Division / Business Unit:	Safety and Systems
Function:	Track & Civil
Document Type:	Guideline

Guideline for Using Bridge–Track Modelling to Assess Fatigue in Existing Critical Transom-Top Bridges

ETG-09-06

Applicability

ARTC Network Wide

Publication Requirement

Internal / External

Primary Source

Document Status

Version #	Date Reviewed	Prepared by	Endorsed	Approved
1.0	08 Oct 25	Engineering Services	Principal Structural Engineer	Manager Engineering Services 27/10/2025

Amendment Record

Amendment Version #	Date Reviewed	Clause	Description of Amendment

Table of Contents

1.1 Purpose

The purpose of this guideline is to provide recommendations for assessing fatigue in existing critical steel transom-top bridges using realistic bridge–track modelling.

The objective is to prevent unnecessary extensive and expensive maintenance actions—whether costly permanent measures, such as repair, strengthening, or replacement of the bridge and its critical members, or temporary measures, such as speed or load restrictions—from being planned or implemented solely on the basis that a structural member or the bridge is deemed critical, prior to applying this guideline and when there are no visible signs of fatigue in the bridge.

In this context, "criticality" in theoretical remaining fatigue life assessment refers to any structural member that has an estimated remaining fatigue life of 20 years or less at any point in time. However, this guideline may be applied to other bridges with an estimated remaining fatigue life of more than 20 years, if requested by ARTC.

1.2 Scope

This guideline recommends a more realistic assessment of fatigue for critical steel transom-top bridges, which may consist of, but are not limited to, simply supported transom-top spans, through or lattice truss spans, half-through (U-frame) spans, or any member in a superstructure or substructure that may be subjected to cyclic fatigue loading.

To apply this guideline, it is assumed that the user is an experienced engineer who is aware of the bridge assessment principles in Ultimate Limit State (ULS), Serviceability Limit State (SLS), and Fatigue Limit State (FLS) in accordance with AS 5100, and has an acceptable level of knowledge in structural modelling, particularly as needed for the load rating and fatigue assessment of ARTC underbridges in accordance with ARTC ETE-09-05 Load Rating of Underbridges. Accordingly, this guideline does not cover the fundamentals of structural modelling, structural engineering principles, or bridge loading requirements, as addressed in AS 5100, relevant ARTC standards or procedures, and other applicable design or assessment codes.

1.3 Document Owner

The Manager Engineering Services is the Document Owner. Queries should be directed to standards@artc.com.au in the first instance.

1.4 References

AS 5100	Bridge Design
ETE-09-05	Load Rating of Underbridges
ETP-09-04	Bridge Transom

1.5 Definitions

Term or acronym	Description
CWR	Continuous Welded Rail
DLA	Dynamic Load Allowance
FE	Finite Element
FFU	Fibre-Reinforced Foamed Urethane
FLS	Fatigue Limit State
GM	Grillage Model
HCF	High-Cycle Fatigue
SLS	Serviceability Limit State
ULS	Ultimate Limit State

2 Fatigue Assessment

In brief, fatigue assessment has three main components:

1. Stress range determination (stress analysis):
   Evaluation of the stress range experienced by critical structural details under cyclic loading, which is the algebraic difference between two extremes of a particular stress cycle.
2. Cumulative loading (number of cycles) determination:
   Estimation of the total historical number of stress cycles acting on the bridge over its service life.
3. Fatigue resistance classification (detail category):
   Characterisation or numerical designation of fatigue resistance given to a particular detail for a given direction of stress fluctuation, which is typically a standard category that accounts for variability in material properties, fabrication tolerances, and residual stresses.

Together, these three components form the basis for estimating fatigue life, typically using a linear cumulative damage calculation based on the Palmgren-Miner rule. This method facilitates the assessment of safety margins against cumulative damage.

From the above three components, stress analysis due to a nominated loading event is the focus of this guideline. For critical ARTC steel transom-top bridges, stresses should be obtained using a bridge–track modelling approach, rather than relying solely on a simplified bridge Grillage Model (GM) that excludes the track, as the latter may lead to inaccurate results. Although this guideline addresses fatigue assessment only for steel bridges, the stress analysis part of the fatigue assessment may also be applied to other old members, such as wrought iron or other ductile materials, or other ULS or SLS problems.

Determining the exact historical loading spectrum and the corresponding number of stress cycles experienced by an existing railway bridge since its construction—as well as the current or future expected number of cycles—is challenging due to limited historical data availability. For fatigue assessment of a specific bridge, ARTC should be consulted to establish the nominated loading events and the required number of cycles for each historical period.

Detail categories are normalised in AS 5100 in a way that a particular S-N curve can be assigned to a specific standard detail.

3 Step-By-Step Fatigue Assessment Guide

1. Where requested by ARTC, a theoretical remaining fatigue life assessment should be conducted for a transom-top bridge, as usual, where rails and transoms are not included in the stress analysis. This assessment should involve either applying point loads to a developed GM of the bridge or using uniformly distributed loads (In accordance with AS 5100 requirements for open-deck steel bridges). Refer to Appendix A, Model 1. Alternatively, a simplified calculation of nominal stresses may be carried out manually. The objective is to obtain the normal (bending and/or axial) and/or shear (or torsion) stresses at critical details induced by the nominated loading event.
2. The second moment of area, I_z, for a corroded member, should be calculated as below:
   - No cross-sectional loss applied until 1990.
   - 50% of the as-is cross-sectional loss applied from 1990 to 2020.
   - 100% of the as-is cross-sectional loss applied from 2020 onwards.

If a repair or strengthening (other than full member replacement) was carried out—for example, in 2010—that enhanced the geometric properties of a member, a 100% cross-sectional loss should be assumed for the period from 1990 to 2010.

For defects other than a cross-sectional loss or corrosion (e.g., a crack or buckling that affects capacity), an appropriate reduced second moment of area in each period should be taken based on engineering judgment.

3. If the fatigue assessment using (1) indicates that the member is critical, this guideline should be used. No costly permanent works, such as repair, strengthening, or replacement of the member, or speed or load restriction, should be proceeded with or planned before using this guideline when the member or bridge does not exhibit any visual signs of fatigue.
4. Fatigue assessment of bridges should comply with AS 5100:2004. The detail categories should comply with AS 5100:2017 (2024 amendment), which have lower reference fatigue strengths compared to AS 5100:2004.
    - At this stage, the $S - N_R$ graphs in AS 5100: 2017 (2024 amendment) should not be used, as they do not provide corresponding equations for calculating N_{Ri} and then D_d, but rather rely specifically on the graphs. The absence of clear equations may lead to subjective interpretations of numbers and significantly affect the resulting theoretical damage and remaining fatigue life of old steel bridges, as obtaining definitive results solely from logarithmic $S - N_R$ graphs without specific equations are challenging.
    - The application of Dynamic Load Allowance (DLA) should comply with AS 5100:2017 (2024 amendment); however, the DLA should not increase in any historical period due to the lack of or defects in transition approaches. Where fatigue assessment is required for members or details—such as connections, bracings, or lattices—it should be performed using methods like the rainflow counting method, rather than applying arbitrary multipliers (e.g., 1.4, 1.5, or 2) to the number of stress cycles. This could lead to either an increase or a decrease in the number of stress cycles.
5. Structural Modelling:

    All models should be developed using a GM; no further extensive work, such as a rigorous structural analysis or development of a Finite Element (FE) model, is needed.

    - Geometry:

        For multi-span simply supported transom-top or half-through spans, one span (bearing to bearing) may only be modelled with the transoms and rails. Refer to Appendix A, Model 2.

        For through or lattice trusses, the transoms and rails should be modelled for the whole truss rather than only one bay; however, only one span (bearing to bearing) should be modelled. Refer to Appendix A, Model 7.

    - Rail:

        For current rail profiles over the bridge, consult with the Structures Representative. It should be noted that, currently, most ARTC interstate freight lines (23–25 TAL) should have a minimum rail profile of 53 kg/m, while the heavy haul lines (30 TAL) should have a minimum rail profile of 60 kg/m. In cases where the rail profiles are uncertain, both currently and historically, in the models, 47 kg/m rails should be assumed for 23–25 TAL, and 60 kg/m rails for 30 TAL. For fatigue assessment, a vertical section loss of maximum

20% in the railhead should be applied for all the periods throughout the bridge's service life.

For through or lattice trusses, it is worth noting that, prior to the 1960s, tracks mostly featured jointed rails. Continuous Welded Rail (CWR) should be assumed on through or lattice trusses for fatigue assessment after 1960. If fatigue assessment is required before 1960 for these bridges, a fixity release in rotation about the minor axis of the rail (FFFFFR, where -Y is direction of gravity) at the middle of the stringers (bending) and at the end of stringers (shear) should be modelled to simulate the unknown location, condition, or behaviour of the old and possibly damaged rail joints (possibly fishplates) at these locations (no expansion is required). It is possible that older mechanical joints were suboptimal and did not provide full bending restraint of the rails in the direction of wheel loads. For other transom-top bridges, CWR should be assumed throughout the bridge's service life.

- Transom:

  Bending or shear stresses in stringers or main girders in models are not very sensitive to crossing elements, such as transoms, but rather to rail profiles. Timber transoms with a reduced size of 140 (depth) × 250 (width) × 2600 (length) mm^3, F22-stress grade, with a reduced modulus of elasticity of E=14,400 MPa, should be used to be on the conservative side. No missing transom is required in models. For all transom types other than timber (such as Fibre-Reinforced Foamed Urethane, FFU), a conservative modulus of elasticity of 8,000 MPa, regardless of the material, should be applied only to the period after FFU was installed. For transom spacing, a typical spacing of 500 mm may be used. The stresses should not be very sensitive to the transom spacings either, so their spacings may vary in a model.

- Connection:

  Connection between the rails and transoms, as well as the transoms to the top flange of the girders or stringers, should utilise standard node-to-node connections (no restraint or constraint). These connections may be modelled with or without geometric offsets. In practice, while rail fasteners would not provide full rotational fixity to the rail, research demonstrates that this limitation has a minimal impact on the resulting stresses caused by the actual vertical or lateral train loading, regardless of the type of rail fastener[1]. Moreover, incorporating any nominal rotational stiffness or node constraints may unnecessarily complicate models without significant accuracy gains. This is largely due to the sufficient provision of rail fasteners, installed at e.g., 500 mm intervals beneath the rail along the spans, which contribute to the reduction of girder stresses by incorporating the rail's capacity to resist bending and shear forces. Historically, and prior to the development of modern resilient or zero toe load rail fasteners, connections between rails and transoms or sleepers were typically made using dog spikes, screw spikes (also known as coach screws), bolted chairs, or bolts with baseplates. All these fasteners were much more rigid than the modern rail fasteners.

---

[1] *Ghiasi et al., "Estimation of nosing load in existing railway transom top bridges based on field testing and finite element modelling," Advances in Bridge Engineering, vol. 5, issue 14, 2024.*

- Loading:

  Static moving loads should be applied to rails to obtain cyclic stresses.

6. Structural Analysis:
   - ARTC ETE-09-05 Load Rating of Underbridges requires that nonlinear analysis (geometric) be carried out rather than the linear analysis for load rating of existing truss spans. The reason for this is that the old through or lattice trusses predominantly comprise non-compact braced members, and model analyses are typically performed prior to the manual calculation of flexural buckling loads. In such cases, there is a risk of non-compliance with AS 5100 requirements, as first-order linear elastic analysis neglects the global system ($P - \Delta$) and local member ($P - \delta$) second-order effects, as well as the potential slackening of tension-only members. For fatigue assessment, in the absence of thorough sensitivity analyses that account for geometric imperfections or loading-path deviations, it is recommended that both first-order and second-order analyses (incorporating appropriate tension-only members) be carried out. Member stresses obtained from the two approaches should then be compared, with the more conservative stress results adopted. In most cases, the outcomes of both analyses are expected to be reasonably close, unless the through or lattice truss possesses an equal or relatively lower capacity compared with the applied loads.
   - A GM is fundamentally formulated on the assumption of linear elastic material behaviour and the derivation of the stiffness matrix based on beam theory. However, when the applied loading in the GM exceeds the critical buckling capacities of some beam members (node to node, i.e., only local $P - \delta$ in some members, not global $P - \Delta$)—for instance, when a higher load, such as 30 TAL, is adopted in the analysis of a through truss model located in the interstate freight line (23–25 TAL), while buckling calculations indicate that certain member(s) in that model lack sufficient capacity under such conditions—the underlying assumptions of the GM are no longer valid. Under these conditions, the stress and force distributions predicted by the GM may not reliably capture the actual structural response, as the model neglects material and geometric nonlinearities, including post-buckling effects of those individual members on the overall bridge system. Accordingly, always prior to undertaking stress analysis for fatigue assessment, it is essential to assess the bridge's safe load rating and check the applied historical loading events to ensure that they are within the bridge's capacity. These nominated historical loading events should typically correspond to 23–25 TAL for interstate freight lines and 30 TAL for heavy haul lines.

4 Recommendations

1. After applying this guideline for critical members, if these members have shown that they passed or are very close to the end of their theoretical remaining fatigue life, a risk management strategy that considers planning for the repair, strengthening, or replacement of members should be implemented. Temporary measures, such as speed or load restrictions over the assessed bridge, may also be implemented before a permanent maintenance action is completed. It is anticipated that the bridge will exhibit some degree of visual signs of fatigue if the nominated loading spectrum and the associated number of cycles are representative of the actual cumulative load history experienced by the bridge.
2. Review the nominated loading spectrum and the associated number of cycles to ensure a more accurate representation is considered; when the bridge–track fatigue assessment

suggests that there is still a reasonable remaining fatigue life and no immediate risk of fatigue initiation, while the assessed member shows visual signs of fatigue within the bridge.

3. Review the cause and location of the cracks. Such cracks may originate from multiple sources; historical derailments that induce local damage and increase stresses within individual members, additional mechanisms include localised out-of-plane distortion of small, e.g., welded brackets or gusset plates arising from secondary effects such as misalignment or poor track geometry, vehicle impact loading on the bridge superstructure, excessive rigidity at end connections of stringers, or cracking associated with advanced corrosion or delamination. In addition, materials other than steel may develop microcracks during fabrication, such as during casting, rolling, or forging. These issues do not necessarily correspond to classical High-Cycle Fatigue (HCF) mechanisms; rather, they may generate large stress ranges over a limited number of load events, leading to localised crack initiation and propagation.
4. If strengthening of a member is planned, evaluate whether this is beneficial before moving forward. The strengthening of a structural member may only extend its fatigue life for future periods; it does not improve fatigue for the member's originally designed cross-section or as-is status. This is because the existing member has already endured a significant number of fatigue cycles over its lifespan, along with its original detail category and accumulated stresses.

Appendix A

Field Testing

A field test was carried out on a steel transom-top bridge under the loading of a real short train.

Bridge Description:

As shown in Figure 1, this bridge comprises 28 spans, each with a nominal length of 8.4 m, giving a total bridge length of over 235 m. All spans are simply supported on concrete pier walls and feature CWR on timber transoms, which rest on steel girders. The steel girders are braced using wind and sway braces.

The bridge is in good condition, with some minor signs of rusting on some steel parts, with no visible distressed members or cross-sectional losses. Transoms are also in good condition and are typically 150 (depth) × 250 (width) × 2600 (length) mm^3. Transom spacing is 450 to 500 mm as measured. Rails are 53 kg/m, and the line section is 25 TAL. Figure 2 shows a sketch including site measurements.

Figure 1-View of the Tested Bridge

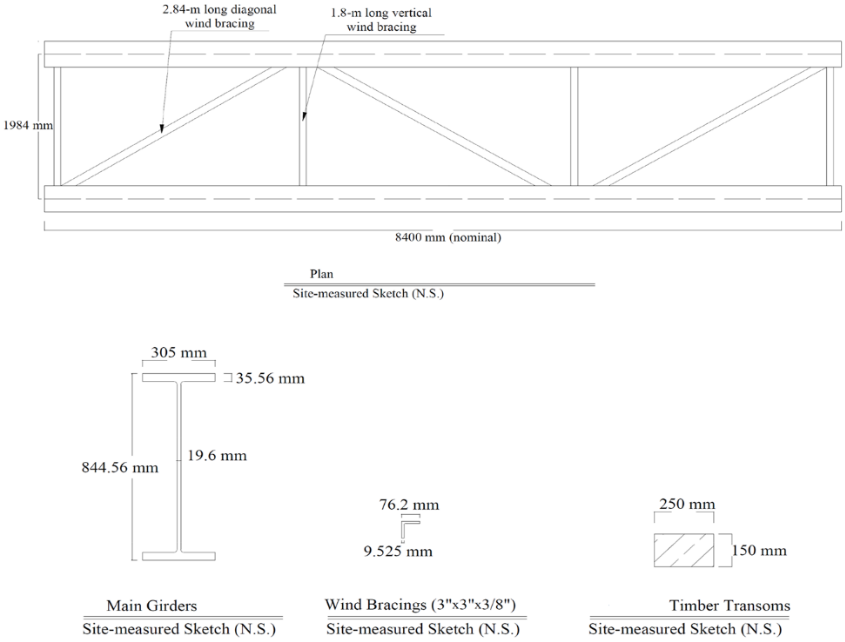


Figure 2-Site Measurements

Instrumentation:

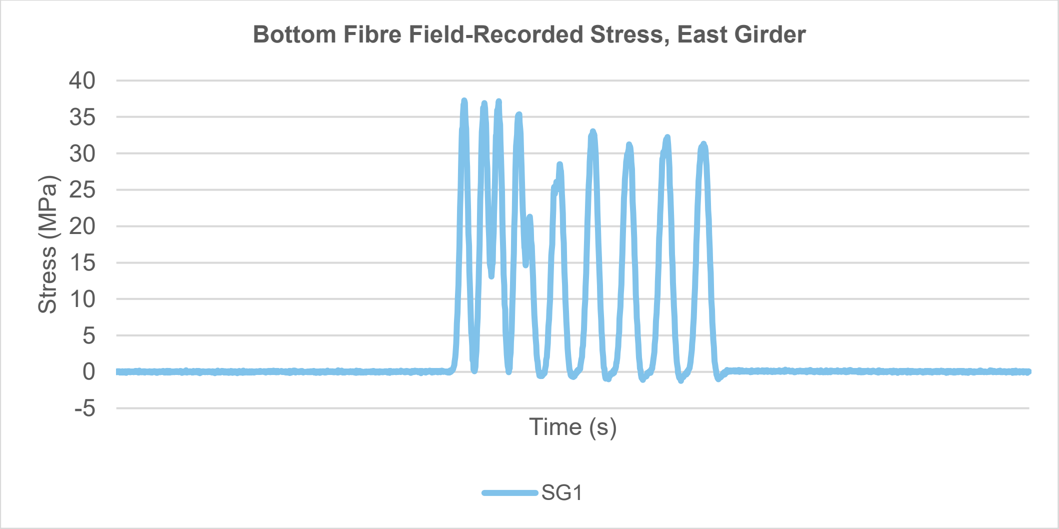
Two random spans were selected for field testing. BDI strain transducers (gauges) were installed under the bottom of the girders at mid-spans (Figure 3) to measure the bottom fibre stresses due to the passage of a tested short train.

The short train (Figure 4) comprised two six-axle locomotives hauling five four-axle container wagons, giving a total of 32 axles with a maximum reported axle load of 22.97 t. When crossing the bridge, the train's speed was recorded as 52 km/h.

Figure 3-View of the Tested Short Train Crossing the Bridge

Figure 4-Strain Gauges Installed under the Bottom of the Girders (at Mid-Span)

Figure 5 shows the field-recorded stresses at strain gauges SG1 and SG2 in one of the tested spans due to the passage of the short train. As can be seen, the maximum (tensile) and minimum (compressive) stresses were recorded to be around 37 MPa and -1.5 MPa, respectively.



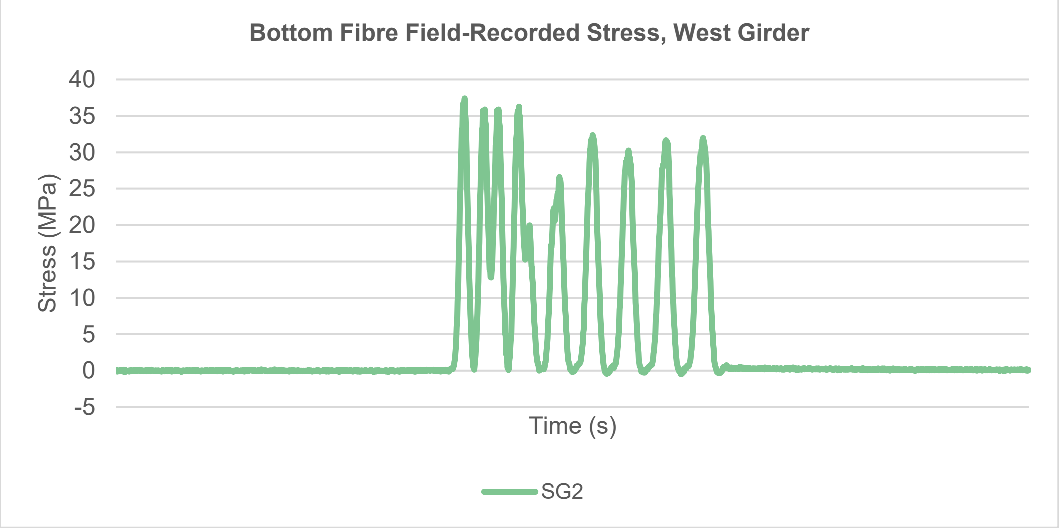


Figure 5-Bottom Fibre Field-Recorded Stresses at SG1 (East Girder) and SG2 (West Girder) due to the Passage of the Tested Short Train

Bridge Modelling and Sensitivity Analysis

A simple GM for the tested span of this bridge was developed in Space Gass (Model 1). The tested short train with the same axle loads and spacing was modelled in two scenarios, named SC1 and SC2. SC1 was the scenario in which the moving point loads of the tested short train were applied directly to the girders. SC2 was the scenario that followed the AS5100 requirement for open deck steel bridges, in which moving loads were distributed over a length of 1.2 m. Although the transom spacings at the bridge were measured to be closer, a 1.2 m length was selected to obtain the lowest possible stress over the maximum required length in accordance with AS 5100. Both scenarios, SC1 and SC2, included a dynamic factor of 1+DLA for the bending effect, equal to 1.35, which was a reduced standard DLA for a speed of 52 km/hr, and was applied to each moving axle. Figure 6 illustrates representative snapshots of Model 1 for loading scenarios SC1 and SC2. Figure 7 shows the graphs of cyclic bending and axial stresses at the mid span, the girder's bottom fibre stress due to the passage of SC1 and SC2.

As can be seen, both modelled SC1 and SC2 exhibit significantly higher stresses (nearly double in some cycles) than those actually recorded at the SG1 and SG2. SC2 shows marginally reduced values compared to SC1.

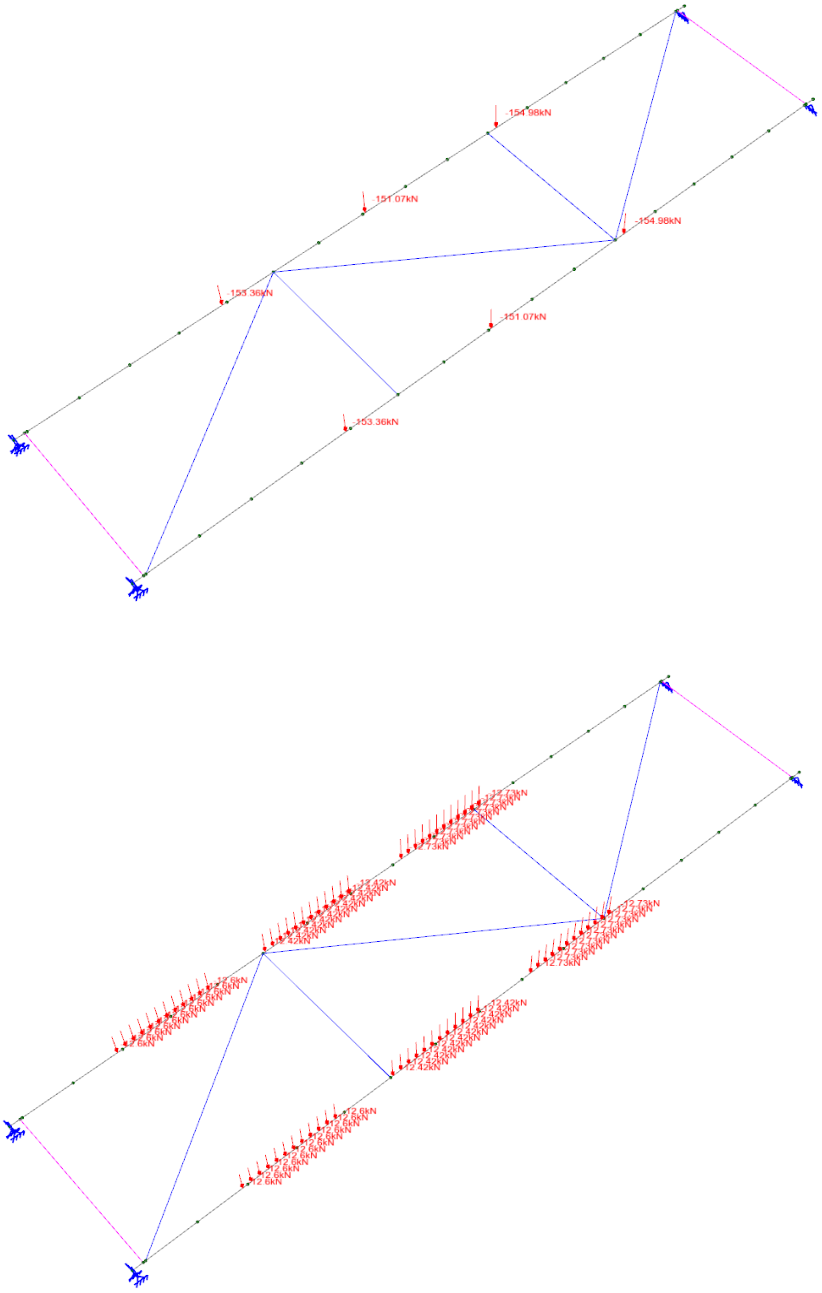


Figure 6-Model 1: A Single-Span Bridge Model, Top SC1 and Bottom SC2 (Some Snapshots)

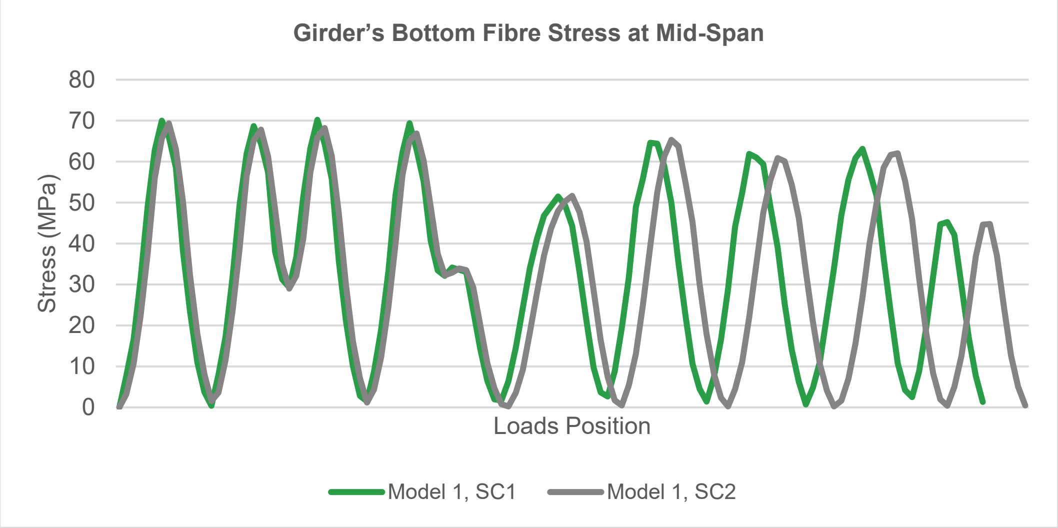


Figure 7-Girder's Bottom Fibre Stress (Mid-span) Obtained from Model 1 for SC1 and SC2

Since Model 1 (common industry practice) proved to be inaccurate, a second model (Model 2) was developed to generate more realistic results. This model was identical to Model 1; however, it featured timber transoms spaced at 500 mm and 53 kg/m rails, as measured in the field. The transoms were modelled as ideal and undamaged F22-stress grade, typically 150 (depth) × 250 (width) × 2600 (length) mm^3. The rails were also modelled as ideal and undamaged 53 kg/m rail profiles. In Model 2, moving point loads (including DLA-bending) were applied to rails to obtain stresses in the girders (mid-span). Figure 8 illustrates a representative snapshot of Model 2. Figure 9 also shows the graphs of cyclic bending and axial stress at mid-span, the girder's bottom fibre stress in this model. Although the stress cycles better matched the recorded stresses at SG1 and SG2, the obtained stress extremes were entirely tensile, which did not fully reflect the actual measurements at SG1 and SG2 that also included some compressive ranges.

In the next phase, three additional models (Models 3, 4, and 5) were developed. These were based on Model 2 but extended to three spans, incorporating CWR along the full length. The gaps between adjacent girder ends were set to match field measurements to replicate realistic bridge–track conditions further. Model 3 was a direct extension of Model 2 to three spans. Model 4 was identical to Model 3 but incorporated damaged timber transoms, reduced in size to 140 mm (depth) × 250 mm (width) × 2600 mm (length), with a reduced modulus of elasticity of E=14,400 MPa. Model 5 was identical to Model 4 but represented a more severe condition (only for the purpose of the sensitivity analysis) with damaged or missing transoms and a reduced rail profile of 47 kg/m. Figure 10 presents a representative view of these models. Figure 11 presents the cyclic bending and axial stress at mid-span, illustrating the bottom-fibre stress of the girder induced by the train passage. As anticipated, reducing the rail size led to a marginal increase in stress; however, the condition and dimensions of the transoms had a comparatively minor effect on the stress results and were barely distinguishable on the graph.

Figure 12 presents a comparison of Model 2 (single-span bridge–track model), Model 5 (three-span bridge–track model with damaged transoms and reduced rail sizes), and the field-recorded stresses (time-step adopted) at SG1 and SG2, all shown on a single graph. As can be seen, Model 5 (or Models 3 or 4) closely matches the field-recorded stresses. These models accurately capture the development of negative bending moments at the pier regions due to the CWR restraint along the full span length, as evidenced by the compressive regions at the dips corresponding to negative stresses.

Model 2 can be considered a simplified yet reasonably good model; although it does not perfectly follow the pattern of the field-recorded stress fluctuations, it provides sufficiently realistic stress cycles. The minor discrepancies between Model 5 (or Models 3 or 4) and the field-recorded stresses would be attributed to the time-varying nature of wheel–rail interaction forces in coupled vehicle–bridge interaction, as well as unknown conditions of some wheels, which cannot be fully represented by a constant standard DLA. A comprehensive analysis of this effect may require a transient dynamic vehicle–bridge interaction model that accounts for interactive stiffness, mass, and damping, as well as understanding the wheels' conditions. However, in the absence of such a detailed assessment, AS 5100 recommends approximating dynamic effects by applying DLAs to static loads to account for the influence of vehicle–bridge interactions based on the characteristic length of the member, bridge geometry, and vehicle speed.

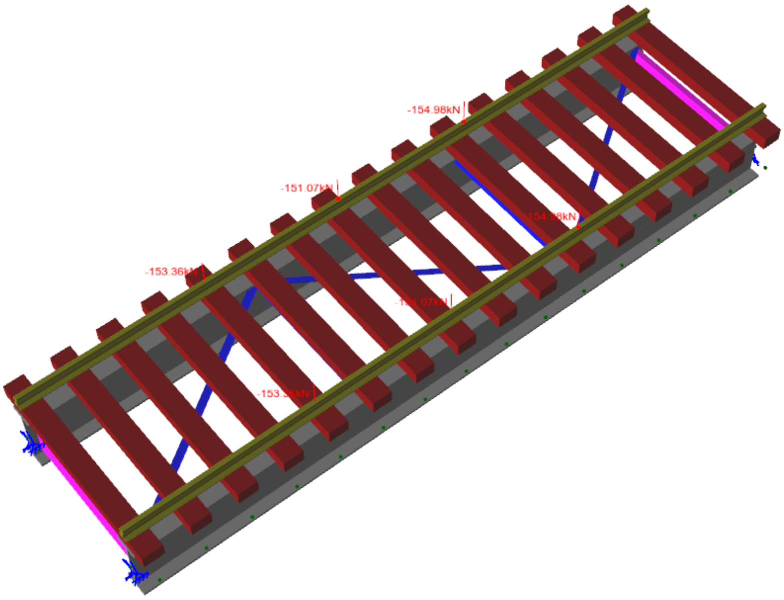


Figure 8-Model 2: A Single-Span Bridge–Track Model (a Snapshot)

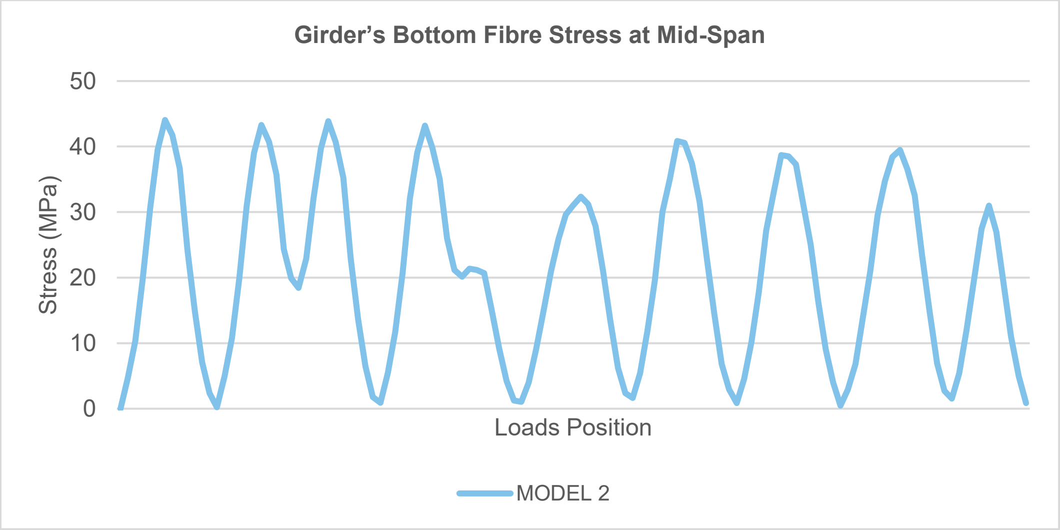


Figure 9-Girder's Bottom Fibre Stress (Mid-span) Obtained from Model 2

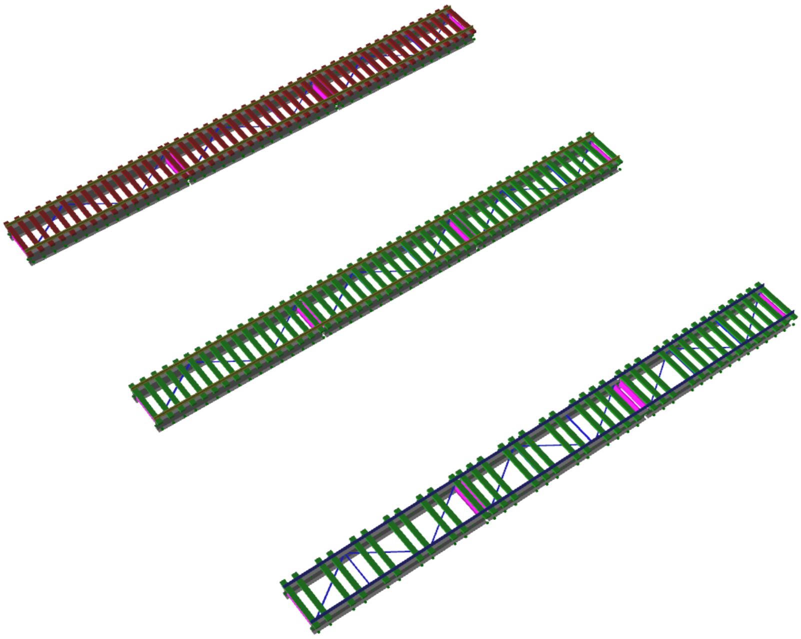


Figure 10-Model 3 (Top), Model 4 (Middle), and Model 5 (Bottom): Three-Span Bridge–Track Models (Some Snapshots)

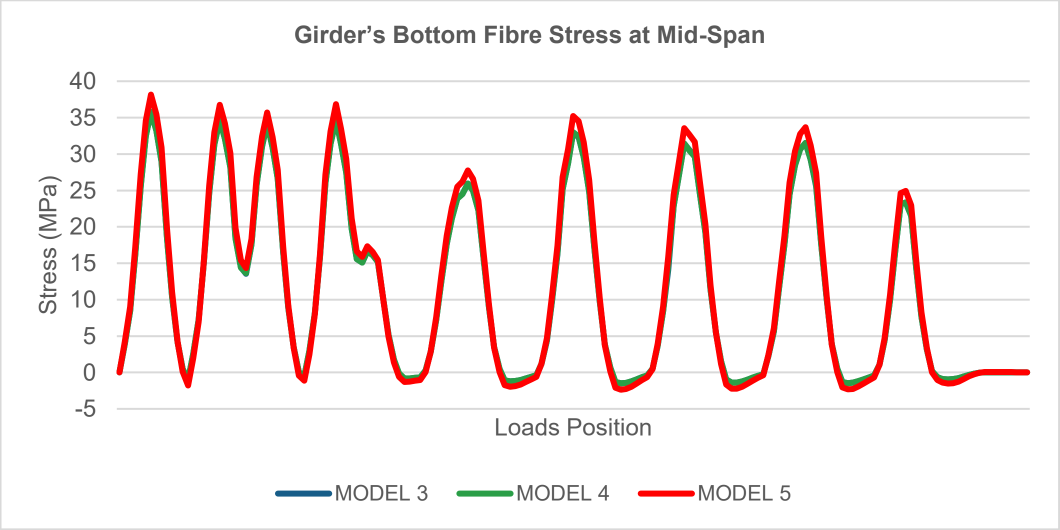


Figure 11-Girder's Bottom Fibre Stress (Mid-span) Obtained from Model 3, Model 4, and Model 5

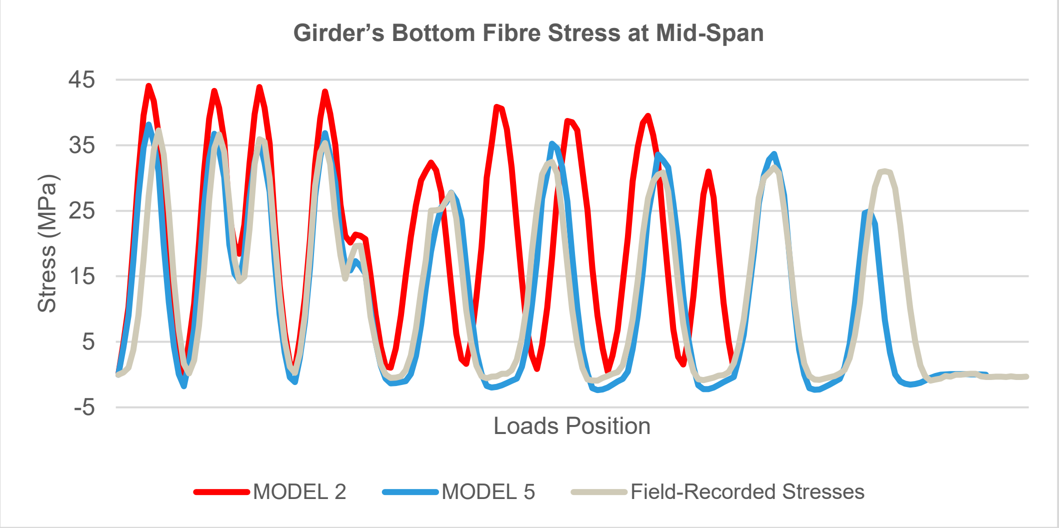


Figure 12- Girder's Bottom Fibre Stress (Mid-span) Obtained from Model 2, Model 5 (or Model 3 or 4), and the Field-Recorded Stresses (Time-Step Adopted)

Fatigue Assessment of a Through Truss Span- An Example

As demonstrated, stresses in critical bridges should be assessed using realistic bridge–track modelling to ensure that theoretical estimates of remaining fatigue life are better aligned to reflect actual conditions.

In this section, an example is presented in which a simply supported through truss span is modelled both without track (Model 6) and with track (Model 7) to determine the stresses in selected members and to assess the accumulated damage in each case.

Figure 13 shows Models 6 and 7. This 48 m-long simply supported through-truss span is verified to possess sufficient capacity for 30 TAL loading. An intermediate stringer, cross girder, vertical member, and diagonal member were selected for stress analysis under a representative historical loading event of 3 × 90 Class locomotives hauling 80 × 120 t wagons. Accumulated fatigue damage was then estimated for the intermediate stringer using both the single-span bridge model (Model 6) and the single-span bridge–track model (Model 7). Figure 14 illustrates stresses in the selected members due to the nominated historical loading event. As anticipated, incorporating the track into the model results in a substantial reduction in bending and axial stress cycles in the intermediate stringer (e.g., approximately 45%), while the intermediate cross girder experiences a moderate reduction (e.g., 20%). The reduction in axial stress is comparatively smaller, e.g., approximately 10%, for the selected intermediate vertical and diagonal members.

In this example, fatigue assessment is performed for the intermediate stringer. A detail category of 112 (for normal, bending stress) is assumed, with the effective stress range evaluated only for the bottom (tensile) flange. Considering a 10-year period (after 1960) with 2,500 vehicles per year under the assumed historical loading scenario—including the full fatigue dynamic factor of 1+0.5×DLA = 1.22 for a 6.9 m long intermediate stringer—the accumulated damage is calculated as 0.66 for Model 6 and 0.06 for Model 7. Accordingly, Model 7 reduces the accumulated damage by elevenfold and increases the estimated theoretical remaining fatigue life from 34% to 94%, representing nearly a threefold improvement.

Note- The improved predictions of stress levels for the estimation of accumulated damage and theoretical remaining fatigue life for critical details of main girders, stringers, and cross girders using the bridge–track modelling approach are promising, assuming that the condition of the running rails and rail connections is always verified through general and detailed visual and patrol inspections. However, the theoretical distribution of wheel loads and the symmetrical response of old truss members not directly supporting the rails remain influenced by the stiffness continuity of braced members and connection details, as well as vehicle–bridge interaction and wheel–rail contact effects. Asymmetric responses were observed in field tests of some old railway truss bridges[2]*: non-ideal member and connection stiffnesses (and realistic boundary conditions) produced local load-redistribution that caused some truss members' stresses to be under- or over-predicted by idealised models. Such effects may not be reliably captured by simplified, statically analysed GMs that have not been calibrated with field vibration data through deterministic or stochastic model-updating techniques, as these models often fail to represent localised stiffness variations within members and unmodeled connections. While ULS analyses (such as load rating) for older through trusses address these issues through appropriate load factors, full DLAs, and higher-band virtual loading (e.g., RAS), FLS analyses apply no load factors and solely rely on lower DLAs. In all cases, an arm's-length inspection is essential to confirm the bridge's as-is condition and to refine models prior to conducting load rating and fatigue assessments.*

Nonetheless, the bridge–track modelling approach demonstrates a substantial potential to enhance predictions of elastic stresses and estimation of theoretical remaining fatigue life of critical members—typically the main girders, stringers, and cross girders—compared to simplified bridge-only GMs that do not incorporate the track.

---

[2] *Bacinskas et al. "Field Testing of Old Narrow-Gauge Railway Steel Truss Bridge", Procedia Engineering: Modern Building Materials, Structures and Techniques, 2013.*

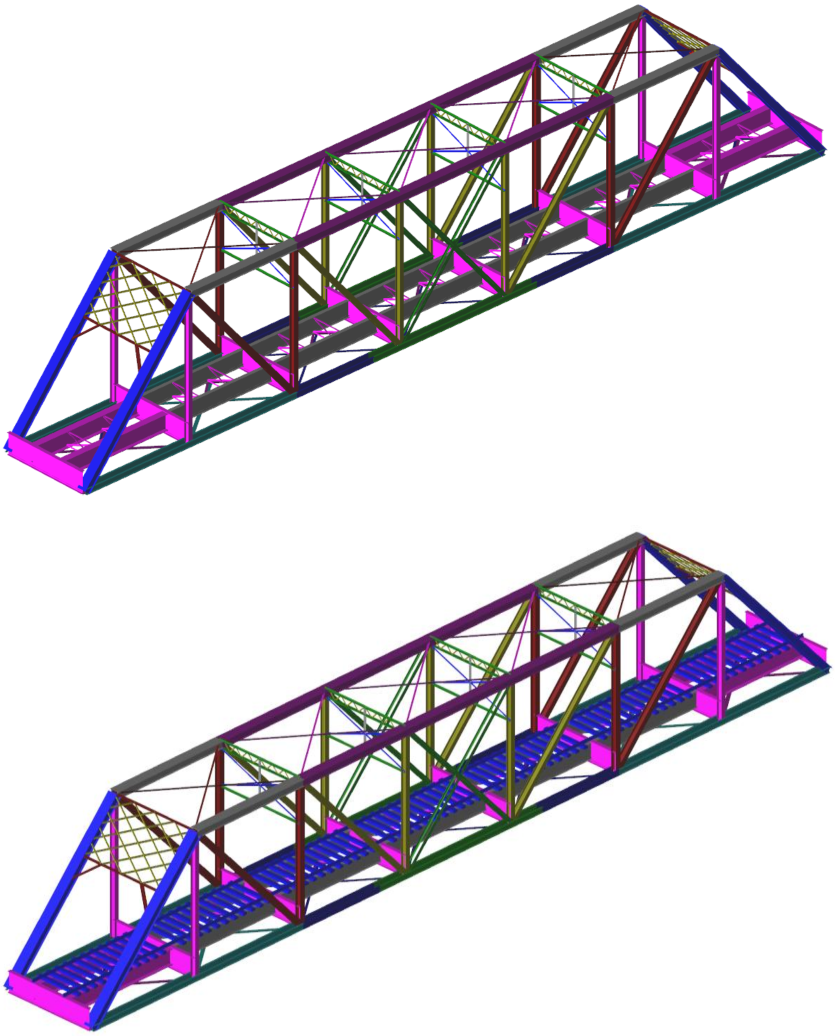
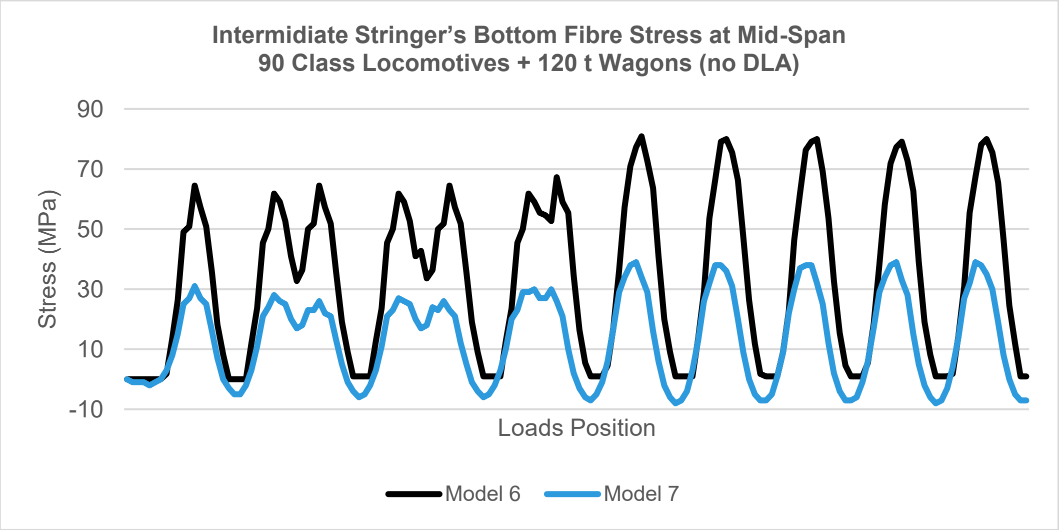
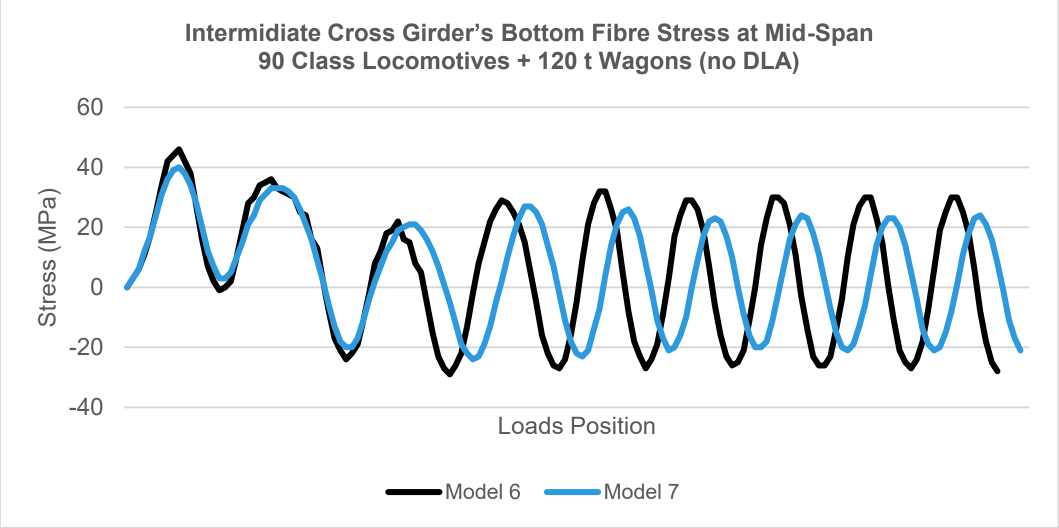
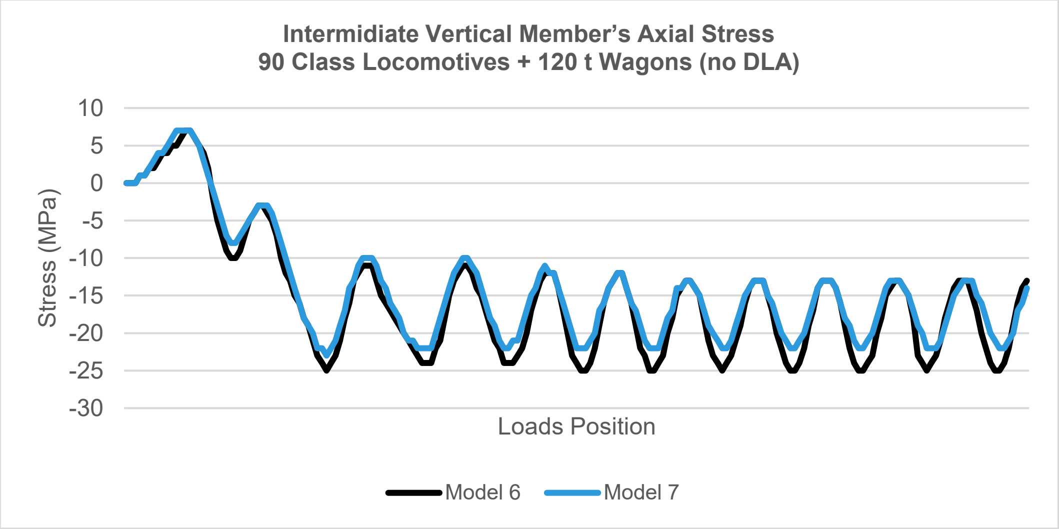


Figure 13- Model 6 (Top) and Model 7 (Bottom): A Single-Span Bridge and Bridge–Track Model of a Through Truss (Some Snapshots)







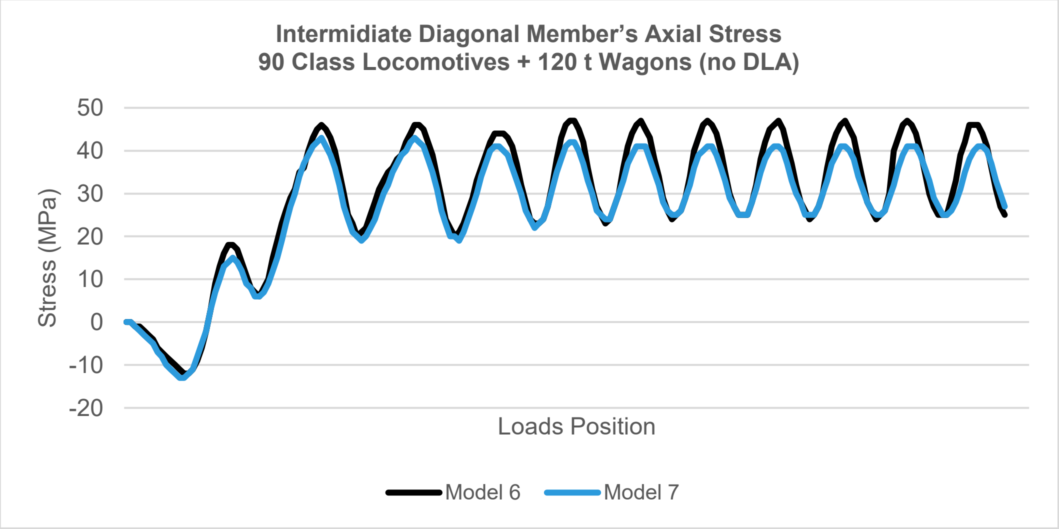


Figure 14- Stresses in Selected Analysed Members in Model 6 and Model 7 (In the Above Graphs, Positive Stresses Indicate Tension)