequence of deck concrete with the cross-beams in place, then removing the cross-beams after deck concrete had cured. However, the final option chosen was to use the cross-beams to straighten the girders, retro-fit permanent plan-bracing with the cross-beams in place (Figure 5b), remove the cross-beams, then pour the concrete as planned. This was thought to offer the least risk. This went ahead without difficulty and the girder stability was maintained by the plan bracing. Clearly the detailing of plan bracing in the first instance would have mitigated the need for this additional construction activity but the decision to omit it had been made with considerations of safety foremost. In hindsight, it would have been better to detail plan bracing, paying particular attention as to how to detail its attachment with the minimum risk to erectors.

The results of this work led to national box-girder design advice being updated by way of an 'Advisory Desk' note published by the SCI in New Steel Magazine<sup>2</sup>. This was particularly necessary as the existing guidance in reference 1 did not draw attention to either the strength or deflection issues associated with open top boxes.

## Footbridge L01

### Bridge description

Bridge L01 provides a pedestrian and cycle link between the Northern Spectator Transport Mall and the Olympic Park. It will serve as one of the main gateways to the Park during the games and so a striking appearance was a key criterion for the design. Construction of the bridge commenced in May 2010 and is expected to be completed in June 2011. After the 2012 Games it will link the Hackney Marshes playing fields to the Legacy facilities of the park. Figure 7 shows the location of Footbridge L01.

The main architectural feature of the footbridge is the exceptionally slender arch with a dense distribution of steel plate hangers, referred to as slats (Figure 8). The arch spans 42m and each rib is formed from a single steel plate 400mm x 90mm. The vertical slats are formed of 135 mm x 25 mm plates at 135mm centres. The 5.5m wide steel deck is formed from two 550mm x 300mm box sections with transversely spanning "T" ribs at 540 mm centres. The deck is finished with concrete screed, waterproofing and resin bound aggregate surfacing. The deck is tied to the arch with a fully welded connection at deck level. Either side of this connection the vertical slats are replaced with a ribbed steel plate joining the deck and arch. The deck is integrally connected to the foundations which comprise reinforced concrete abutment walls on piles. The bespoke parapet at the ends of the arch match the vertical slats. At midspan, the square hollow section parapet rail is welded directly to the vertical slats.



The visual concept for the bridge was developed by Allies and Morrison Architects and Atkins and the detailed design was carried out by Atkins. The key challenge of the design was resolving the structural integrity of the bridge without compromising the architectural vision. The resulting structure is a unique form of construction whose behaviour is a hybrid between that of a tied arch and Viereendeel

girder; the arch rib achieves its high slenderness by exploiting the overall Viereendeel girder behaviour.

### Structural design

The footbridge was designed to BS5400 Part 3<sup>3</sup> but reference was made to BS EN 1993-1-1<sup>4</sup> for checks on buckling of the structure. The bridge was modelled using beam elements representing the main

structural elements. The lateral restraint provided by the steel deck plate was not included in the structural model but the section of hand rail welded to the vertical slats was included. Careful consideration needed to be given to the modelling of the foundations as the bridge is integral with them and the fixity to the foundations and foundation stiffness had a significant impact on the structural behaviour.



Figure 7 - 2012 Olympic Park plan

Figure 7 - 2012 Olympic Park plan



Figure 8 - Final concept for Bridge L01

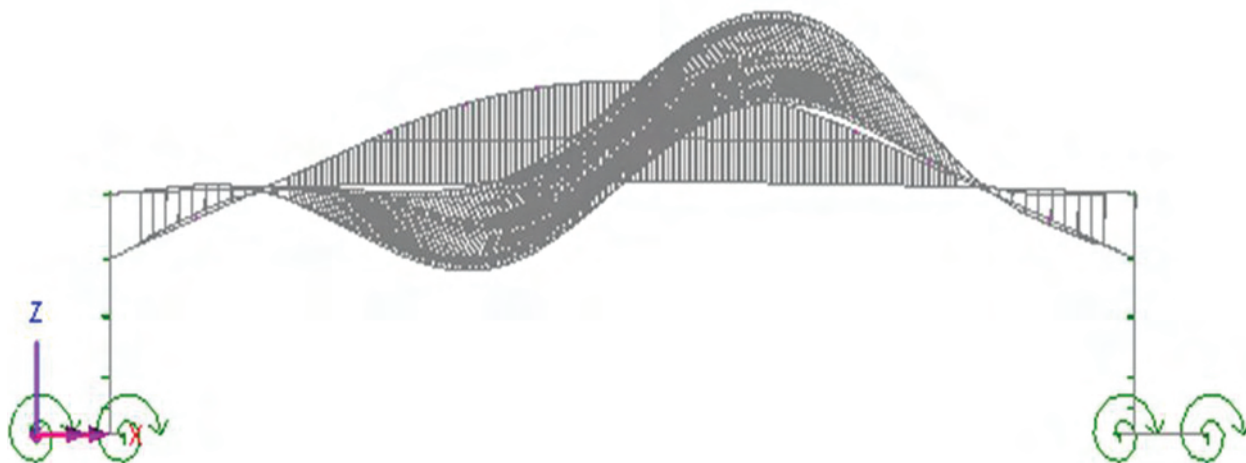


Figure 9 - In-plane buckling mode

The abutment walls were modelled using a grillage of beam members and the forces from the soil pressure were applied to the structural model as an applied load. The piled foundations were modelled using spring elements with an appropriate stiffness.

### Global buckling

The arch ribs are necessarily very slender and rely on the vertical slats heavily for its stability. One of the key issues in the design was accurately assessing global buckling effects. The beam element model was used to carry out an elastic critical buckling analysis to determine the susceptibility of the structure to global buckling, both in and out of plane. The lowest of these buckling loads determined from this analysis was then used to determine a slenderness for buckling and codified buckling curves were used to determine a reduction factor to apply to the resistance of the arch and deck.

The structure was susceptible to two main modes of global buckling, in-plane and out-of-plane as shown in figures 9 and 10. The critical mode varied depending on the exact dimensions chosen for the main structural components, the foundation design parameters and the loading applied. Throughout the design iteration process the critical mode of buckling changed as dimensions of different elements were varied and the arch profile was finalised. In the final form of the bridge, the in-plane buckling mode involved the arch and deck and was critical for temperature loading on the structure. The out-of-plane mode just involved the arch and was critical for pedestrian loading effects. The out-of-plane buckling of the arch was resisted by the slats through u-frame restraint. The resistance to in-plane buckling was higher than might be expected for such a slender arch profile.

The greater buckling resistance is provided by the vertical slats which, although providing little stiffness individually, act together at 135 mm centres to provide significant restraint to the arch through their moment connections at each end, forcing the deck and arch to act together.

The British Standards provide little guidance on how to account for the buckling arched structures and so for this structure the approach provided in EN 1993-1-1 Design of Steel Structures Clause 6.3.4 was used. Clause 6.3.4 is written as a general method for checking out of plane (lateral) buckling of members and frames when the axial force and bending moment both give rise to out of plane buckling of the element(s) i.e. the axial force or bending moment applied separately would lead to lateral buckling of the element(s).

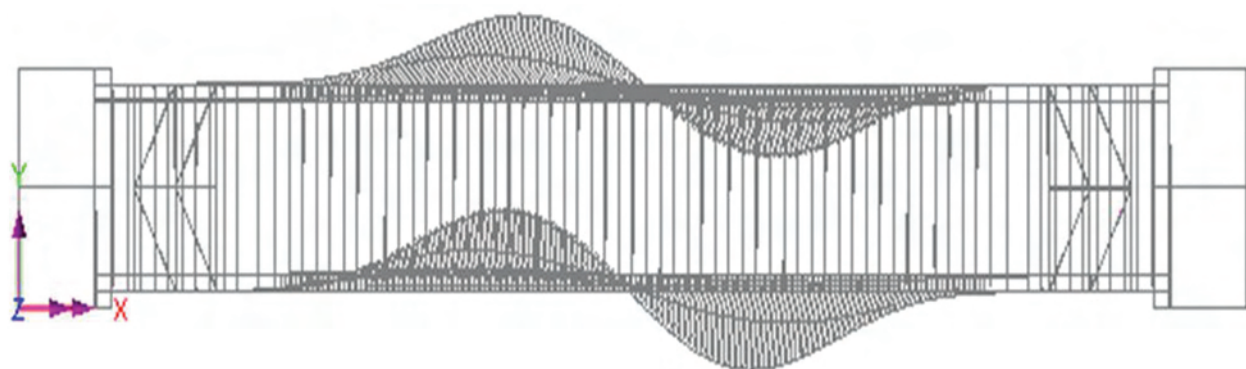


Figure 10 - Out-of-plane buckling mode

In such cases, it is logical that the cross section resistance used in the slenderness calculation be based upon both the axial force and the bending moment together, because both cause lateral buckling of the system i.e.

$$\bar{\lambda}_{op} = \sqrt{\frac{\alpha_{ult,k}}{\alpha_{cr,op}}}$$

BS EN 1993-1-1 expression (6.64)

where:

$$\frac{1}{\alpha_{ult,k}} = \frac{N_{Ed}}{N_{Rk}} + \frac{M_{y,Ed}}{M_{y,Rk}}$$

$\alpha_{cr,op}$  is the load amplifier for elastic critical out-of-plane buckling.  $N_{Ed}$  and  $M_{y,Ed}$  are the applied axial force and bending moment and  $N_{Rk}$  and  $M_{y,Rk}$  are the characteristic axial force and moment resistances.

It was noted that the assumptions behind clause 6.3.4 were not satisfied for bridge L01 for two reasons. Firstly, only the arch axial force produces lateral buckling, not the bending moment. Secondly, the application of this clause to the buckling of the arch for in-plane buckling is outside the intended scope of the clause. Despite these apparent violations, the method was established as providing a safe approach through comparisons with the results obtained from a full non-linear analysis as discussed below. This was important to establish as the use of elastic analysis to do the design had obvious benefits over non-linear analysis given the number of load cases and iterations of member sizes needed to refine the design and the relative speed of elastic analysis compared to non-linear analysis.

An alternative and apparently more theoretically-sound approach for in-plane buckling would have been to base the slenderness of the arch on the axial force alone and then amplify the coexisting moments in accordance with the verification below. However the distribution of moment within the half wavelength of buckling varied significantly along the arch so the moment magnifier would have been very conservative in this case, and was shown to be so from the results of the non-linear analysis.

$$\frac{N_{Ed}}{\chi N_{pl,Rd}} + \frac{1}{1 - (1/\alpha_{cr,ip})} \frac{M_{y,Ed}}{M_{Rd}} \leq 1.0 \quad (2)$$

$\alpha_{cr,ip}$  is the load amplifier for elastic critical in-plane buckling.

One final point to note on the application of clause 6.3.4 of EN 1993-1-1 is that the UK National Annex limits its application to nominally straight members.

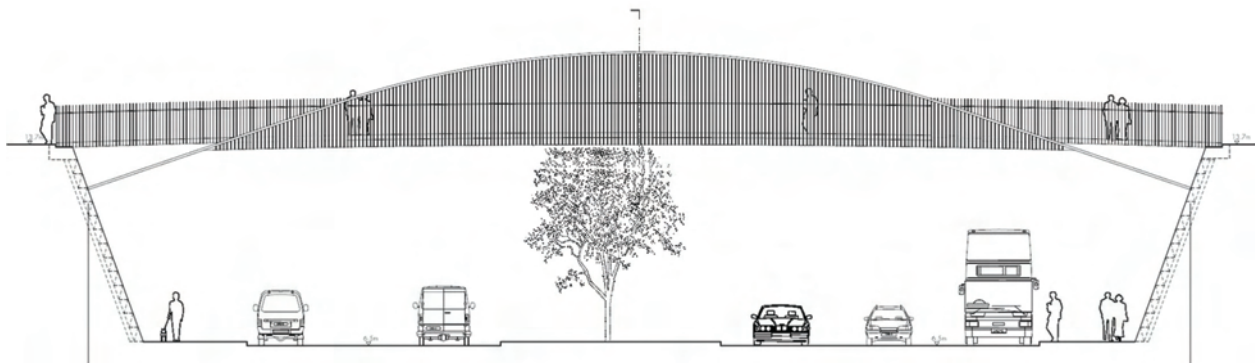


Figure 11 - Original concept for Bridge L01

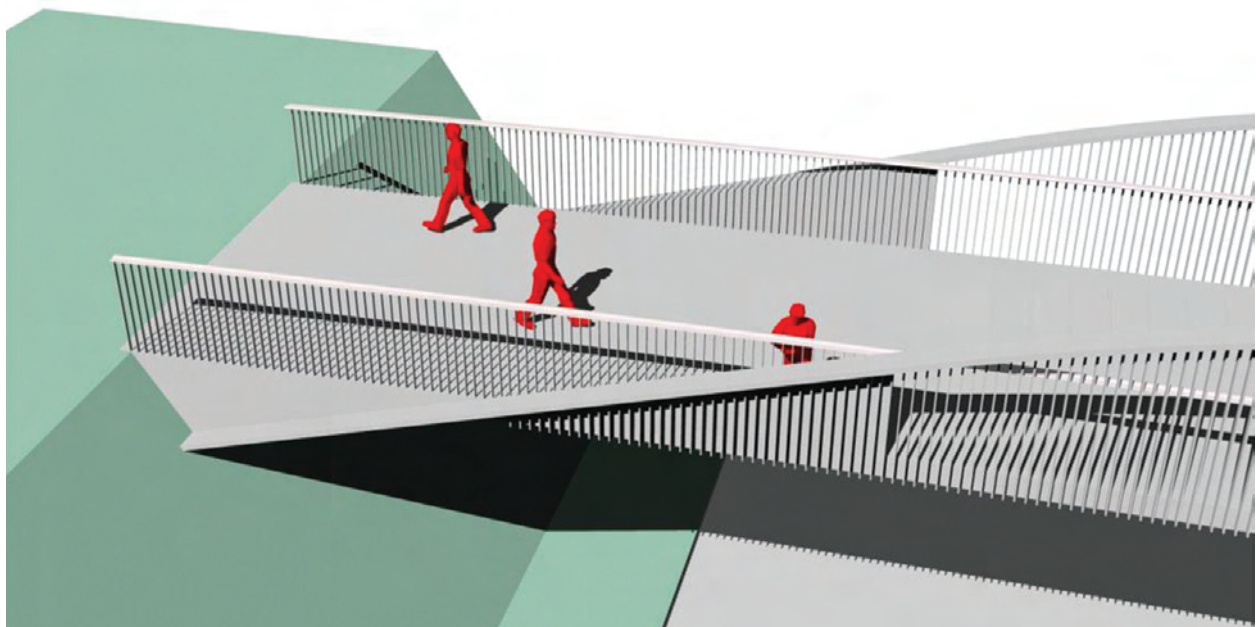


Figure 12 - Arrangement of slats at end of arch



The restriction is considered to be unnecessary by the authors of this paper and was not intended by the Eurocode drafters; moments from initial curvature are included in the calculation of  $M_{y,Ed}$  satisfactorily. Indeed, the example of application of the clause prepared by the EN 1993-1-1 Project Team features a curved member<sup>5</sup>. For the purpose of this design, which was to British Standards, a Departure from Standard was obtained for the application of the Eurocode clause, disregarding the restriction in the UK National Annex.

### Design of the vertical slats

The elastic critical buckling approach is sufficient for the design of the arch members but the buckling loads obtained rely on restraint provided by the vertical slats and the elastic critical buckling analysis provides no information about the second order effects generated in the slats. As the arch deforms, the slats are bent into double curvature and bending moments are generated even if there was none produced by the initial first order analysis. A method based on magnification of the first order moments (in the absence of modelling initial imperfections) is therefore inappropriate in this case.

In order to establish these secondary bending effects in the slats and their connections, a finite element analysis with geometric nonlinearity was carried out. Conservatively and for comparison with the codified member buckling rules, material nonlinearity was not included and the stress-strain behaviour was restricted to be linear elastic with stresses limited to first yield. Initial imperfections were introduced to the structure in the buckled shapes as shown in Figures 9 and 10. The magnitude of these initial imperfections was set in accordance with the guidance of EN 1993 -1 -1 for equivalent geometric imperfections which include allowances for both construction tolerances and residual stresses in the structure. The results of the nonlinear run were used to establish the increased stresses in the vertical slats in restraining buckling of the arch and to check that the arch buckling resistance predicted by clause 6.3.4 of EN 1993-1-1 was satisfactory.

In the former regard, it was found that the stresses in the slats were typically twice that predicted from first order analysis. In the latter regard, a comparison of the applied loads predicted to cause first yield from the elastic critical buckling analysis and from the non linear analysis showed that they were within 5% of each other, with the nonlinear analysis giving the slightly higher resistance.

### Other design considerations

During the design process there were a few aspects of the initial concept that had to be modified to provide structural integrity. The original concept (Figure 11) had the arch unconnected to the deck below deck level. This long free length of slender compression member proved unfeasible and a number of options for restraining it were considered. The final solution was to continue the vertical slats below the deck to restrain the arch. These would be acting in compression and so a solid plate was provided to restrain the vertical plates. Cross bracing between the two arches was also provided below the deck. Analysis of the bridge also showed that in some loading situations the shorter slats above the deck were acting in compression. The buckling resistance of these flat plate elements is extremely low so a solid plate was again provided in this region to restrain the slats as shown in Figure 12.

## Conclusion

Bridge H01 and L01 are very different structures, the first being an established form of construction and the second being a highly novel form. However, this paper identifies that existing industry and codified guidance is not always sufficient to ensure that a structure with unusual buckling behaviour is correctly modelled and the behaviour correctly accounted for. This paper serves to illustrate a suitable approach to the design of open top boxes for all designers to use and offers some cautionary notes for designers of unusual structures.

## Acknowledgements

The authors would like to acknowledge the Olympic Development Authority for their support and permission to publish this paper and Allies and Morrison Architects for permission to use their images of bridge L01.

## References

1. Design Guide for Composite Box Girder Bridges (Second Edition), Steel Construction Institute, Ascot, UK
2. SCI Advisory Desk note AD 330, "Open top box girders for bridges", New Steel Construction, February 2009
3. BS 5400-3 (2000) (incorporating Corrigendum No.1) Steel, concrete and composite bridges-Part 3: Code of practice for design of steel bridges , BSI, London
4. BS EN 1993-1-1 (2005): Design of Steel Structures. Part 1.1: General rules and rules for buildings. British Standards Institution, London.
5. CEN/TC250/SC3/N1639E, CEN background document