Cite this article

Imam B, Salter PA and Hayward ACG (2019)

Discussion: Historical load effects on fatigue of metallic railway bridges.

Proceedings of the Institution of Civil Engineers – Bridge Engineering **172(1)**: 65–69,
<https://doi.org/10.1680/jbren.18.00024>

Discussion

Paper 1800024

Published online 30/11/2018

ICE Publishing: All rights reserved

Discussion: Historical load effects on fatigue of metallic railway bridges

Boulent Imam BSc, MSc, PhD

Senior Lecturer, Department of Civil and Environmental Engineering,
University of Surrey, Guildford, UK

Paul A. Salter BEng, MSc

Engineer, Ramboll UK Ltd, Southampton, UK

Alan C. G. Hayward FREng, CEng, FICE, FStructE

Consultant, Cass Hayward LLP, Chepstow, UK (corresponding author:
ahayward123@btinternet.com)

Contribution by Alan C. G. Hayward

The paper by Imam and Salter (2018) is very useful in quantifying historic train loading. The discussor has several comments that, hopefully, will be taken as constructive.

1. Dynamic factor

According to Equation 1 in the original article (Imam and Salter, 2018), the authors quote the dynamic factor (Daf) for fatigue as

$$3. \quad \text{Daf} = 1 + 0.5 \left(\phi_1 + \frac{\phi_{11}}{2} \right)$$

where ϕ_1 is the dynamic increment representing inertial response of the structure and ϕ_{11} is the dynamic increment representing track irregularities.

It appears that the dynamic effect of hammerblow for steam locomotives has not been allowed for. However, according to the Bridge Stress Committee (BSC) report of 1928 (DSIR, 1928), the dominant dynamic effect from steam locomotives was hammerblow, resulting from an extensive bridge-testing programme. The BSC report introduced what became known as RB loading, based on the static RA1 load train enhanced by the measured dynamic factors. RB loading was used for bridge design until British Railways adopted UIC 71 loading in 1973 with the elimination of steam traction, appearing as RU loading in BS 5400 (BSI, 1978). The BSC report included the impact effects from rail joints but did not comprehensively cover track quality.

A dynamic factor for fatigue to cover steam locomotives and allowing for track irregularity might be taken as approximately

$$4. \quad \text{Daf} = \left(\frac{\text{RB}}{20 \text{ RA1}} \right) + \phi_{11}/4$$

where the value of RB loading corresponds to the train speed considered. This would be approximate and does not take account of the wheel arrangements and hammerblow characteristics of the various locomotives considered by the authors.

From published data (including the BSC report), the discussor has estimated hammerblow forces for each locomotive type considered by the authors, at the appropriate train speed. The values are averaged from the many different locomotive classes owned by railway companies. The hammerblow values presented in Table 5 are for

- a whole locomotive – relevant for main girders
- per axle – relevant for cross-girders
- per wheel – relevant for railbearers and spans <3 m.

The number of successive hammerblows from the wheels (N) within a span depends on the relative driving wheel circumference (πd), the coupled wheelbase (w) and the bridge span (L), and can be taken as

$$5. \quad N = \left\lceil \frac{(L-w)}{\pi d} \right\rceil + 1$$

Magnified hammerblow from resonance with a bridge occurs if the rotational speed of the driving wheels (n_L) is equal to or greater than the natural frequency (n_0) of the bridge member (Figure 17). It is modified by bridge damping. Magnified hammerblow generally occurs in spans of 11 m or more. The magnification factor, m , can be expressed as

$$6. \quad m = \alpha \left(\frac{n_0}{n_L} \right)^2$$

where α defines the magnification of hammerblow caused by the number of blows (N), as modified by damping, but with n_0/n_L taken as not greater than 1.

Values for the log decrement of damping can be taken from dynamic testing on the West Coast Main Line (Mott MacDonald, 2001) and published data (Fryba, 1996). A value of 0.08 is typical for steel bridges with a span of 20 m or more, and 0.11 for 11 m spans. The effects on the bridge can be estimated conservatively by assuming that the driving wheels are spinning while stationary at the mid-span of the bridge, as has been justified by Inglis (1932).

Table 5. Typical hammerblow values

Locomotive type	Driving wheel diameter, d : ft-in. (m)	Speed: mph (km/h)	Rotational speed, n_L : rps	Number of driving axles	Axle load: t	Hammerblow: t		
						Locomotive	Axle	Wheel
0-6-0	5-3 (1.60)	30 (48.3)	2.7	3	18.0	1.9	1.8	1.0
4-4-0	6-9 (2.06)	50 (80.5)	3.5	2	18.3	3.5	2.6	2.1
0-4-4T	5-6 (1.68)	30 (48.3)	2.5	2	17.2	1.2	1.1	0.8
2-8-0	4-8 (1.42)	40 (64.4)	4.0	4	17.2	3.8	3.3	2.5
4-6-0	6-0 (1.83)	70 (112.7)	5.5	3	19.7	8.2	4.2	3.5

Figure 17. Schematic illustration of hammerblow

Table 6 shows the number of blows (N) and magnification factor (α) for each locomotive type considered by the authors. The 0-6-0, 4-4-0 and 0-4-4T types do not produce magnified hammerblow at the assumed speeds because n_0/n_L is greater than 1, even for low-frequency bridges.

From the above, it should be possible to produce a fatigue loading history that takes into account hammerblow from steam locomotives to compare with the authors' assumptions.

2. Historic traffic

In covering the period 1900–2010, the paper excludes most wrought-iron bridges. The discussor asks the authors why they did not include nineteenth century bridges in their investigation.

The authors state that an assumption of modern traffic representing historical loading is over-conservative. In a study conducted in the USA, Tobias and Foutch (1995) arbitrarily assumed that 50% of the life of riveted bridges had been used up in estimating the residual life. The assessment document NR/GN/CIV/025 (NR, 2006) allows appropriate historic information to be used, if available. The ideal would be reliable historic data from working timetables. For routes from coal-mining areas such as the Midland Main Line from Nottinghamshire to London, the traffic type has now changed completely. Figure 18 is merely illustrative, but indicates a

contrast in fatigue life between historic 10 t and modern 100 t bogie freight vehicles in carrying the same annual tonnage.

3. Bridge arrangement

The authors show half-through riveted plate girders with transverse troughing. Historically, cracking has occurred at trough ends where connected to main girders (e.g. see Berridge, 1963), but trough floors have been found to be otherwise efficient and not prone to fatigue. The discussor asks whether the authors could explain the choice of arrangement.

A railbearer/cross-girder arrangement is prone to fatigue arising from direct wheel loading, short spans and uncertain fixity of connections, as studied by Imam *et al.* (2006) (Figure 19). Although there have been few fatigue failures with riveted bridges, any defects that do occur tend to occur at connections; for example, cracking of re-entrant notches to railbearer webs. Assessments of riveted girder web/flange connections frequently indicate limited capacity, yet rivet 'give' tends to relieve the calculated horizontal shear – an attribute of riveting.

Welded construction, although efficient, is more susceptible to fatigue. For example, problems have occurred in a number of the former standard Western Region box girder bridges from the 1960s. This standard was modified in 1989 to eliminate stress concentrations. Cracking has also occurred in half-through bridges due to unintended floor interaction from the rigidity of welded connections.

4. Fatigue in bridges

The authors mention that fatigue performance only became a requirement since the 1970s. However, fatigue was included in BS 153-3-4-5:1923 (BSI, 1923) under 'alternating stresses', by the provision of an extra section area. Similar criteria appeared in BS 153-3-4-5:1937 (BSI, 1937) in 1937 and the 1949 IStructE code (ICE and IStructE, 1949). Later editions of BS 153 in 1958 and 1972 (BSI, 1958, 1972) progressively introduced fatigue for railway bridges (Gurney, 1963; Gurney *et al.*, 1977).

By this time, welding had replaced riveted fabrication, so rules were more specific to welded bridges (BRB, 1968). British

Table 6. Typical values of N and α for up to 20 m spans

Locomotive type	Rotational speed, n_L : rps	Driving wheelbase, w : m	Minimum span for magnified hammerblow: m	Maximum N in 20 m span	α
0-6-0	2.7	5.0	NA	4	1.00
4-4-0	3.5	2.9	NA	3	1.00
0-4-4T	2.5	2.4	NA	4	1.00
2-8-0	4.0	5.3	19	4	3.55
4-6-0	5.5	4.5	16	3	3.55

Figure 18. Fatigue for coal trains – historic and recent

Railways produced new fatigue loading with the appearance of heavier freight vehicles and the end of steam traction (Spindel *et al.*, 1975).

Figure 19. Cross-sections of riveted bridges

BS 5400: part 10 (BSI, 1980) used the latest fatigue research and introduced load spectra of light, medium and heavy traffic. For riveted bridges, studies continued (Beagles, 1993; Xie *et al.*, 2001), including tests on bridges. For example, strain gauge testing was carried out on the River Severn Bridge, UK, under two ‘Castle’ class 4-6-0 locomotives (BR, 1956) (Figure 20). Alternating stresses in the truss diagonals arising from cross-girder–truss interaction led to the decision to strengthen all 22 spans in 1960/1961, based entirely on fatigue life considerations.

For assessments since 2006, NR/GN/CIV/025 (NR, 2006) covers fatigue requirements, drafted by the discussor, based on BS 5400: part 10 (BSI, 1980).

Figure 20. 1956 testing of the River Severn Bridge, UK

The symbol ϕ , which appeared in the original paper and which is retained in this discussion, should strictly be φ . ϕ is normally taken to mean the dynamic factor in BS EN 1991-2 and other published documents.

Authors' reply

The authors would like to thank the discussor for his constructive comments and the additional data provided.

Due to the unavailability of such data on the hammerblow effect of steam locomotives while writing the paper, the authors were not able to consider this within the calculations. With the data provided in Table 6, it is possible to predict dynamic effects of steam locomotives and quantify their effects on fatigue damage accumulation.

The historical load model that the authors developed in collaboration with Network Rail included trains that started running on the railway network roughly from the 1900s onwards; thus, the authors did not include any bridges constructed before that time.

The authors agree that the availability of historical train data from timetables would lead to a more reliable assessment of fatigue damage accumulation. The location of a bridge structure within the railway network significantly affects the type of train loading it has predominantly felt over time; for example, freight compared with passenger trains. Indeed, the models proposed in the paper are indicative and represent 'typical' conditions. However, there are bridges in the network that may have experienced completely different train traffic from what is suggested by the proposed traffic models.

It was out of the scope of the authors' study to compare fatigue damage across different types of bridges, so the choice

was made to represent a typical bridge configuration. It would be an interesting study to compare relative fatigue damage across a range of typical riveted bridge configurations, as this will shed light into which are more susceptible to higher fatigue damage accumulation.

The authors agree with the discussor that assessing the effects of uncertain fixity or secondary effects is quite challenging through current assessment procedures and may often require the use of more advanced modelling.

It is true that welded bridge construction has shown greater likelihood of fatigue damage than riveted railway bridges, which appear to have been coping satisfactorily with current loading demands.

Finally, the authors thank the contributor for the inclusion of the additional useful information on the development of fatigue standards and codes.

REFERENCES

- Beagles M (1993) *Static and Fatigue Properties of Wrought Irons and Early Steels*. British Railways Board, London, UK. British Rail Research Report LR MF 115.
- Berridge PSA (1963) Factors governing the choice between repairing, strengthening, and reconstructing railway girder bridges. *Proceedings of the Institution of Civil Engineers* **26**(1): 1–30, <https://doi.org/10.1680/iicep.1963.10452>.
- BR (British Railways) (1956) Stresses in bridges. Tests made on the River Severn Bridge at Sharpness, Gloucestershire. In *Proceedings of the British Association Meeting, Sheffield, UK*. BR, Derby, UK.
- BRB (British Railways Board) (1968) *Design of Steel Bridges*. BRB, London, UK, Technical note no. 15.
- BSI (1923) BS 153-3-4-5:1923: Specification for girder bridges: 3, loads and stresses; 4, details of construction; 5, erection. BSI, London, UK.
- BSI (1937) BS 153-3-4-5:1937: Specification for steel girder bridges: 3, loads stresses; 4, details of construction; 5, erection. BSI, London, UK.
- BSI (1958) BS 153-3B-4:1958: Specification for steel girder bridges: 3B, stresses; 4, design and construction. BSI, London, UK.
- BSI (1972) BS 153-3B-4:1972: Specification for steel girder bridges. Stresses. Design and construction. BSI, London, UK.
- BSI (1978) BS 5400: Steel, concrete and composite bridges – part 2: specification for loads. BSI, London, UK.
- BSI (1980) BS 5400: Steel, concrete and composite bridges – part 10: code of practice for fatigue. BSI, London, UK.
- DSIR (Department of Scientific and Industrial Research) (1928) *Report of the Bridge Stress Committee*. HMSO, London, UK.
- Fryba L (1996) *Dynamics of Railway Bridges*. Thomas Telford, London, UK.
- Gurney TR (1963) The basis of the revised fatigue clause for BS 153. *Proceedings of the Institution of Civil Engineers* **24**(4): 519–540, <https://doi.org/10.1680/iicep.1963.10703>.
- Gurney TR, Boutle NF, Spindel JE and Burdon ES (1977) Informal discussion. Implications of the proposed revision of the fatigue clauses in BS 153 on future design of structures and railway vehicles. *Proceedings of the Institution of Civil Engineers* **62**(5): 317–319, <https://doi.org/10.1680/iicep.1977.3234>.

-
- ICE (Institution of Civil Engineers) and IStructE (Institution of Structural Engineers) (1949) *Code of Practice for Simply Supported Steel Bridges*. ICE Publishing, London, UK.
- Inglis CE (1932) Dynamic effects in railway bridges. *The Structural Engineer* **10(11)**: 450–462.
- Imam B and Salter PA (2018) Historical load effects on fatigue of metallic railway bridges. *Proceedings of the Institution of Civil Engineers – Bridge Engineering* **171(1)**: 49–62, <https://doi.org/10.1680/jbren.15.00046>.
- Imam B, Righiniotis TD, Chrysanthopoulos MK and Bell B (2006) Analytical fatigue assessment of a typical riveted UK rail bridge. *Proceedings of the Institution of Civil Engineers – Bridge Engineering* **159(3)**: 105–116, <https://doi.org/10.1680/jbren.2006.159.3.105>.
- Mott MacDonald (2001) *West Coast Route Modernisation, Resonance Study Phases 2 and 3*. Mott MacDonald, London, UK, project ref. W084-040-CV-REP-222256-Issue PO1.
- NR (Network Rail) (2006) NR/GN/CIV/025 Issue 3: The structural assessment of underbridges. NR, London, UK.
- Spindel JE, Goodman DF, Sowden AM, Wigley PJG and Lucas GJ (1975) Informal discussion. Railway bridge structures. *Proceedings of the Institution of Civil Engineers* **58(1)**: 95–99, <https://doi.org/10.1680/iicep.1975.3887>.
- Tobias DH and Foutch DA (1995) Overview of a research program investigating riveted railway bridges. *Structural Engineering Review* **7(3)**: 231–243.
- Xie M, Bessant GT, Chapman JC and Hobbs RE (2001) Fatigue of riveted bridge girders. *The Structural Engineer* **79(9)**: 27–37.