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Assessment of U-type wrought iron railway bridges

Lee Canning MEng, PhD
Principal Engineer, Jacobs Engineering, Manchester, UK
(corresponding author: lee.canning@jacobs.co.uk)

Mohammad M. Kashani BSc (Hons), MSc, PhD
Lecturer, Department of Civil Engineering, Faculty of Engineering, University of Bristol, Bristol, UK

A number of assessment codes and guides exist that take into account the particular details of old U-frame types and riveted wrought iron construction; however, a significant number of old bridges still have details that cause difficulty in reliable assessment or require very conservative assumptions, giving low assessed capacity. In this paper, an alternative assessment method using non-linear finite-element analysis of an old U-frame railway bridge is presented. By using the analysis results, some guidelines and recommendations are suggested that are considered to be useful for assessments by the engineering community. However, this approach also has limitations, as it is reliant on assumptions that can be difficult to confirm without extensive field monitoring and experimental testing. The analysis results showed that main girders with non-code-compliant U frames do indeed benefit from a low but significant level of restraint to resist lateral torsional buckling and also that girders that are marginally non-compact based on codified assessment methods may actually have sufficient local stability to provide additional flexural capacity beyond the elastic moment capacity.

1. Introduction

The reliable structural assessment of old bridges is fundamental to effective asset management of the UK bridgestock. In particular, monitoring, maintenance and strengthening/replacement decisions are heavily based on assessed capacity as well as structural condition. A large proportion of the railway bridges in the UK comprise steelwork U-frame types, and a significant number of these bridges comprise older types of U frame that have not been used in bridge design and construction for many decades. A common example of this is the presence of cross-member end connections not coincident with the main girder web stiffeners, which was explicitly not covered in BS 5400-3:2000 (and not recommended) (BSI, 2000) and is also not always covered by assessment standards. This now obsolete bridge type also exists outside the UK and for road bridges, with known examples in Australia and elsewhere.

This situation leaves the assessing engineer in a quandary, as either the U frame is assumed to not exist (giving a very low assessed capacity, often much less than the bridge has obviously experienced previously on a daily basis) or a more optimistic engineering judgement is made that the U frame does exist and is stiffer and stronger (typically giving a much higher assessed capacity). There is no reliable middle ground if only simple assessment methods are used. It is also difficult to incorporate reliably the effects of poor condition (e.g. section loss) or other defects. The assumption of whether a reliable U frame exists or not, and the type of U-frame connection, will also have an effect on web buckling capacity, although this is not considered in detail in this paper.

Here a case study of an example of this problem is presented for a very common type of railway underbridge that was assessed recently. The bridge comprised a 10-m span with riveted wrought iron main girders and transverse wrought iron troughing carrying ballasted track. The assessment methods comprised both simple assessment to codified rules in the Network Rail assessment standard NR/GN/CIV/025 (Network Rail, 2006) and more complex assessment by finite-element analysis (FEA). It should be noted that FEA is recommended as one additional approach in NR/GN/CIV/025 if a structure does not meet the assessment criteria using simplified methods. Comparison of these methods shows that many significant assumptions have to be made for both the simple assessment and the assessment using FEA.

No firm conclusions are given on what may be the most appropriate assumptions to use, but the range of assumptions and an indication of their effect on assessed capacity are discussed. It is of interest to wonder whether some form of, and to what extent, U-frame action was ever intended to exist in the original design of this type of structure. In many instances U-frame fixing details appear to be located only to suit construction ease rather than structural effectiveness or at least a balance of these factors. This paper gives an insight into this complex problem to the assessment community and researchers. Given the significant paucity on this subject in the technical literature and assessment standards, the aim of this paper is to raise debate in the assessment community and enable improved, more consistent assessment methods or assumptions to be developed. In the absence of this development, a large number of bridges will need to be considered for strengthening or replacement in the future at great cost and disruption.

2. Typical underbridge case study

The underbridge is a simply supported 10-m span U-frame type bridge crossing the River Cam, Cambridge, UK in a rural setting (Figure 1), originally constructed *c.* 1895. The bridge superstructure comprises riveted wrought iron girders supporting transverse troughing and two ballasted tracks, at a variable skew due to a differing abutment orientation. The superstructure is supported on metal-bearing plates seated on concrete-bearing

Figure 1. Underbridge over River Cam

shelves and brickwork abutments. The general arrangement of the bridge is shown in Figure 2. This underbridge type is common in the UK with hundreds of bridges of similar age and structural form, although occasionally the track is supported on longitudinal timbers directly fixed onto the trough rather than ballasted track.

Inspections and examinations showed that the bridge superstructure is generally in good condition, with the majority of the paint system intact and only minor section loss to localised areas (such as the lower part of the webs in the trough area). The centre girder had been strengthened previously using a 'top-hat' solution bolted onto the top flange of the original girder top flange including packing plates. Record drawings also showed that in some locations the packing plates providing a level bearing surface for the bottom of the trough had also been replaced.

A particular defect identified on this structure was the loss of rivets, or evidence of loose rivets, at a small number of trough–girder connections. Additional inspections focusing on each and every trough–girder connection showed that the details of the connections vary (Figure 3). In some connections, presumably original, the upper trough connection comprises only an angle cleat connected to the trough and girder web with two rivets each. However, the rivet edge distance and the general 'good fit' of the angle cleats vary considerably, suggesting that when the bridge was constructed the girders were initially installed, followed by the trough units, which were then connected to the main girders using angle cleats to give the best fit possible considering tolerances. Other connections have been strengthened by welding of the top of the angle cleat to the girder web or replacement of rivets with bolts. A small number of angle cleat connections have missing rivets; the condition of existing

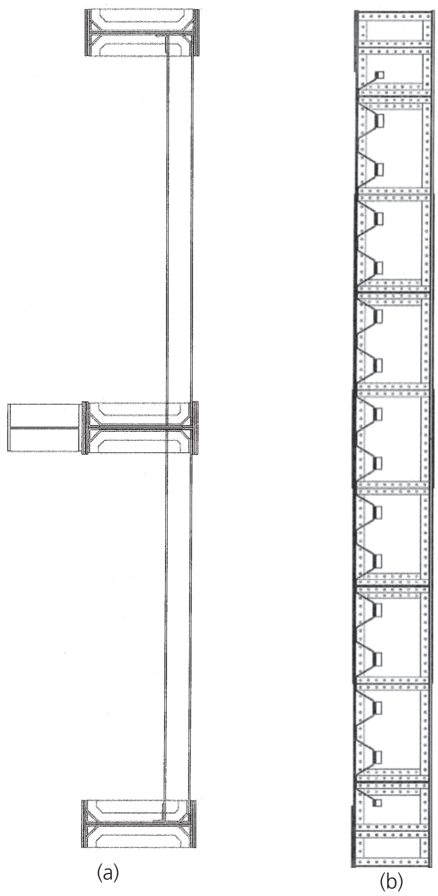

Figure 2. (a) Cross-section of underbridge (track omitted). (b) Elevation of underbridge showing edge girders and trough deck

(a) (b) (c) (d)

Figure 3. Angle cleat fixings showing variability due to rivet location/loss, bolt replacement, welding and fitting tolerance

angle cleat rivets is good, which provides tentative evidence that this loss of rivets is due to true fatigue/static failure rather than corrosion, although it is not certain. A small number of angle cleat rivets also appear to be loose (Figure 4) as evidenced by rust staining locally around the rivet heads – possibly allowing limited movement at particular U-frame connections.

An additional detail of interest was the presence of two tie bars per span and per track, connecting the edge girders to the centre girder within the ballast just above trough level (Figure 5). However, one of the tie bars was confirmed by inspection to be broken, and the condition of the others could not be determined without intrusive investigation, as they were buried within the ballast. It is thought that the tie bars were not part of the original construction and may be a historical attempt to improve girder stability due to concerns regarding the effectiveness of any U-frame action that may or may not exist. It is unclear how the tie bars were intended to prevent inward movement or rotation of the edge girders, as nuts are located only to the external faces of the edge girder and the tie bar itself would have a very low compressive buckling capacity.

It is noted that most of these defects are not particularly unusual and can be found on many old bridges of this type.

3. Structural investigation and assessment

Additional investigation was undertaken to this bridge to confirm material type and construction tolerance. Removal and tensile testing of small sections of metal from low-stressed areas of the girders confirmed the material type to be wrought iron in accordance with NR/GN/CIV/025. Construction tolerance, in particular geometric bow in the webs and top flange of girders, is known to affect the real and the assessed capacities of girders susceptible to lateral torsional or web buckling. The NR/GN/CIV/025 assessment standard acknowledges this by placing limits on out-of-straightness

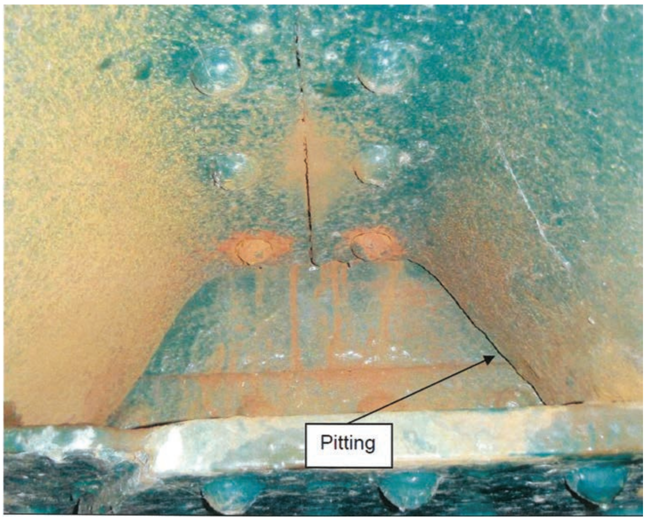

Figure 4. Loose rivets between girder and trough

Figure 5. Tie bar above trough deck level

in elements where buckling instability may be critical (e.g. U-frame girders) based on the construction tolerances stated in BS 5400-6:1999 (BSI, 1999) and relied on in BS 5400-3:2000 (on which the NR/GN/CIV/025 assessment standard is based). Outside these limits the simple assessment methods cannot be used or are modified based on measurements. Measurements taken on site using inclinometers, string lines and laser lines confirmed that the in-plan bow of the top flange of the edge girders was at the tolerance limit of span/1000 or approximately 10 mm (towards the centre girder) and the verticality of the web panels was outside the tolerance limit of twice the web depth/200 or approximately 9 mm.

A number of assessments of this bridge had been undertaken previously, and they show how assessment methods have varied over recent decades. Although no records are available, the bridge would originally have been assessed about 20 years ago using the ‘lucky dip’ factor approach (Baulk and Yu, 1995), which essentially comprised an estimate or judgement of the likely effective length of a girder based on particular U-frame types (including now obsolete types). This method uses empirical judgement, although it was based on numerical work by British Rail and generally provided an effective length for girders of 0·5–0·7 of the effective span (i.e. the distance between bearing stiffeners, or centroid of the bearing pressure zone, at each end of a bridge girder). In contrast, the simple assessment method in NR/GN/CIV/025, applied strictly, would suggest that a number of these U-frame types were invalid and the girders would have an effective length equal to the effective span, giving a much lower assessed capacity. A recent assessment of this bridge was undertaken on that basis and gave an assessed capacity of RA1 at 80 miles/h (129 km/h; less than the normal required capacity for this section of the railway line of RA8 at 80 miles/h (129 km/h).

Further assessment required a more detailed consideration of the U-frame connection type, condition and capacity. Although the NR/GN/CIV/025 assessment standard provides a useful list of particular U-frame connection types not usually used in modern construction and their flexibility, the U-frame connection on this bridge is not fully covered. The U-frame type included in NR/GN/CIV/025 that is closest to that on the bridge is shown in Figure 6, comprising a single fabricated trough with vertical webs between web stiffeners, with the main girder web stiffener at the trough valley location.

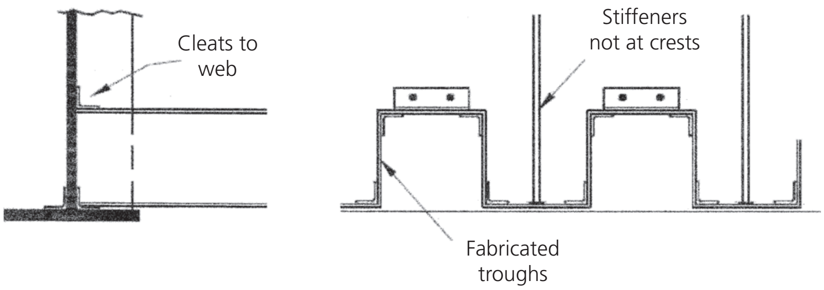

Figure 6. Trough with stiffeners, U-frame type

Consideration of whether this U-frame type can be assumed for an assessment (noting that no dimensional limits are given) raises a number of uncertainties such as the following:

(*a*) How similar does the trough geometry have to be to meet the specific types in the assessment standard?
(*b*) How does the number of troughs between web stiffeners and web thickness affect U-frame flexibility?
(*c*) Does the presence of a discrete U frame at an unstiffened web reduce web buckling capacity?
(*d*) At what point do condition defects (e.g. corrosion) become critical for U-frame flexibility?
(*e*) What are the implicit assumptions for the flexibility of the particular U-frame types given in the assessment standard – are they based on test data, theoretical analysis or a combination of the two?

The only other alternative U-frame type that would be relevant, and in this case conservative, is the type where a web with no stiffeners is assumed with continuous restraint provided by a trough.

A literature review on this subject showed a dearth of information, opinion or guidance other than that a discrete U frame should have girder web stiffeners at the U-frame connection and the general comment that, if this is not the case, then U-frame capacity and stiffness, and main girder stability, is adversely affected. However, a number of papers on non-linear FEA of bridges (e.g. Mehrkar-Asl *et al*., 2005) show that, in principle, FEA can be used to provide a more accurate assessment of capacity based on certain assumptions. Upstand grillage models (essentially acting as frames) have also been used for detailed analysis of U-frame type bridges but are not applicable in this particular instance, as the effect of local web flexure away from the girder web stiffeners would not be adequately modelled.

Disregarding the above uncertainties and measured out-of-tolerance and assuming the U-frame type in Figure 5 for the purposes of assessment, the assessed flexural capacity is RA7 at 80 miles/h (129 km/h; approximately 1700 kN m). Alternatively, if the conservative continuous U-frame type with no web stiffeners is assumed (Figure 7), the assessed flexural capacity is RA1 at 80 miles/h (or approximately 1200 kN m) due to the high flexibility of the assumed unstiffened web rendering the U frame almost completely ineffective.

The simple assessments based on no U frame and compliant U frame essentially give a range within which the assessed capacity actually lies. One method that is known to give potentially a reliable estimate within this range is to undertake FEA.

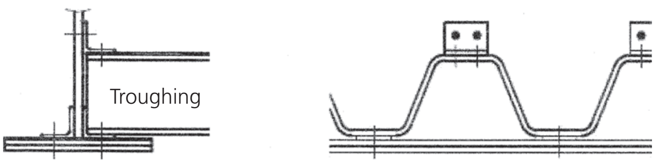

Figure 7. Trough without stiffeners, U-frame type

3.1 FEA of old U-frame bridges

FEA of old bridges has been used occasionally to provide a more reliable or improved estimate of structural capacity. One of the few examples that have limited applicability to this bridge is the study of Mehrkar-Asl *et al*. (2005), where non-linear analysis (geometric and material) was undertaken on a road bridge over the railway comprising riveted wrought iron girders with cross-girders and brickwork jack arches between. The only lateral restraint provided to the girders away from the supports was by the cross-girder to the main girder connection just above the bottom flange (noting that the girder web stiffeners were coincident with the cross-girders). The purpose of the assessment was to confirm the beneficial continuous lateral restraint effect of the deck between the girders and hence provide a higher assessed capacity for the girders. The general analysis method proposed in the paper was used as the basis for an FEA on the underbridge discussed in this paper with the following differences and comments noted.

(*a*) The edge girders are non-compact in accordance with NR/GN/CIV/025 due to compressive web depth; hence, non-linear material analysis (i.e. plastic analysis) was not used. However, it is noted that in some instances FEA can show that yield is acceptable in marginally non-compact girders.
(*b*) Wrought iron is known to have less ductility than steel, further supporting the decision to not allow any yield in the FEA.
(*c*) Discrete U frames exist (not at web stiffeners) rather than continuous restraint provided by a solid deck.
(*d*) Site measurements of the top flange bow and web verticality were used rather than assumed worst-case construction tolerance from TSO BD56/10 (2010).

For FEA of the proposed case study bridge, only a single edge girder is modelled using shell elements and boundary conditions applied to provide the relevant end restraint and U-frame restraint. The choice of boundary conditions is acknowledged in NR/GN/CIV/025 as a critical aspect if FEA is to be used, and they should be chosen with care to avoid unintended restraint and an overestimate of girder flexural capacity.

For the initial forced imperfection analysis and linear elastic analysis (no imperfections), the following boundary constraints were used:

(*a*) Vertical and lateral restraints were applied at the nodes at the bearing stiffener/bottom flange/web location and a single node either side, to both ends of the bridge. Longitudinal restraint to the same nodes was also applied at only one end of the bridge.
(*b*) There were no other U frame or other restraints.

Comparison of the simple assessment and FEA results for the initial analyses above (generally within 5%) provided confidence in the general reliability of the FEA and assumed boundary conditions. Reactions at vertical restraints were also checked for uplift. The FE model for this stage is shown in Figure 8.

For the non-linear geometric analysis, the same end restraints were used but the additional U-frame restraints were also included comprising the following:

(*a*) Imposed was a lateral displacement at the top and bottom trough fixings equal to that due to simply supported trough end rotation under RA10 at 80 miles/h (129 km/h) at ultimate limit state (hereafter called 'full load').
(*b*) Where the trough top fixings were missing or loose, the imposed lateral displacement was omitted.
(*c*) Vertical load was applied where the trough sits on the edge girder bottom flange. Due to the bridge skew, less load was applied at the acute corner, although the skew was sufficiently small that at all restraint positions the trough was continuous between the edge and centre girders (i.e. there was no need to omit or modify restraints for partial lengths of trough).
(*d*) Nodal reactions at the ends of the girder were checked to ensure that no unintended uplift resistance occurred.

Extensive consideration was given to item (*a*), as this provides the fundamental additional U-frame restraint and also forms an implicit assumption in the simple assessment methods. An alternative approach would be to use lateral springs, although calculation of a spring stiffness would require further assumptions to be made regarding the angle cleat stiffness (bearing in mind uneven fitting and occasional welds to some angle cleats). An alternative approach would be to model the transverse trough directly in addition to the edge girder, although assumptions would still need to be made regarding trough–girder connectivity. In this particular instance, it was concluded that a more detailed analysis would not be justified by the level of detail available. A simply supported assumption for the trough end rotation was considered appropriate, as it is generally the case that open cross-section main girders with low torsional stiffness provide little moment fixity at the ends of the trough and the girder local web flexibility would further reduce the marginal moment fixity. The proposed finite-element model for this stage is shown in Figure 9.

A non-linear geometric analysis was also undertaken on the edge girder without any U-frame restraints to understand whether when fully loaded

(*a*) the girder alone would tend to torsionally rotate more than that due to simply-supported trough end rotation (hence the trough restrains the edge girder)
(*b*) or the girder alone would tend to torsionally rotate less than that due to simply supported trough end rotation (hence the trough increases instability of the edge girder).

From the analysis it was found that item (*a*) was valid and the trough would indeed tend to provide additional restraint to the edge girder. This was true both near the support (where minimum girder torsional rotation would occur) and at midspan (where the greatest girder torsional rotation would occur) for this bridge, although this would not necessarily be the case for other structures depending on the girder slenderness ratio and the trough (or cross-girder) stiffness and spacing.

Further assumptions also have to be made regarding the locations where load is imposed onto the edge girder from the trough, and, unfortunately, these are partly dependent on construction sequence. The load from the trough is transferred onto the edge girder at the top or bottom trough fixings or shared between them. Without some form of strain gauging or other intrusive measurement, it is not possible to confirm the load share between

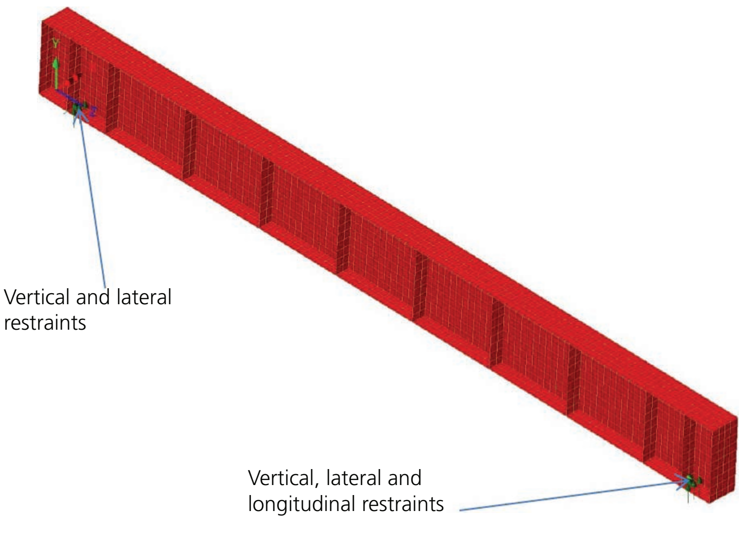

Figure 8. Finite-element model for initial linear elastic analysis

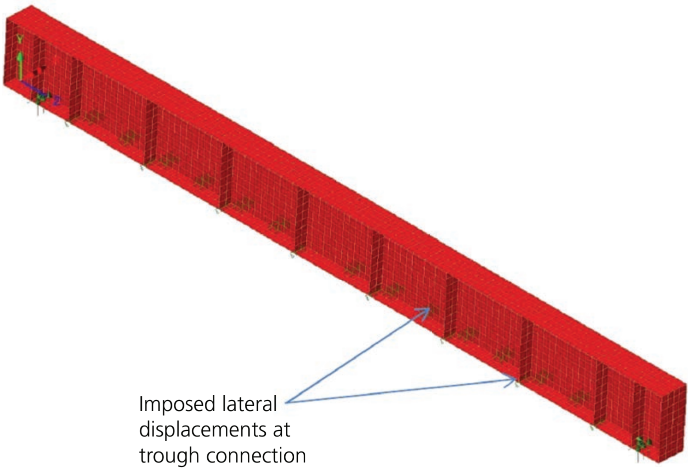

Figure 9. Finite-element model for non-linear geometric analysis

these fixings, and, therefore, a simple assumption was made that the trough load is shared equally between the top and bottom fixings. It could be assumed that the majority of the load is transferred at the trough bottom location, but the historical replacement of trough packing plates renders this uncertain.

There is an additional assumption regarding the load application to the trough bottom fixing which is not explicitly covered in NR/GN/CIV/025 or in BS 5400-3:2000. This is the position across the girder bottom flange width at which the load is applied. If no web stiffener was present at the flange, then local flange flexure would cause the reaction point to be at or close to the web/flange junction; however, the presence of the web stiffener means that negligible local flange flexure would occur and the reaction point would be nearer the edge of the trough-bearing zone. This assumption affects the analysis, as loads that are not on the same vertical plane as the girder shear centre will tend to cause additional torsional rotation and instability (this is similar but not as critical as the destabilising load effect in NR/GN/CIV/025 and BS 5400-3:2000 when load is applied to a compression flange with lateral freedom).

Also, although the measured top flange bow to the edge girders for this bridge was inwards towards the centre girder, it is noted that this may not necessarily be the case for other structures. An outward bow to the top flange of an edge girder would generally tend to reduce the stress in the top flange due to lateral torsional buckling but increase the restraint forces in the connection between the girder and trough or cross-girder.

It can be seen that, even with a relatively simple FEA, there are a large number of assumptions to be made that, taken together, could significantly affect the assessed capacity. However, one benefit is that for the FEA the assumptions are plain and clear (whether they are considered to be 'right' or 'wrong'), whereas if using engineering judgement to go beyond the strict limits of NR/GN/CIV/025, it is not necessarily clear what underlying assumptions may be contradicted.

The following comments are relevant for this particular bridge based on the assumptions actually used.

(*a*) Using a trough end rotation based on simply supported conditions for the trough is slightly conservative, as in reality a small amount of end fixity would exist.
(*b*) Spreading the trough load evenly on the edge girder bottom flange may be slightly unconservative, as in reality a greater proportion of the trough load could be present to the edge of the bottom flange.
(*c*) The majority of the trough load may actually be applied through the girder bottom flange rather than the trough upper fixing into the girder web (which has no eccentricity to the girder shear centre).

3.2 Analysis results and discussion

A summary of the assessed flexural capacity using the simple assessment method or FEA based on the previously stated assumptions is shown in Table 1. The non-linear geometric FEA with U frames modelled as stated in Section 3.1 gave a flexural capacity (RA5 at 80 miles/h (129 km/h) or approximately 1500 kN m) limited by lateral torsional buckling causing lateral bending in the top flange (Figure 10).

One particular ambiguity that makes direct comparison between the simple assessment and FEA difficult is the flexibility factor f used in NR/GN/CIV/025 and BS 5400-3:2000. This is an empirical factor that defines the flexibility of the U-frame connection, although it is not fully clear what effects are included within this factor for unusual or now obsolete U-frame types. An example of this is girder web flexibility due to U-frame connections between girder web stiffeners – the effect of web flexibility in this situation may or may not be included within the

Analysis type	Flexural capacity at 80 miles/h (129 km/h): kN m	Comment
Linear elastic	1790 (RA9 at 80 miles/h (129 km/h))	Imperfections and lateral torsional buckling ignored
Linear elastic buckling (eigenvalue)	1630 (RA7 at 80 miles/h (129 km/h))	Imperfections and yield stress limits ignored
025 best U-frame	1680 (RA7 at 80 miles/h (129 km/h))	Assumes rigid and strong U-frame connection
Non-linear FEA with U-frame	1480 (RA5 at 80 miles/h (129 km/h))	Imperfections and defects included (including missing U-frame rivet connections); local web flexure at U-frame fixings also critical
025 worst or no U-frame	1240 (RA1 at 80 miles/h (129 km/h))	Effective length equal to effective span

Table 1. Assessment results for different analyses and U-frame assumptions

flexibility factor, but a single flexibility factor cannot account for the significant differences in web flexibility due to the variation in web thickness, web stiffener spacing and U-frame connection spacing and location that often occurs on older bridges.

In addition to the critical flexural capacity based on lateral torsional buckling, shown in Table 1, web plate splice capacity was also critical (in particular web-bearing strength at riveted connections).

4. Strengthening proposals

To improve the flexural capacity of the edge girders a number of strengthening methods were considered:

(*a*) new steelwork internal web stiffeners at frequent trough peak locations
(*b*) new steelwork external web stiffeners at frequent trough peak locations
(*c*) replacement of rivets at inadequate web splices with pre-tensioned bolts
(*d*) new steelwork flange plates bolted to existing flanges
(*e*) top-hat steelwork universal column (UC) section bolted to edge girder top flange.

It is noted that all the considered strengthening options were such that, if assessed in the future, they would clearly meet assessment standards without the need for engineering judgement or complex

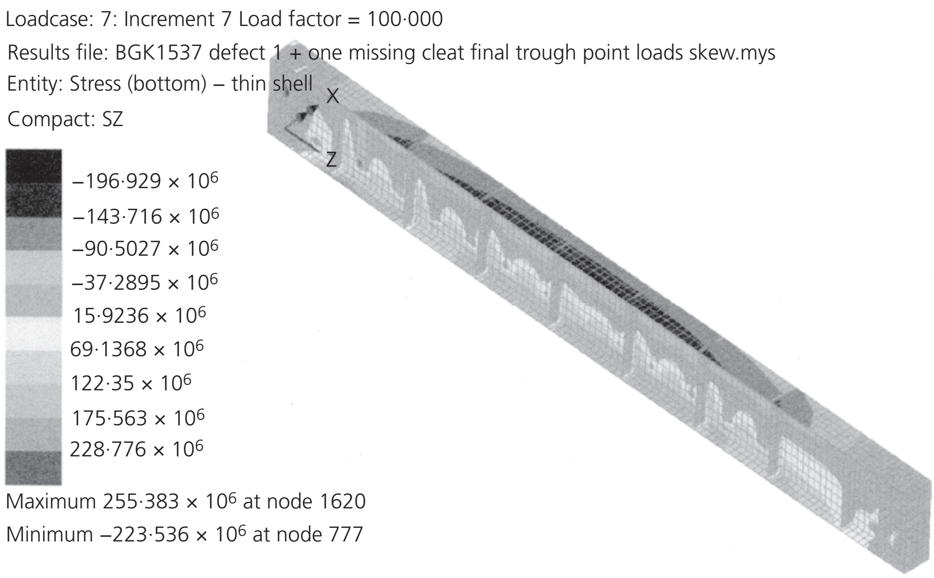

Figure 10. Longitudinal stress in edge girder (note stress concentrations to unstiffened web at trough connections)

analysis. In particular, local strengthening to the existing web stiffener locations was not considered appropriate as, even if FEA showed such a strengthening option to be adequate, it would not have met current assessment standards or necessarily be recognised by other future assessors as a valid strengthening measure.

The top-hat strengthening option had the benefit of being a single activity that dealt with all the regions with inadequate capacity. However, it would have required the greatest amount of temporary and rivet replacement and required the use of heavy plant for lifting of the UC sections.

New steelwork flange plates bolted to the full length of the edge girder top and bottom flanges (including packing plates and splice plates) would similarly require a large amount of rivet replacement and the use of plant for handling of heavy steelwork sections.

New internal steelwork web stiffeners to the edge girder required the least amount of new material, but for access would have required temporary removal of ballast plates, ballast and permanent way, at significant cost and disruption.

Therefore, the favoured option for improving flexural capacity was to use new external web stiffeners at trough peak locations. The amount of rivet replacement work for this option (combined with a small amount of rivet replacement at inadequate web splices) was less than half of that required for the top-hat solution, and the lightweight steelwork components can be manually handled with ease.

5. Alternative assessment methods

A number of alternative, or enhanced, assessment methods are available that were not used for this underbridge but may be appropriate for other similar bridges. These include the following.

(*a*) Non-linear material analysis combined with extensive metal testing to confirm reliably the plastic strain limit and ultimate tensile strength. This would be particularly useful for compact sections, but may also be useful for marginally non-compact sections where FEA can show a slightly less conservative local buckling limit for webs and flanges than that suggested by codified methods, or post-buckling capacity.

(*b*) Detailed FE modelling of the transverse trough as well as girders. This would take full advantage of any marginal moment fixity provided to the ends of the trough by the torsional stiffness of the girder. It would also allow consideration of additional fixity provided by the centre girder and other transverse trough. However, as stated previously, engineering judgement and assumptions would still need to be made regarding the exact stiffness and fixity of the trough–girder connections.

(*c*) Controlled testing and monitoring of the structure (or representative U frames in the laboratory). This approach is suggested as a possibility in NR/GN/CIV/025 for U-frame types that do not correlate with those explicitly shown in the assessment standard. Such testing could comprise monitoring of flange strains and lateral displacement under known loads, and perhaps additionally strain in the connections (or possibly direct load measurement using instrumented bolts). The information from this approach would also be very useful for realistic fatigue assessment of the U frame.

Item (*c*) in particular would be useful, as it would allow reliable design guidance to be provided for the stiffness and strength of U frames not strictly covered by NR/GN/CIV/025, together with the associated connection forces. The need for a detailed assessment or load testing and monitoring for other similar bridges would then be avoided.

6. Conclusions

A number of assessment codes and guides exist that take into account the particular details of now obsolete U-frame types and riveted wrought iron construction; however, a significant number of old bridges still have details that cause difficulty in reliable assessment or require very conservative assumptions, giving low assessed capacity. An example of this is old railway bridges with U-frame types that do not meet those suggested in BS 5400-3:2000 or NR/GN/CIV/025.

An alternative assessment method is to use FEA; however, this approach can also have limitations, as it is reliant on assumptions that may be difficult to confirm without extensive field monitoring and testing. The use of FEA can be useful in showing that girders that are marginally non-compact based on codified assessment methods may actually have sufficient local stability to provide additional flexural capacity beyond the elastic moment capacity.

In some instances where an assessment is unduly conservative, this may mean that needless measures such as strengthening or speed restrictions are instigated with associated cost and disruption to the operational railway and public.

It is suggested that a review of the more common, although now obsolete, U-frame types found on old railway bridges combined with field testing and monitoring would enable a significant number of bridges to be shown to have additional capacity than that based strictly on current assessment codes and guides. It is also suggested that this activity would enable the development of useful guidance on limiting factors for U frames that are non-code-compliant or with significant defects, such as loss of fixings or corrosion. This would also enable improved consistency in assessment methods.

It is hoped that this would enable, in turn, the ongoing maintenance and management of such historic structures to be optimised.

Disclaimer

The opinions expressed in this paper are solely those of the authors and not necessarily those of Jacobs or Network Rail.

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