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EVALUATION OF HALF-THROUGH BRIDGE LOAD-CARRYING CAPACITY BY USE OF NON-LINEAR ANALYSIS METHODS – CASE STUDIES

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Abstract- UK's existing railway network relies on many half-through bridges that were built during Victorian era and are already beyond their designated service life. Therefore, maintaining bridge assets in serviceable condition to ensure sustainable replacement rate without major network disruptions is vital. Current UK practice applies the British Standards and Network Rail's assessment codes to quantify the load capacity of such bridges using hand methods. If the codified methods rate the bridge capacity as substandard more advanced analysis (based on finite elements) is usually commissioned in an attempt to improve the capacity. In this advanced analysis the structure is modelled with 3D shell elements and the load group rating is extracted from a non-linear buckling and plastic analysis. Currently, there is not enough formal guidance regarding the pre-processing and more importantly the post-processing techniques of a FEA based assessment and quite often the knowledge and experience is passed over solely from more experienced individuals. This paper presents two case studies of real half-through bridges that demonstrate how additional FE model refinement, mainly through convergence enhancements, could reveal essential information about bridge behavior at or close to collapse load. Based on findings from the case studies, a set of generic recommendations is produced to inform both pre- and post-processing aspects of future assessments where bridge capacity is extrapolated directly from FE model results. The main aim of this paper to add more clarity in the interpretation of Nonlinear Analysis results. This can lead to more appropriate maintenance or strengthening recommendations and savings in the budget.

Keywords- Half-through, Finite element model, Non-linear analysis, Network rail.

1 INTRODUCTION

Bridges play a vital role in supporting railway infrastructure in the UK. Out of 30,000 bridges owned by Network Rail (NR), one third of them are of metallic composition such as steel, wrought iron and cast iron [1]. One of the most common configurations for metallic bridges encountered on railway is the 'half-through' type shown on Figure 1 (a) [2]. This form of construction is preferred on railway sites over others for its distinct positioning of trafficked surface with respect to the structural envelope. As opposed to other deck types, the trafficked surface lies within construction depth of a deck which allows to partially accommodate track components and traffic within deck boundaries, resulting in greater clearances beneath a bridge. The drawback is that only U-frame action provides lateral restraint at the level of compression flange, making this deck type susceptible to torsional buckling. A large fraction of 'half-through' type bridges, constructed during Victorian era, have detailing which renders U-frame action partially ineffective. An example of such detail is a non-coincident position of cross girder with respect to web stiffener as shown on Figure 1 (b). With non-utilised U-frames, the spacing of lateral restraints, i.e. effective length, becomes equivalent to girder span. This results in a massive increase of compression flange slenderness and reduced bending capacity of a girder, as evident from Figure 1 (c). Moreover, many investigations [3] demonstrate that standard rules for girder web and stiffener capacities are unduly pessimistic. Unsurprisingly, many old bridges fail assessments undertaken by codified hand methods.

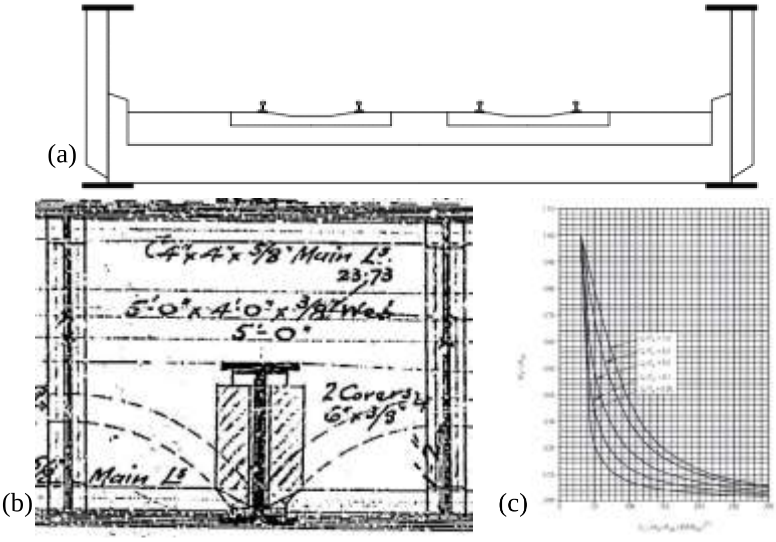


Figure 1: (a) Example of half-through construction [2] (b) Cross girder not coincident with web stiffeners in the bridge built circa 1900; (c) Chart indicating relationship between girder slenderness and limiting moment of resistance, MR [4]

For bridges that have been rated as sub-standard by codified hand assessment, NR usually commissions a more advanced assessment, utilising Finite Element Analysis (FEA) where girder capacities are extrapolated directly from a FE model (primary shell). Some bridges with peculiar geometric features, e.g. fish-bellied geometry of girders, high skew, slanted girders on plan, bypass codified hand assessment if asset owner deems that hand methods are unsuitable. FEA assessments rate individual girder elements against buckling and yielding. The governing buckling modes in U-frame bridges are lateral torsional buckling to the compressive flange and shear buckling of web plates. Web plates are normally accompanied by transverse stiffeners to enhance web's post-buckling capacity due to tension field action [5]. Buckling of plate structures is characterised by biaxial bending due to out-of-plane deflection of the elements. In contrast with axially loaded columns where buckling limits their ability to support more than the critical axial load, plates under compression will continue to support higher axial forces by utilising membrane action and they will fail in loads significantly higher than the theoretical critical load [6]. Additionally, in the design of new bridges, it is common to first size the structure for strength and stiffness and then carry out SLS and ductility checks over the remaining capacity, whereas target deflections and fatigue criteria can be used as a benchmark in determining reserve capacity of existing bridges. Similar to seismic or other complex designs some degree of plastic structural response can be expected when assessing old bridges under service loading. This relies on the inherent ductile properties and slenderness of steel and wrought iron girders that allow for internal redistribution effects [10,11]. However, plastic deformations are generally avoided in the assessment of existing old bridges because it is difficult, in practise, to define a point at the load-displacement curve where the reduction of the load-carrying capacity can be deemed as acceptable.

Many Victorian era bridges are well beyond their design life yet continue to demonstrate resilience against NR's performance metrics for safety and reliability. Numerous of these bridges are in poor condition with highest proportion (~33%) scattered over Scotland route [7]. Condition of bridges will continue to deteriorate and, eventually, replacements will need to be commissioned. NR strategic plans for Control Period 6 [8] highlight how unsustainable the current rates of remediation are and how it will have a knock-off effect on railway network capability in the future. For example, with Wessex route, at the present replacement rate, it is projected that some of the bridge assets will be 300 years old before they can be replaced [8]. This suggests that more influx of funding will be required to either increase the replacement rate or maintain old bridges in serviceable condition. In order to prioritise bridge replacements in a sustainable manner, improvement in the assessment reporting is required to ensure that replacement scheme appraisals are not driven by inconclusive results.

Guidance on conducting non-linear assessments of NR bridges primarily lays emphasis on pre-processing aspects of FEA bridge modelling. A research gap has been identified in the post processing requirements which, in UK practice, are limited to examples of load-displacements charts and screenshots of yield labels on stress contours at the locations of interest. With the aid of two real bridge examples, in order to address the research gap this paper demonstrates that by blindly executing the assessments to the guidance requirements may lead to erroneous conclusions. The aim of this paper is to add more clarity to the interpretation of post-processing NL analysis results and to highlight the significance of buckling displacement history in extrapolating bridge load carrying capacity.

2 METHODOLOGY

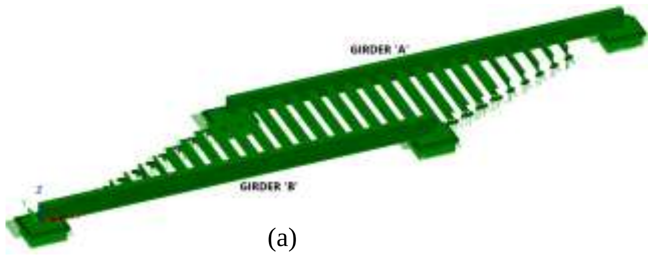
2.1 General formulation of bridge models for analysis

3D thick shell FE LUSAS v17.0 models of two real half-through bridges, Grand Union Canal and Battersby Lane, are used to investigate buckling behaviour of web panels. Shell thickness of surfaces in both models accounts for corrosion section losses that have been rationalised from inspection reports. A non-linear buckling analysis is undertaken in two steps, incorporating both geometric and material non-linear characteristics described in Section 2.2 and 2.3 respectively. First analysis step includes application of permanent actions as a single load increment. In the second step, it is intended to inherit residual stresses and deformation contour from first step and apply rail traffic at 0.05 load increments. A 0.05 increment represents each load group number in RA (route availability) 0-15 range, capturing all locomotive classes that are used in the UK. Articulation is assigned to suit bearing conditions of each bridge (bottom flange bearings seated directly on bedstones) – simply supported arrangement with restraints applied at centre line of bearing stiffener. As the selected bridges experience convergence issues early into the non-linear analysis, model refining is undertaken to alleviate stress concentrations in the bearing zones. Slide surfaces are used in Grand Union Canal Bridge to idealise contact between bedstones and bottom flanges and lift-off support capability is incorporated in Battersby Lane Bridge. Table 1 and Figure 2 below showcases the LUSAS shell models and the key characteristics of the bridges that are analysed.

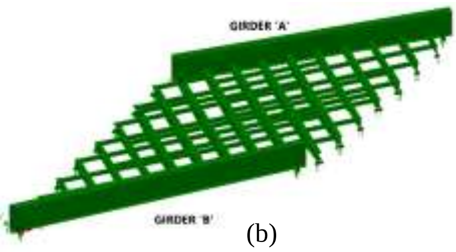
Table 1: Key characteristics of bridges analysed

Bridge Name	**Bridge Span (m)**	**Bridge Width (m)**	**Girder Depth (m)**	**Skew**	**Material**	**U-frame notes**
Grand Union Canal	27.3	6.8	1.40	67°	Wrought Iron	Partial compression flange restraint provided by unstiffened moment connection between transverse and main girders. Transverse beams are not coincident with stiffeners.
Battersby Lane	22.0	12.0* (average)	2.25	50°	Wrought Iron	Compression flange restraint provided by stiffened moment connection between transverse and main girders. Transverse beams and stiffeners form an effective U-frame.

*Width varies along the bridge due to slanted alignment of girders on plan



(a)



(b)

Figure 2: (a) Grand Union Canal LUSAS Shell Model; (b) Battersby Lane LUSAS Shell Model. Floor deck excluded for clarity

2.2 Prescribed initial perturbations

Non-linear buckling analysis is undertaken with initial deformations generated by web buckling modes from elastic eigenvalue analysis. Magnitudes for displacements are be derived in accordance with Table 8 in BS 5400-6 [12]. Displacement path has significant weight on how RA load group number for capacity is derived and it is undesirable to introduce excessive initial deformation especially if it has not been observed on site. The scale of imperfection only needs to be enough for triggering anticipated buckling behaviour.

2.3 Materials

The plasticity models that are more suitable for ductile materials which exhibit little volumetric strain such as isotropic metals are typically that of von Mises and Tresca. Both can appear side by side with little or no difference depending on complexity. Calibration of both criteria with respect to tensile (or compressive) strength and shear strength has shown that the maximum difference between the two is approximately 15% which, compared to assessment or design safety factors is relatively small. Von Mises plasticity criterion is selected for model material attribute which in contrast to the more conservative Tresca (lower limit) is considered as more consistent to experimental data and more widely accepted for civil engineering applications [9]. Elastic-Perfectly-Plastic material is also assumed. A strain hardening gradient is assigned in the plastic material properties to further refine the model and improve convergence. A plastic strain limit is set at 0.15 (15% elongation) according to NR/GN/CIV/025 which applies to wrought iron materials that have not been tested. The slope (E2) of the hardening gradient has been calculated using equation 1 as follows:

$$E2 = ((\sigma_u - \sigma_y) / (\delta_2 - \delta_2) \qquad (1)$$

Where: E= Modulus of Elasticity; σ_u *= ultimate tensile strength;* σ_y *= minimum yield strength;* $\delta_1 = \sigma_y/E$*;* $\delta_2 = 0.15$

2.4 Post processing and interpretation of results

Instead of following the scope prescribed in the Level 2 assessment guidance document which indicates RA0-15 (14t-31t) axle load range for non-linear analysis, the FE models are refined to improve convergence and run extended non-linear analysis with more increment steps with loading beyond RA0-15 range. The intent is to investigate whether there is any benefit of overloading a bridge deck to extract a more informative post-buckling displacement history graph.

3 RESULTS AND DISCUSSION

3.1 Grand Union Canal Bridge – sensitivity of buckling response to articulation

As described in section 2, a non-linear finite element model is developed for Grand Union Canal bridge. This case illustrates how the use of non-linear boundary conditions have allowed to run a more extensive analysis and evaluate behaviour from larger pool of results. Figure 3 (a) shows the load-displacement curve at the critical web panel using basic linear support conditions. Due to peak stress concentrations at the nodes restrained by simple supports, convergence issues aborted the analysis early and engineer rated the bridge as substandard (RA0; <14t per axle) governed by buckling of web panel adjacent to obtuse corner bearing. This was followed by recommendations for immediate speed and load restrictions across the structure. Evaluation of the boundary conditions using NL slide line supports with contact surfaces to distribute the load over a more 'realistic' area enhanced solvability of the model at the latter load increments.

Figure 3 (b) shows how the displacement history of buckling response for the same element was extended to capture the non-linear behaviour up to factor of 3.0x. Moreover, as snap-through buckling behaviour is not identified in the refined analysis, it highlights how buckling response is sensitive to articulation attributes. As a result, sufficient displacement range allowed to extrapolate conclusive load group for buckling (RA6; 20t-22t per axle, a 42%-57% increase).

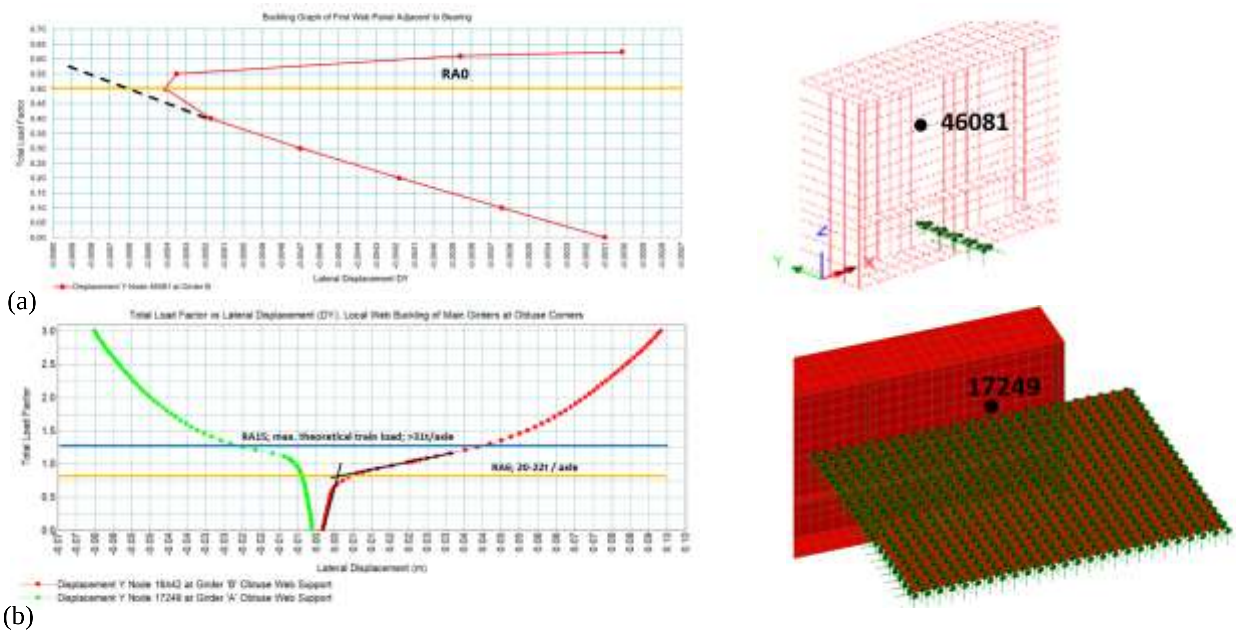


Figure 3: Web buckling behaviour at obtuse corners of the girders. (a) Simple support conditions; (b) Slideline surfaces.

3.2 Battersby Lane Bridge – a case with two local buckling modes with inherently different behaviours

This case demonstrates two local buckling modes obtained from web panels at the opposite obtuse corners of Battersby Lane Bridge. Figure 4 shows displacements paths against load increments for two non-linear analyses undertaken for main girder webs. The analyses have been prescribed with initial perturbations with maximum displacement applied to node 15274 which has been identified by elastic eigenvalue buckling analysis. The only difference between the two analyses sets is the extent of displacement history. Figure 4 (a) graphs cover displacements up to 1.25 load factor which is equivalent to RA15, whilst Figure 4 (b) proceeds beyond load factor of 2.0 to capture displacements from higher load increments.

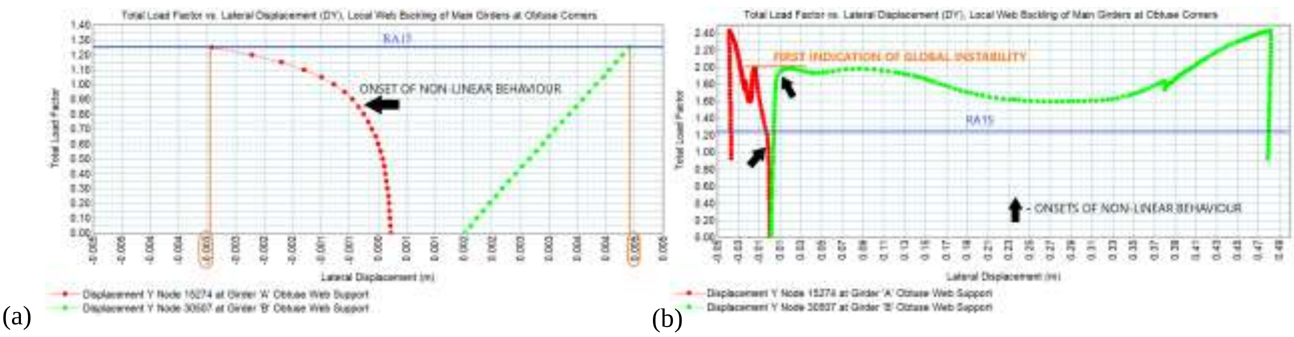


Figure 4: (a) Non-linear buckling analysis 1, last load increment at factor 1.25 x RA1 live load; (b) Non-linear buckling analysis 2, highest load increment peaks at ~2.4 x RA1 live load.

From Figure 5 (a) it is clear that commencement of non-linear behaviour for node 15274 occurs within 0.7-0.9 load factor range. The displacement path peaks at 3mm at the last load increment (1.25x) and signals the development of gradient plateauing. In contrast, node 30507 shows no signs of non-linear behaviour within 0-1.25 load factor range yet its displacement peaks at 5mm. However, Figure 4 (b) illustrates that curve flattening at node 15274 is within narrow displacement range if compared to maximum displacement at node 30507. It is also evident that buckling at node 15274 does not evolve into collapse load and does not show any signs of instability before sudden flattening at node 30507. This suggests that buckling at node 30507 triggers global instability and earlier commencement of non-linear path at node 15274 does not warrant reporting of bridge failure as highlighted by Ryjacek, 2019. This conclusion is reinforced by observing non-amplified displacement contours shown on Figure 5 that shows how buckling at node 30507 (Figure 5 (a)) evolves into plastic collapse of the girder, whilst displacement at node 15274 (Figure 5 (b)) remains small with respect to adjacent elements.

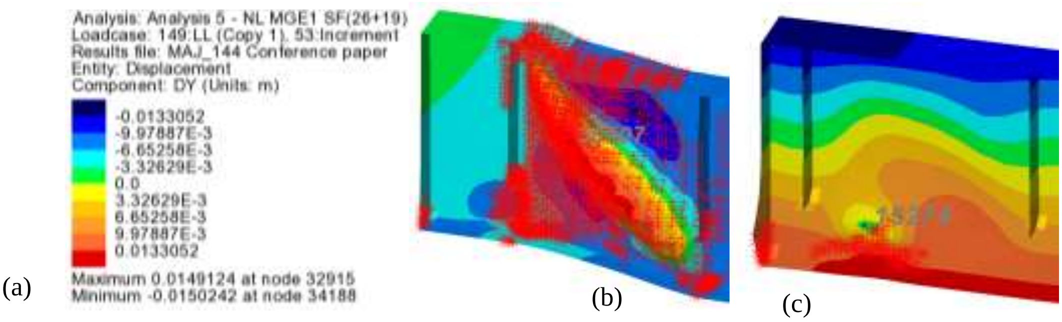


Figure 5: Displacement contours at buckled panels at load increment 2.0 x RA10 railway live load (non-amplified). Red crosses indicate yielded mesh. (a) Contour legend; (b) Node 30507, buckling at obtuse end of Girder 'B', signifying tension field action; (c) Node 15274, localised buckling at obtuse end of Girder 'A'.

4 CONCLUSION AND RECOMMENDATIONS

Non-linear buckling analysis results for web panels of two bridges have been presented in this study. Following conclusions are drawn from this study:

- Through additional refinement of models to enhance solvability, it has been revealed that interpretation of results could vary depending on the quantity of data, primarily displacement history. With the bridges analysed, inadequate displacement range has led to erroneous capacity reporting.
- It has also been demonstrated how buckling response is sensitive to idealised boundary conditions and that further refinement of articulation can eliminate 'dummy' buckling modes. In addition, as demonstrated by review

of Battersby Lane Bridge results, eigenvalue buckling analysis is not always a reliable predictor of critical buckling modes.
- Not all these matters are considered in the guidance note for undertaking advanced Level 2 assessments which has led to reporting of overly conservative results

To avoid potential erroneous reporting in the future, the following recommendations are made:

- The minimum threshold limits for buckling displacement history should be defined in the guidance document. From case studies presented in this paper, it is clear that 5mm displacement range is too narrow to make an informed judgement. Through inspection of graphs on Figure 3 (b) and 4 (b), 10mm lateral displacement range for local buckling of web should be sufficient to provide a plot from which load group against web buckling failure could be determined. Displacement range for flanges requires a separate study as flanges can undergo much larger lateral deformations before stability of the whole bridge is compromised.
- For complex bridges, provisions for further sensitivity analyses to resolve convergence issues or improve results should be discussed and be considered in advance. Ideally additional budget will be included during the bidding stages solely for that. Bridges with large spans and excessive skew may take hours to complete one single analysis which has resulted in engineers taking shortcuts in order to complete the work within budget.
- The scope of assessment should incorporate a more in-depth discussion of appropriate remediation measures for the failure mode under consideration. For example, it should be outlined which intervention strategy, such as planned preventative, do nothing, aggressive monitoring etc., best fits the behaviour of governing element. Having visibility of post-buckling reserve capacity would help to identify the urgency of strengthening or maintenance measures.

5. ACKNOWLEDGMENT

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6. REFERENCES

[1] Network Rail, "NR Bridge List.xlsx," 2015.
[2] D. C. Iles, "Design Guide for Steel Railway Bridges," Ascot, The Steel Construction Institute, 2004, pp. 13-43.
[3] C. R. Hendy and F. Presta, "Transverse web stiffeners and shear moment interaction for steel plate girder bridges," in Portugal, Guimarães, 2008.
[4] British Standard Institution, "BS 5400-3:2000. Steel, concrete and composite bridges - Part 3: Code of practice for design of steel bridges," London, 2000.
[5] SCI, "Design Guide for Steel Railway Structures," Ascot, Berkshire, The Steel Construction Institute, 2004, p. 164.
[6] ICE, "ICE Manual of Bridge Engineering (Second Edition)," London, Thomas Telford Ltd, 2008, p. 735.
[7] Network Rail Scotland Route, "Scotland Route Strategic Plan," Network Rail, 2019, pp. 89-90.
[8] Network Rail Wessex Route, "Wessex Route Strategic Plan | 2019 to 2027," Network Rail, 2019, p. 111.
[9] N. E. Dowling, Mechanical Failure of Materials 4th Edition, Blacksbury, Virginia : Pearson, 2012.
[10] W.-F. Chen and L. Duan, "Bridge Engineering Handbook Second Edition (Seismic Design)," Boca Raton, Florida , Taylor & Francis Group , 2014, p. 722.
[11] R. Karoumi, J. R. Casas, M. Plos, C. Cremona, C. melbourne and J. S. Jensen, "Guideline For Load and Resistance Assessment Of Existing European Railway Bridges," 4th International Conference on Bridge Maintenance, Seoul S. Korea, 2008.
[12] British Standard Institution, "BS 5400-6:2000. Steel, concrete and composite bridges - Part 6: Specification for materials and workmanship, steel;," London, 2002.
[13] P. Ryjacek, The diagnostic techniques for the assessment of the historical steel bridges, *IABSE Symposium, Guimaraes 2019: Towards a Resilient Built Environment Risk and Asset Management – Report*, pp. 1651-1657.