IDEA

Innovations Deserving
Exploratory Analysis Programs

Rail Safety IDEA Program

Development of a Fatigue Load for Railway Bridges

Final Report for
Rail Safety IDEA Project 45

Prepared by:
Stephen Dick, PhD, SE,
Robert Connor, PhD, PE,
Cem Korkmaz, PhD
Myriam Sarment, MSCE, EI
Cecelia Maginot, BSCE
Purdue University Bowen Laboratory

September 2022

NATIONAL ACADEMIES Sciences Engineering Medicine

TRB TRANSPORTATION RESEARCH BOARD

Innovations Deserving Exploratory Analysis (IDEA) Programs Managed by the Transportation Research Board

This IDEA project was funded by the Rail Safety IDEA Program.

The TRB currently manages the following three IDEA programs:

- The NCHRP IDEA Program, which focuses on advances in the design, construction, and maintenance of highway systems, is funded by American Association of State Highway and Transportation Officials (AASHTO) as part of the National Cooperative Highway Research Program (NCHRP).
- The Rail Safety IDEA Program currently focuses on innovative approaches for improving railroad safety or performance. The program is currently funded by the Federal Railroad Administration (FRA). The program was previously jointly funded by the Federal Motor Carrier Safety Administration (FMCSA) and the FRA.
- The Transit IDEA Program, which supports development and testing of innovative concepts and methods for advancing transit practice, is funded by the Federal Transit Administration (FTA) as part of the Transit Cooperative Research Program (TCRP).

Management of the three IDEA programs is coordinated to promote the development and testing of innovative concepts, methods, and technologies.

For information on the IDEA programs, check the IDEA website (www.trb.org/idea). For questions, contact the IDEA programs office by telephone at (202) 334-3310.

IDEA Programs
Transportation Research Board
500 Fifth Street, NW
Washington, DC 20001

Development of a Fatigue Load for Railway Bridges

Rail Safety IDEA Program Final Report

IDEA Project RS-45

Prepared for

The Rail Safety IDEA Program
Transportation Research Board
National Academies of Sciences, Engineering, and Medicine

by

Stephen Dick, PhD, SE, Co-Principal Investigator
Robert Connor, PhD, PE, Principal Investigator
Cem Korkmaz, PhD
Myriam Sarment, MSCE, EI
Cecelia Maginot, BSCE

Purdue University Bowen Laboratory

September 19, 2022

Table of Contents

Acknowledgement

The researchers of this project wish to thank the project sponsors, the IDEA program. Their support and funding of this project are key in producing this critically needed research. The researchers also want to extend their appreciation to the expert review panel serving as technical advisors to the project. The expert review panel includes the following members:

Melvin Clark, Chairman – Hatch LTK
Ronald Berry, PE, P. Eng. – BNSF Railway
Tom Murphy, PhD, PE, SE – Modjeski and Masters, Inc.
Duncan Paterson, PhD, PE – Alfred Benesch & Company
John Unsworth, P.Eng. – AECOM, Retired Canadian Pacific Railway

Glossary

A	Fatigue category coefficient
AREA	American Railway Engineering Association
AREMA	American Railway Engineering and Maintenance-of-Way Association
CAFL	Constant Amplitude Fatigue Limit
DL	Dead Load
ft	foot or feet
GRL	Gross Rail Load (pounds)
I	Live Load Impact (percent)
k	kip (thousands of pounds)
ksi	kips per square inch
L_O	Overall length of a railcar between the coupler pulling faces (ft)
L_S	Span Length (ft)
lb(s)	pound(s)
LL	Live Load
Lt Wt	Light (empty) weight of a railcar (lb or k)
M_{MAX}	Maximum bending moment expected at any point (k-ft)
M_{RAM}	Absolute maximum moment range for a given railcar (k-ft)
M_{RMC}	Root-Mean-Cube Bending Moment Range
MRE	Manual for Railway Engineering published by AREMA
m_i	number of trains of type i
n	number of axles per railcar
n_i	number of axles per train type i
N	number of cycles based upon a stress range
OL	Overall length of a railcar over the coupler pulling faces (published dimension)
P	Axle load (lb or k)
P_{GRL}	Axle load at Gross Rail Load of railcar (lb or k)
P_{LW}	Axle load of the empty railcar (Light Weight) (lb or k)
Plf	pounds per linear foot
RMC	Root-Mean-Cube
S_I	Inside axle spacing of the railcar (ft)
S_O	Outside axle spacing of the railcar (ft)
S_{Reff}	Effective RMC stress range (using the Palmgren-Miner Rule
S_T	Truck axle spacing of the railcar (ft)
TC	Distance between truck center pins on a railcar (published dimension) (ft)
TW	Truck wheelbase; the distance over the extreme axles of a truck (ft)
VAFL	Variable Amplitude Fatigue Limit (ksi)

EXECUTIVE SUMMARY

The research developed a fatigue load for steel railway bridges. The task was undertaken to fill a knowledge gap to provide a fatigue evaluation method consistent with the loading behavior of actual railway equipment on bridge spans. The current methodology provides insufficient guidance for determination of cyclic behavior nor does it provide any sufficient basis for developing a rating system for fatigue of railway bridges, contrasting with providing an overall load capacity rating in terms of the basic design loads; the Cooper E80 Load. Design recommendations in North America for steel railway bridges are under the purview of the American Railway Engineering and Maintenance-of-way Association and are found in Chapter 15 of the Manual for Railway Engineering (MRE 15) (*1*).

The concept in this research was an examination of the behavior of typical railcar configurations for analysis of the characteristics of the cyclic behavior and applying those characteristics for developing a design load specifically for fatigue in steel railway bridges. The innovation was the creation of both the fatigue design load and development of computer software, known as CyclRR, to perform the analysis. The data for development of the fatigue design load was created by CyclRR. The software was used in multiple ways to create data and perform simulations.

CyclRR performs two basic functions. First, it creates the bending moment time-history is created for a train loading based upon simply-supported spans (nearly universal for railway bridges). Second, it calculates the typical section properties needed for fatigue analysis for the span length under examination. The software provides the section properties for any of the five standard design eras that have been defined by MRE 15. It can also be modified to use any standard loading and design criteria as the reference load instead of the Cooper E Load for railway bridges which was common for railroads prior to the development of the common AREMA criteria. With the bending moment time-history and section properties, the software calculates the stress time-history and cycle count so that all of the stress ranges relevant to fatigue analysis can be cataloged and counted for application in bridge fatigue analysis. This software is innovative in that no program has been specifically built for that purpose, and it can be modified beyond research activities. Potential uses include a cycle counting algorithm for use in real time so that the bridge engineer can track the severity of bridge fatigue for the railway system providing forecasting of when special inspections, repairs, or replacements may need to be made. The methods shown herein, allow for general comparisons across the entire inventory of the bridges without having to repeat the process for each individual bridge. The results of the analysis are considered estimates only, but these estimates serve to pinpoint areas of critical need where extra effort and resources can be reduced by avoiding time examining areas of less concern.

The fatigue load developed from the data provided by the software is called the F80 Load. The F80 Load takes advantage of the Alternate Load in MRE 15 by using its length dimensions in combination with adding spacing between multiple Alternate Loads to create a representative railcar configuration. The F80 Load reduces the magnitude of the Alternate Load axle loadings to reduce the amount of bending moment and moment range needed for fatigue evaluation and design to be consistent with typical actual axle loadings and avoid conditions that result in an unrealistically high stresses and stress ranges.

Current (as of this publication) design recommendations and details of fabrication, if followed properly, result in new designs that successfully avoid for the initiation of fatigue cracking. Older spans built prior to the current fatigue criteria of MRE 15, especially riveted bridges, may have concerns exist about the remaining fatigue life. The F80 Load is appropriate for analysis of the older bridges, and for development of a common rating system for fatigue of these bridges. One of the benefits of the rating systems for normal and maximum design loadings is the ability to judge the adequacy quickly on the basis of the equipment and strength ratings. The same approach would be a major advancement for fatigue assessment of older steel bridges as well. New steel bridges could have their fatigue resistance known at time of installation.

Knowledge of fatigue resistance and a specific point of comparison provided in design or rating is of great benefit to everyone involved with railway bridges, from the inspectors and engineers directly responsible for them, to senior staff responsible for overall structural health of the railway bridges and budget planners prioritizing repairs and replacements. One major benefit of this research was the ability to locate where fatigue effects can be expected to manifest themselves.

Owners are also aided by having a system that is simple to understand and quantify so that they can quickly prioritize issues. This is important in relation to public agencies who may be responsible for rail lines owned by public agencies needing to provide the general public with information on these issues. For Class 1 railroads, the need is to evaluate and screen a large number of bridges on the system. The same is true for short lines and regional railroads where budgeting for maintenance with limited resources can be focused on the critical needs. For public agencies budgeting is also an issue along with the ability to display measures demonstrating the need for proposed repairs or replacements.

IDEA PRODUCT

Fatigue of steel railway bridges has been an issue since steel was first used for railway bridges. The mechanism driving fatigue cracking has been understood since extensive research in the 1970s (*2*). While the mechanism causing cracking in the materials and details of the bridges was understood, less knowledge has been available concerning cyclic behavior in terms of railway service loading.

Railway bridges have not received the same level of study as highway bridges in understanding the cyclic behavior of trains. The purpose of this research was to fill the knowledge gap on cyclic behavior of railway loadings. Evaluating railway bridge fatigue is a more complex process than that caused by highway bridge loadings. Trains consist of a series of loads whereas highway loadings are more often individual loads from one vehicle. Additionally, different types of trains require different analysis techniques and solutions to obtain the necessary data for calculation of fatigue life. Moreover, an important interaction exists between the length of each railcar and the length of the span. Finally, each position on the span will have a unique reaction (e.g., quarter point versus midspan) where the results at one location are different from other locations.

The products of this are computer software for fatigue analysis of railroad bridges and a proposed fatigue design load for railroad bridges. The computer program was used to generate the data used in development of the proposed fatigue design load. Both the computer program and the proposed fatigue load have future uses.

The computer software, named CyclRR, performs bending moment beam analysis for simply supported structures under a series of moving loads. The loads are modeled after railroad cars and the program input requires the necessary data for setting the proper dimensions for structural analysis. CyclRR provides the five basic design load levels and associated design dynamic impacts to perform preliminary design calculations representing actual load design levels used by railroad bridge design specifications. The use of a simple span for loading follows common railroad practice. CyclRR creates a virtual train from user input and passes that train over the bridge. The program develops a bending moment time-history and an associated bending stress time-history to document the effects of the train passage over the virtual bridge. It then performs rainflow cycle-counting analysis on the stress cycles above a user-defined threshold. The Root-Mean-Cube (RMC) stress range and RMC moment range are calculated along with the associated number of cycles. At this point, the process of determining the effects on the bridge spans can be examined analytically. Flexibility in the software is such that the program was used in multiple ways in the research, including simple one-time runs for a specific loading and Monte Carlo simulations with development of the mixed train model. The latter is crucial for railway bridge fatigue analysis.

The fatigue design load is the second product of the research. Current railroad bridge fatigue analysis uses the Cooper E80 design load with an adjusted number of equivalent stress cycles to estimate the effects of actual traffic. This load provides adequate overall design quantities for moments and forces but does not act like actual railway loads, a key difference for a fatigue load. The proposed fatigue load was developed using data developed by CyclRR and corroborates previous work including field testing (*3*). The proposed fatigue load resembles a railcar and provides moment magnitudes and cyclic behavior such that it can serve as a reference for developing a fatigue design load and quantifiable rating system for fatigue. The authors believe the proposed fatigue load model can also be used to evaluate existing simple-span girder railway bridges with an estimated initial fatigue life expenditure, which is typically negligible for historical actual loads before 1965.

In conjunction with the fatigue load development, sample calculation examples are provided along with the tables necessary to perform those calculations. Application of the fatigue life estimation method was included with this report to demonstrate the ability to perform different types of calculations from the results of CyclRR. The key to reliable results from any calculations for fatigue form railway trains is knowing the number of cycles per train on the particular span length that is under investigation.

CONCEPT AND INNOVATION

Railway bridge fatigue is an issue that needs an examination to understand the status of the entire inventory in general instead of examining each individual bridge in detail. The desire is to create tools and methods that focus on those general design features that may be predisposed to accumulating active damaging fatigue cycles. In a general sense, railway operations need a filter to facilitate identification of affected spans. The data to provide for these calculations have not been available previously.

With this new tool in hand, the railroad bridge engineer responsible for a company's inventory has means to more properly plan for future repairs, strengthening, and/or replacements. It can aid by indicating critical areas for inspection. It also provides public agencies owning or operating rail lines with objective measures of fatigue damage without the need for a complex evaluation Passenger trains and transit trains are "unit" trains in their own right, while their weights are such that little fatigue life is exhausted by them, but many of these agencies host freight train services on their lines for which they may be responsible. This tool assists them in assessing their needs. of a chronological age measure. Railway bridge fatigue with its current evaluation tools allows for considerable subjectivity in developing an answer. The results can be quite divergent from different engineers. Having this methodology provides for a consistent unbiased evaluation.

The behavior of the railway loading as a string of separate loads (i.e., a train) traversing a bridge is fundamentally different from highway vehicles with a single vehicle with a small number of axles. Accounting for the fatigue cycles from a train is more complicated given that the effects of the train on a short bridge is markedly different than for the same train on a long bridge. Railway bridge design criteria have been subjected to a series of different design load level magnitudes and impact allowances over the years, which affect the constructed bridge properties.

The concept is to provide data and define the missing process in the current methodology enabling engineers to perform the higher-level calculations to investigate bridge inventories Computer software able to model the loading effects is the best method for providing the data. The calculations are straightforward and no additional theory needs to be developed. The software, named CyclRR, provides the opportunity to develop a train and virtually pass the train over the bridge while capturing the time-history of the load on the bridge. The software virtually develops the bridge span section properties based upon design information commonly used in North America. Given the design method, the calculation of section properties is simple with an estimate of dead load, but this is acceptable as fatigue calculations in general are estimates. The magnitude of the railcar loads compared to bridge dead loads is such that the estimation of dead load does not create substantial issues in estimating section properties.

CyclRR provides innovation by generating novel and concise data. This data allows further analysis used in development of a new and more accurate design load for fatigue. Railway bridge design is hampered by the lack of a fatigue design load specifically developed for fatigue separate from the Cooper E Load used for strength design. As mentioned, fatigue in rail structures depends upon load pattern and span length, neither of which are considered when evaluating Cooper E Loads. CyclRR has additional uses as a research tool and can be modified to provide full time monitoring of fatigue on a real-time basis, Monte Carlo simulations and "what-if" scenarios, along with useful calculation for individual bridges in need of more detailed scrutiny.

In summary, fatigue design for railway bridges requires additional background information and data to reduce subjectivity and provide a common basis for evaluation. The proposed fatigue load in this research provides a common basis for both design and evaluation of railway bridges for fatigue. Moreover, the data generated by CyclRR to develop this fatigue load provides the necessary information to examine railway bridges on a more objective and rational basis.

.

INVESTIGATION

INTRODUCTION

The average age of steel railway bridges is a concern to the industry. Recent reporting indicated that the percentage of steel bridges over 100 years old is above 50 percent while approximately 2 percent of the steel bridges have passed 120 years (*4*). The period between 1900 and 1920 was a very busy period for replacement of existing bridges due to heavier axle loads and the final period of major construction of new railway lines. These bridges are still performing in terms of overall load capacity, but the bridges are exposed to accumulative load cycles, slowly expending the fatigue life of the bridge span. Fatigue life prediction is well defined but fatigue loading that drives stress cycles is not particularly for railway bridges. Defining a proper railway bridge fatigue load is becoming more critical as these bridges continue to age. This research describes efforts to build additional knowledge into determination of the fatigue life of both existing and future steel railway bridge beams and girders.

Railroad bridge fatigue design and rating has paralleled highway bridge practice since the late 1970's with publication of research results demonstrating tensile cyclic stress range (repeated loading and unloading) as the main factor for fatigue behavior (*5*). Additional testing provided relevant fatigue data for riveted members (*6, 7*). The inclusion of fatigue capacity for riveted steel details was critical for railroad bridges given the age statistics of the existing steel bridge inventory.

Fatigue design is a part of MRE 15 the design recommendations published by the American Railway Engineering and Maintenance-of-Way Association (AREMA) in the Manual for Railway Engineering (MRE). Chapter 15 Steel Structures (MRE 15) (*1*), addresses design and rating of steel bridges. Current recommendations require design stress range from the Cooper E Load to be below the threshold constant-amplitude stress range limits listed with the fatigue detail categories in Article 15-1.3.13. In general, current fatigue detail categories in conjunction with improved fabrication practices recommended in MRE 15 achieve the goal of high fatigue resistance in newly fabricated steel bridge spans.

The criteria are similar to highway bridge design for the details and allowable stress ranges. Adjusted cycles for fatigue design are based upon Cooper E Load, because its characteristics are not representative of actual load patterns. Cooper E Load is adequate for maximum moment magnitude, however its axle loads and spacings do not resemble a load that will approximate a railcar fleet and its cyclic loading actions on a typical bridge span over its life in terms of equivalent fatigue damage. This affects the Engineer's ability to reconcile the number of cycles a train may produce in a typical loading event when moving across the span.

Fatigue analysis of railway bridge spans necessitates a method to generalize conditions. Attempting to manage all of the bridge inventory on an individual basis is not feasible with large bridge inventories. Railway bridge asset management lends itself to generalization. In contrast to highway structures, deflection requirements force section sizes and live load dominates the overall maximum bending moment which minimizes errors in dead load estimation. The result is that section sizes for any given design can be reasonably estimated with the results applicable to any span of a similar design.

Fatigue life calculations by their derivation and definition can only be considered estimates (*8*). An exact fatigue life calculation will never be practical, due to estimates of the loading and not having a molecular-level knowledge of the material under load. This doesn't include potential issues with fabrication. The method described in this paper provides an important improvement to the tools for fatigue life estimation for any given railway bridge design specification using actual railcar equipment and a virtual analysis for moment time-histories and rainflow fatigue cycle counting. For this research, the AREMA design configurations are examined for fatigue under typical modern trains along with common and frequently used historical trains.

The approach was to create a purpose-built software algorithm that calculates typical bridge section properties according to the criteria in use at the time of design and produces both a bending moment and stress time-history of a virtual version of a train. The program performs a rainflow analysis on the stress range time-history providing an effective root-mean-cube bending moment range and stress range along with the associated number of cycles. The program was used to assist in creation of a proposed fatigue load for railway bridges.

Fatigue life evaluation for steel railway bridges ultimately requires a comparison of the estimated life of the bridge in terms of the number of trains. This is because the number of applicable cycles per train can vary by train type according to span length. Each span and span length will have its actions and reactions under a given train, but the same trains travel over a number of different bridges. The number of trains is the common denominator for discussions between engineering

and operating departments. Fatigue life of railway bridges need to be defined by the number of trains it can carry, and the effects of any train need to be quantified for the different span lengths it may traverse. The four items necessary for railway bridge fatigue life calculations are:

- Effective Root Mean Cube (RMC) Bending Moment Range
- Span properties to calculate the RMC Stress Range from the RMC Bending Moment Range
- Number of cycles per train passage
- The category of the fatigue detail that is being examined

The program for fatigue analysis, named CyclRR develops a bending moment and stress time-history from simulations of actual railway loads and the bridge span section properties, and uses a rainflow algorithm to determine the number of significant cycles from the associated stress ranges. The ultimate result was development of a fatigue load for fatigue design and rating for railway bridges.

The fatigue detail category is also critical (*8*). Rolled beams and bolted connections are more resistant to fatigue cracking than rivets and welded details. Older riveted spans will develop fatigue cracking at lower stress ranges than bolted spans while rolled beams display the most fatigue resistance of the three span types. Results from the fatigue analysis must take fatigue detail category into account when performing these calculations.

BRIDGE SPAN SECTION PROPERTY DEVELOPMENT

A necessary part of the fatigue analysis process is knowing the critical section properties for any bridge span under examination. Railway bridge fatigue for bending members depends entirely upon section modulus, needed for calculation of bending stresses from the actions of the live load and impact. Rather than examining individual spans, the section moduli can be calculated for a series of spans that share the same design criteria. This is demonstrated using the design criteria for past and current railway bridges as published by AREMA and its predecessor.

Prior to the introduction of the original 1906 American Railway Engineering and Maintenance of Way Association (later American Railway Engineering Association, AREA) bridge design recommendations, each railroad maintained their own bridge design specifications. The loading diagrams were similar to the Cooper E Load with minor differences in axle spacings and load magnitudes. Steel was in general use in the 1890s with universal acceptance of the material by 1900. Allowable stresses for steel and impact allowances could vary more than the particulars of the design loads. Table 1 provides the details of live load design level, the impact formula, and the allowable steel stresses were used for each of the AREA/AREMA design eras.

These design levels were recommendations for bridge design, not design requirements that demanded total adherence. As such, each individual railway could tailor the recommendations to their needs. Greiner (*9*) surveyed the railroads in 1911 for the bridge design levels used. Although Cooper E40 was the minimum recommended live load design level in 1906, many railroads had opted for increases to Cooper E55 and E60 creating a larger overall section size and thereby providing additional load capacity for future use. The increase was driven primarily by the size of the locomotives purchased by the railroads and their axle weights, many of which were already over the 40,000 pounds per axle used in Cooper E40.

TABLE 1. Bridge design eras and criteria for AREA and AREMA

Design Era	Years	Live Load Design Level	Live Load Impact Year	Allowable Stress (ksi)
E40 - 1906	1906-1919	Cooper E40	1906	16.0
E60 - 1920	1920-1934	Cooper E60	1920	16.0
E72 - 1935	1935-1947	Cooper E72	1935	18.0
E72 - 1948	1948-1967	Cooper E72	1948	18.0
E60 - 1968	1968-	Cooper E80	1968	20.0

References for the details of the design recommendations are included (*10 – 14*). The equation(s) for impact were changed in the years shown and were used in design for the time period shown. The current impact equations were placed

in MRE 15 in 1948 with priority for hammer blow impact over rolling impact. In 1968, with steam locomotion fully retired, the priority for impact was placed on the rolling impact equations. Impact for fatigue evaluation is included in MRE 15 at a reduced magnitude for design impact. For the purposes of this research, impact is included only for the calculation of section moduli of the spans under investigation. This is done to leave this research to focus on the development of the methodology.

Regardless of design era, the critical bridge section property needed for fatigue analysis is the section modulus of the supporting beams and girders. Railway steel bridge design follows Allowable Stress Design (ASD) principles in spite of developments of Load Factor Design (LFD), and the Load and Resistance Factor Design (LRFD) adopted in other industries. ASD persists for multiple reasons, but mainly due to deflection requirements that ensure proper track geometry under any loading configuration, either under normal or maximum rating conditions. The moment of inertia of a railway bridge is the limiting criteria under the AREMA design recommendations. The minimum moment of inertia will nearly always provide the required minimum section modulus for the design load.

Estimation of section modulus is calculated simply by combining dead load, live load, and live load impact bending moments and using the allowable stress. For railway bridges on a tangent alignment and simply supported, the preliminary section can be derived from that simple set of calculations. From Table 1, the values of live load and live load impact can be calculated exactly. The dead load is an estimate, but is consistent for any span as each design era used similar practices for decking and ancillary materials. The assertion of high values of live load plus impact to dead load ratio supposes that estimates of dead load can be imprecise yet still provide a relatively accurate overall section size.

Based upon this knowledge, section moduli were calculated for typical spans and span lengths. The live load levels in Table 1 with the appropriate impact formulae were used. The following assumptions were used for dead load:

- Two-girder deck plate girder construction
- Open timber deck construction, assumed 500 plf total deck weight with track
- Steel weight based on formulae (*15*)

The result of the dead load calculation represents a span able to support one track. The calculation for section modulus provides the net section modulus in design required for the given dead load and live load plus impact. Historically, railway bridges, except for spans under approximately 25 feet, used riveted fabrication built from plate elements until the 1960s when welded fabrication became the standard. With the cover plates used in riveted fabrication, section modulus at the quarter points was approximately 75% of the full section (*16, 17*). The cover plates would increase the span to full section modulus between the quarter points and midspan. With the advent of modern welded girders only one piece is used for a flange. Transitions can be made near quarter points for longer girders when economically advantageous. Rolled beams were predominant for spans under 25 feet (*17*).

The major difference between the two fabrication methods is that riveted construction creates a net section with the rivet holes while a typical welded section remains a gross section with no reductions for fastener holes. A gross section was calculated from the riveted sections by assuming a net/gross ratio of 0.85, the common assumption in ASD. Figure 1 displays the net section modulus demand calculated for the design eras demonstrating

- E40-1906 produces the least section modulus demand
- The greatest demand is from E60-1920
- E72-1935 produces slightly greater values on short span lengths
- The section modulus demand for E80-1968 is midway between the two limiting values

The Cooper E80 bending moment being greater than either E40 or E60 is the result of two factors. The magnitudes of live load impact in Figure 2 provide context to the first factor. The second factor is the magnitude of allowable stresses. Earlier design eras used a lower allowable stress than the 1968 value (Table 1). Lower stresses will inherently generate more section modulus demand. The use of 16 ksi results in a section modulus demand that is 1.25 times the section modulus needed for 20 ksi.

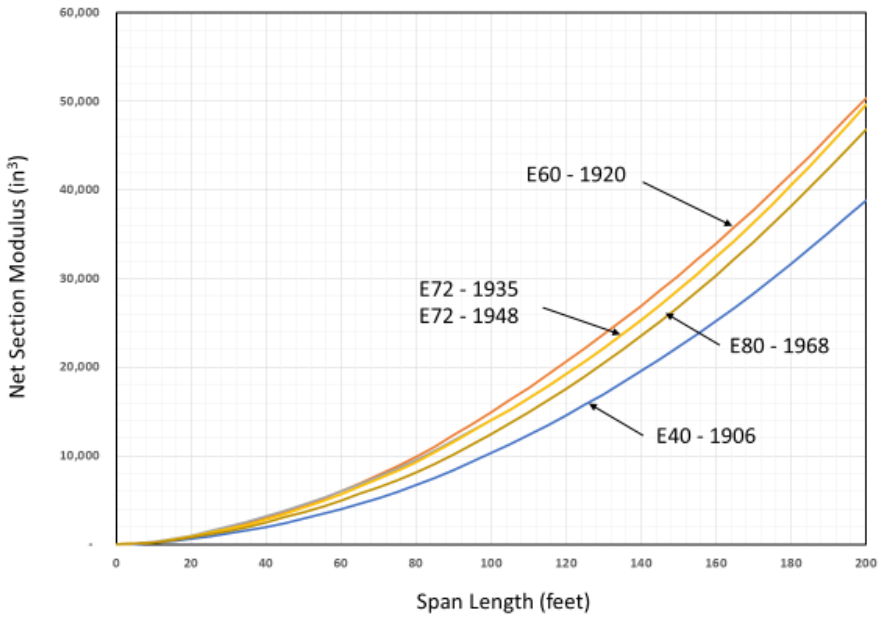


FIGURE 1. Net section modulus demand for the design eras in Table 1

The live load impact displayed in Figure 2 demonstrates the evolution of railway bridge design impact from the original 1906 recommendations through the current 1968 formula (*10-14*). Until 1968, hammer blow impact expected from steam locomotives had been the rule while they were still in service. Testing of steel bridges (*18*) for impact had shown that the early equations for hammer blow impact produced higher impact percentages than were experienced in service. The 1948 impact equations (current) produced more realistic values of design impact. The 1968 switch to rolling impact as the primary equation demonstrates the difference of rolling impact versus the 1948 hammer blow impact. These differences, along with the changes in allowable stress, create the condition where even with a heavier design load, the section size of the span may not be as large as those spans built in earlier design eras. Figure 3 displays additional information when examining gross section of the riveted spans against the as calculated gross section for the welded E80-1968 spans.

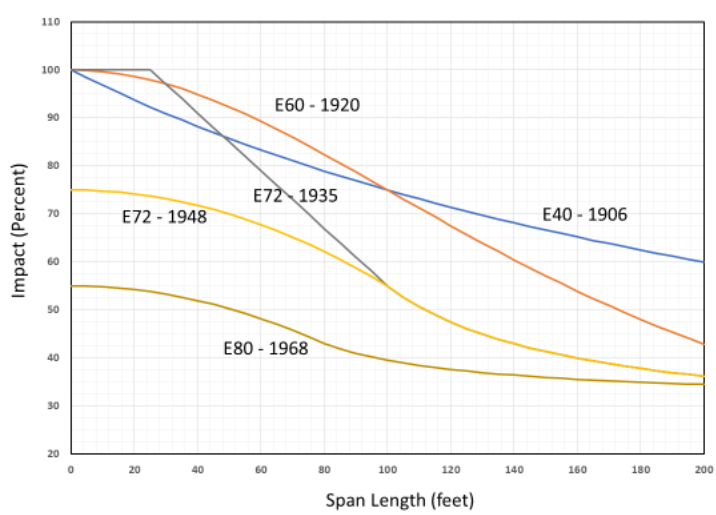


FIGURE 2. Live load impact for the design eras in Table 1

Figure 3 shows that when considering gross section, the E80-1968 designs are the same overall size as E40-1906 designs. The other design eras produce a grouping of their gross section properties. In Chapter 15, deflection is calculated using gross section.

The importance of this is while overall load capacity of a railway girder span depends upon the section modulus, moment of inertia, and the allowable stresses, fatigue resistance is not controlled by allowable stresses. The older design

eras produced similar or larger sections than are currently produced by E80 and 1968 impact. These larger sections, by nature of their larger section modulus, will possess greater fatigue resistance than the current design era depending upon fatigue category.

Validity of the approach still requires discussion. Figure 4 displays the Live Load plus Impact/Dead Load ratio ((LL+I)/DL) for the design eras plotted against the span length. The ratio is extremely large for short spans and decreases as span length increases. Even with the decrease as the span length grows, the ratio of design live load plus impact is substantial through most of the graph. At a 200-foot span length, the values of the ratio become asymptotic to a value of 1.0 for the ratio. Since most girder spans are below 100 feet in length the proportion of section modulus attributed to dead load is still minor in the entire calculation.

As an example, assume an E40-1906 span that is 100 feet long. The ratio of LL+I/DL is approximately 5. This means that dead load represents only 2-3 ksi of the total 16 ksi available from the section. Minor adjustments in dead load for those span lengths have a small overall effect in size of the section, so the assumption of a dead load weight that is approximate is not detrimental to the calculation of section properties. Given the low magnitudes of the dead load stresses on span lengths less than 100 feet, the method of estimating section modulus for a series of span lengths is valid.

Another consideration of Figure 4 is that since live load is dominant compared to dead load for most span lengths using bending members, fatigue is a major concern. Since most of the resistance of the section is devoted to live load, stress ranges due to live load (plus impact) most often are the controlling effect on their expected design fatigue lives.

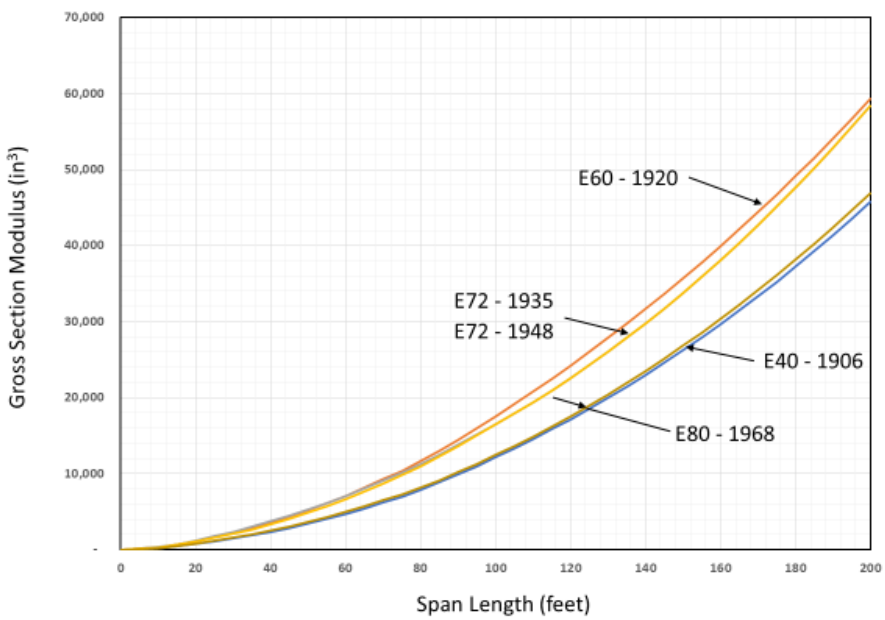


FIGURE 3. Gross section modulus for the design eras in Table 1

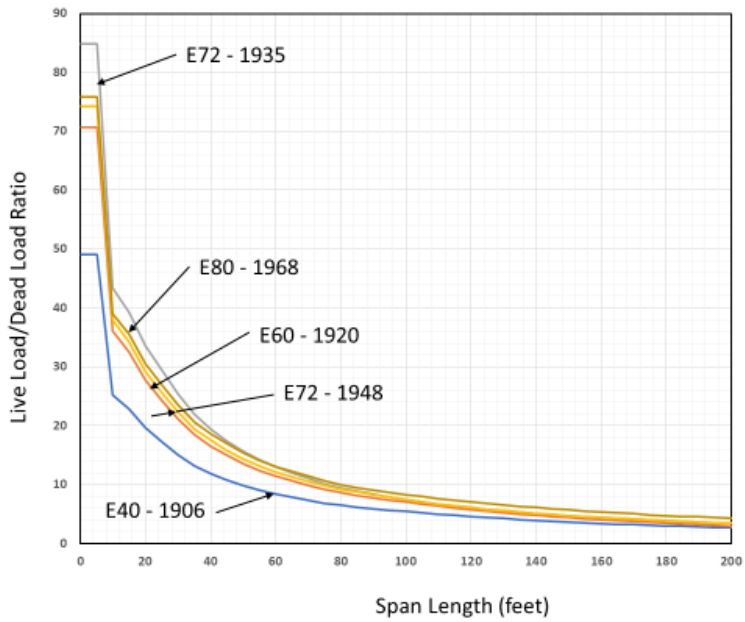


FIGURE 4. (LL+I)/DL ratio versus span length for the design eras

RAILCAR EQUIPMENT ANALYSIS

General Background

Railcar equipment design changed in a similar manner to bridge design load levels although not in lockstep with bridge design load levels. By 1900, all-steel railcar designs (coal hoppers) with 50-ton design capacities were being built along with mandatory use of automatic couplers and air brakes. Typical train weight was transformed from hundreds to thousands of tons. The general features of railcars from those early designs are similar to current railcar equipment with the main difference only in weight capacity and lack of specialization in commodity type. The boxcar and open hopper constituted the majority of the fleet with flatcars, tank cars, and gondolas maintaining a secondary presence.

Although the steam locomotive was in use then, diesel-electric locomotives started in switching and yard service following World War I, with introduction to main line passenger service in the 1930s. The first main line diesel-electric freight locomotives were introduced just before the United States entered World War II. By the end of 1955, except for a few railroads, steam was completely replaced by diesel.

While the change in motive power was a major advancement, allowable railcar equipment weights remained at lower levels. Major changes occurred in the mid-1960s. These changes not only increased weights of railcars significantly; the process of railcar design was simultaneously transformed. The year 1965 serves as the line of demarcation.

Given that line, the loading eras can be placed into two categories; pre- and post-1965. The two loading eras have distinct characteristics beyond the weights of the equipment. Pre-1965 railcar equipment design assumed that generic railcar equipment was satisfactory to transport any load. Post-1965 railcar equipment designs incorporated specialization by commodity or commodity group. Regardless of era, the common railcar is consistent in its configuration. Figure 5 represents the most common railcar configuration with two trucks (bogies) per car. The dimensions used for the structural analysis of railroad loadings are in Figure 5 while Tables 2 and 3 provides details of the railcars and locomotives used in this analysis followed by narrative on the two loading eras.

Although important to the history of rail transportation in the United States, steam locomotives were not included in this research. For steam locomotives, specific knowledge of the locomotives used in service requires detailed knowledge about the axle loads and degree of impact that may have been produced by the locomotives. These are not included in the general work described herein. This is a subject for its own separate study. A convenience for this is to assume for any span length one cycle at maximum design stress per train to represent steam. This is not scientific, but during the period of steam locomotion, capacities of bridges were more concerned with the locomotive than with freight loads and design levels were

driven by locomotive size more than by the railcars. Until such time that more scientific analysis is performed, this provides an expedient method to account for steam locomotives.

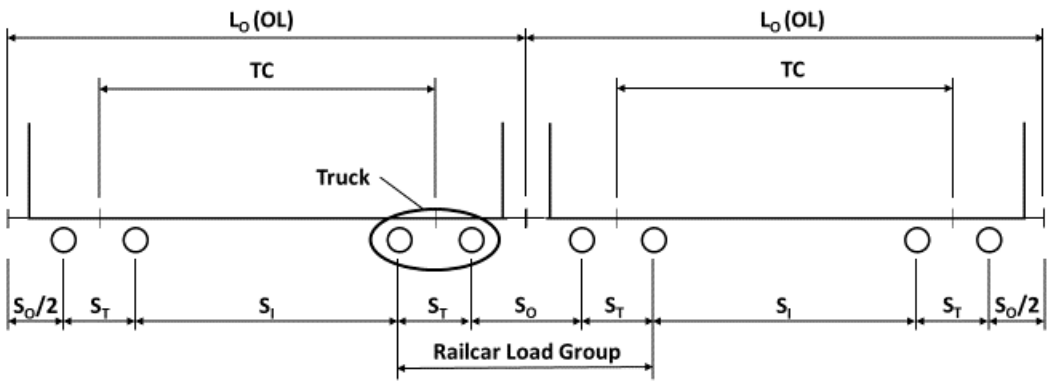


FIGURE 5. Railcar dimensions needed for analysis of bending moments in bridge spans

From Figure 5, the necessary dimensions for analysis are calculated from the available published dimensions. The definitions for Figure 5 and associated equations for loading configurations are:

L_O = Overall (coupled) length of the railcar
OL = Overall (coupled) length of the railcar (published)
n = Number of axles per railcar
GRL = Gross Rail Load; the maximum allowable weight of the railcar
Lt Wt = Light Weight, the empty weight of the railcar
P = Axle load (for either loaded or empty railcar)
P_{GRL} = Axle load of a fully loaded railcar
P_{LW} = Axle load of an empty railcar
TC = Truck Centers, the length between the center pins on the trucks (published)
TW = Truck Wheelbase, the length between the centers of the extreme axles of a truck (Table 2)

$$P_{GRL} = \frac{GRL}{n} \text{ (loaded railcar)} \quad (1)$$

$$P_{LW} = \frac{Lt\,Wt}{n} \text{ (empty railcar)} \quad (2)$$

S_T is the truck axle spacing. Truck wheelbase and truck axle spacing are synonymous for 4-axle railcars using 2-axle trucks. Locomotives and other heavy railcars use 3-axle trucks. Truck wheelbase is the measurement over the outside axles on each end of the truck. For a 3-axle truck, the middle axle is assumed to be centered between the end axles. The number of S_T spacings for any railcar is *n* - 2.

$$S_T = \frac{TW}{\frac{(n-2)}{2}} \quad (3)$$

S_I = Inside axle spacing

$$S_I = TC - \left[S_T \times \frac{n-2}{2}\right] \quad (4)$$

S_O = Outside axle spacing

$$S_O = L_O - S_I - [S_T \times (n-2)] \quad (5)$$

$$S_O = L_O - \left[TC + \left(S_T \times \frac{(n-2)}{2}\right)\right] \quad (6)$$

L_O = Overall length of the railcar; this formula serves as a check on dimensions.

$$L_O = S_I + S_O + [S_T \times (n-2)] \quad (7)$$

Table 2 provides dimensions concerning the nominal load capacity for railcars. Gross Rail Load represents the total weight of the loaded railcars and the truck wheelbase is the minimum wheelbase allowed for a given railcar capacity (*19*).

Pre-1965 Equipment Analysis

Railcar equipment in the period from 1900-1920 was a mixture of pre-1900 construction and the new steel cars quickly populating the fleet. World War I was a nexus for full modernization of the fleet to that time. Issues with operations and equipment availability resulted in the US government assuming control of the railroads for approximately three years during and after World War I. During that time, the United States Railroad Administration (USRA) developed plans for standard locomotives and freight railcars of different types. The railcars designed at that time served as the basis for freight car dimensions until 1965. For pre-1965, the load capacities fell into three categories shown in Table 2.

By the late 1950s and early 1960s, with improvements in materials and design methods, along with creation of larger rail sections and motive power, the railroads wanted to increase the available payload that railcars could handle. This occurred in late 1964 with 1965 being the first full year for larger equipment.

The boxcar (40- and 50-ton capacities) was used for movement of almost everything that wasn't coal or liquid. It was used for grain and mineral bulk shipments in addition to manufactured goods. Perishable traffic used the same basic design. Coal hoppers came in two basic varieties (50 and 70 tons) depending upon the desires of the railroad and the coal company. These types of railcars where the majority of the railcar fleet prior to 1965. The locomotive dimensions shown are for the predominant diesel locomotive of the period. Generally, four of those locomotives would be used on one train.

TABLE 2. Railcar gross weights and truck wheelbases based upon nominal capacity

Railcar Capacity	GRL (lb)	Axles n	Axle Load (lb)	TW (ft)
Pre-1965				
40 Tons	136,000	4	34,000	5.50
50 Tons	169,000	4	42,250	5.50
70 Tons	210,000	4	52,500	5.67
Locomotive	248,000	4	62,000	9.00
Post-1965				
40 Tons	143,000	4	35,750	5.50
50 Tons	177,000	4	44,250	5.50
70 Tons	220,000	4	55,000	5.67
100 Tons	263,000	4	65,750	5.83
110 Tons	286,000	4	71,500	5.83
125 Tons	315,000	4	78,750	6.00
Locomotive	432,000	6	72,000	13.17

TABLE 3. Railcar dimensions for the equipment used in this study

Railcar	Number of Axles	Gross Rail Load (lb)	Axle Loads (k)		Length and Axle Spacings (ft)				Uniform Load (plf)	
			Loaded	Empty	L_O	S_T	S_I	S_O	Loaded	Empty
Pre-1965										
50T Boxcar	4	169,000	42.25	11.25	44.75	5.50	25.50	8.25	3,777	1,006
55T Hopper	4	169,000	42.25	10.00	36.63	5.50	18.50	7.13	4,614	1,092
70T Hopper	4	210,000	52.50	12.50	44.29	5.67	26.00	6.96	4,741	1,129
Locomotive	4	248,000	62.00	-	50.00	9.00	21.00	11.00	4,960	-
Post-1965										
Cement/Sand	4	286,000	71.50	16.50	42.00	5.83	23.58	6.75	6,810	1,571
Unit Coal	4	286,000	71.50	16.50	53.08	5.83	34.67	6.75	5,388	1,243
Oil/Ethanol	4	286,000	71.50	16.50	60.00	5.83	40.67	7.67	4,767	1,100
DDG Hopper	4	286,000	71.50	17.00	69.00	5.83	50.58	6.75	4,145	986
Centerbeam	4	286,000	71.50	15.50	80.58	5.83	54.17	14.75	3,549	769
Long Flat	4	286,000	71.50	20.75	90.00	5.83	60.17	18.17	3,178	922
Auto/TOFC	4	180,000	45.00	15.00	94.00	5.67	60.33	22.33	1,915	638
Locomotive	6	432,000	72.00	-	73.17	6.58	33.08	13.75	5,904	-

As will be discussed in the next section, a unit train is a train of identical cars loaded with identical weights. Although not known as a train type at the time, the use of the boxcars and coal cars in a single train followed "unit train" philosophy. Known more in the post-1965 loading era, their behavior is the same as a unit train.

Post 1965 Equipment Analysis

Post-1965 railcar development has two distinct categories of equipment. General equipment refers to the railcars which conform to the layout in Figure 5. Table 2 provides the pertinent data for weights. This equipment is an extension of the typical railcar layout including the changes prescribed for post-1965 operations. The diversity of the railcar designs is shown in Table 3. In the late 1970s, intermodal railcar equipment went through a major change that made container shipments economical. This transformed the shipment of consumer and manufactured goods and such that it is a major portion of the current traffic mix for many major railway lines. The shipping container is the current boxcar for shipment of consumer and manufactured goods.

General Equipment

Table 3 displays the major change in 1965 which was the increase in capacities and Gross Rail Load (GRL). The increase permitted a nominal load capacity per railcar of 100 tons (263,000 pounds GRL) for free interchange between railroads. The consequences of that change were a major effect on the characteristics of the railcar fleet. The change in the GRL allowed the railcar designer to optimize the size of the railcar to the density and/or size of the commodity the railcar was planned to carry. This resulted in a variety of railcar lengths designed for a limited range of products to be carried in any single car design. Covered hoppers and tank car designs have proliferated with specialization of design by product type. The plain boxcar is still available, but the modern railcar fleet has covered hoppers as the most numerous with tank cars representing the other significant portion of the fleet. Boxcars are still in use for certain products but shipping containers and semi-trailers in intermodal traffic replaced the boxcar for most general shipments.

The Gross Rail Load was again raised in 1995 to allow 110 tons of nominal capacity to its current limit of 286,000 pounds. Changes in equipment design have been subtle. Many railcar designs did not change length dimensions with the increase in weight using other modifications to support the increase in GRL. The current railcar fleet still includes some 50-ton and 70-ton capacity cars but the 110-ton capacity is the norm.

Articulated Intermodal Equipment

The late 1970s saw the development and testing of new railcar equipment for intermodal service. These railcars take advantage of an articulated connection between platforms so that both platforms are supported by one truck (two axles) instead of the traditional four axles grouped at the ends of the railcars. The general layouts of the articulated intermodal cars are shown in Figure 6. The articulated railcars were built for both trailers and containers. At the time of introduction,

trailers were the primary form of intermodal traffic. Because of the ability to stack containers, economies of scale gave the container a major advantage and they are the dominant form of intermodal shipments.

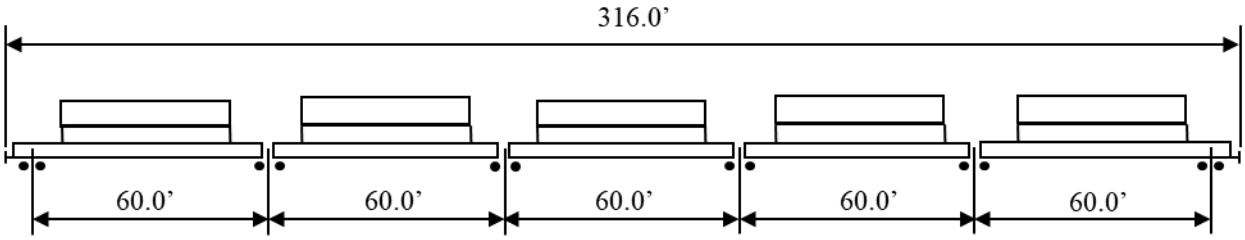


FIGURE 6. General layout of articulated intermodal railcar used in analysis

The length dimensions shown for the articulated railcar in Figure 6 are approximate. Several variations of the articulated railcar have been built. These dimensions represent longer articulated railcars. These are used in the analysis as the car represents longer platforms currently in use and provide an envelope for the different varieties of articulated equipment used in intermodal service.

Articulated equipment is more complicated in determination of axle loads. Articulation between platforms reduced the number of axles carrying the loads on the platforms. To compensate for the reduced number of axles, the interior trucks used for articulated support are based upon the requirements of the 315,000-lb railcar listed in Table 2 to provide flexibility for placement of loaded containers in the same wells with less concern for overloaded axles. While those trucks are necessary for the loads, the end trucks do not require the same load capacity. The end trucks have a capacity conforming to the 70-Ton railcar from Table 2. This is all the capacity needed for the end platforms since those trucks are supporting two containers instead of four for the interior trucks. The axle loads can reach their full capacity as listed in Table 2 for the railcar configuration, but can be below that level. For the purposes of this research, the axle loads were assumed to be at maximum allowable weight.

TYPES OF TRAINS FOR FATIGUE EVALUATION

Since the life of a span needs to be assessed in the number and type of trains that may traverse it, the arrangement of the trains is critical. Three basic train types are developed for analysis in consideration of the railcar equipment. The train types reflect typical railroad operations for each era. These are the unit train, the mixed (manifest) train, and the intermodal train. All trains were combined with diesel locomotives. For trains in the pre-1965 era, four 4-axle locomotives were used for each train and three 6-axle locomotives were used with each train in the post-1965 era.

Unit Train

The unit train has been previously discussed in the context of the pre-1965 trains and the use of the 40-foot boxcar. For pre-1965, the notion of the unit train, all cars with identical dimensions and identical weights, were happenstance given the available equipment. This was especially the case for coal shipments while perishable traffic and agricultural products were also examples.

In the post-1965 environment, the unit train was deliberately marketed in order to reduce operating costs. The general increase in shipments due to economic activity was another factor pushing the use of the unit train. The Powder River Basin for coal in the 1970s was a major source for unit trains but grain shipments and other bulk products quickly moved to the concept. The current popular unit trains are coal, grains, ethanol, petroleum, potash, and frac sand. Other commodities are transported in unit trains on specific routes advantageous to the railroads and the shippers and receivers responsible for the traffic. Bulk commodities are popular for unit train movements.

Articulated Intermodal Train

Intermodal shipments (trailers and containers) were moved to a unit train basis after their introduction due to the overall volume of shipments and the need for specialized terminal facilities for loading and unloading. Semi-trailers were the

largest segment of intermodal traffic from the 1960s through the late 1980s. While the beginnings of intermodal trains were in the pre-1965 loading era, it is associated with post-1965 traffic since the late 1960s were when this traffic burgeoned.

Trailers were transported on TOFC (Trailer on Flat Car) flat cars (similar to the autorack platform used for shipping automobiles) and then onto articulated equipment developed for trailers. The TOFC flat cars can accommodate two standard highway semi-trailers. With articulated equipment, container shipments found economies of scale and this resulted in articulated container equipment comprising the major proportion of intermodal shipments Intermodal traffic is best analyzed as unit trains on the autorack/TOFC car and the articulated equipment.

The analysis of articulated equipment for fatigue is more complex. Standard railcars follow the dimensional structure shown in Figure 5. Articulated equipment includes two separate patterns which complicates analysis. Axle weights can be variable but analysis in this research used two separate weights based upon either being interior or end axles on the railcars. Further information on articulated equipment is included.

Mixed (Manifest) Train

The mixed train has similar meanings in operation and engineering. Also known as a manifest train, this train is a collection of disparate railcars of small blocks of shipment gathered into one train. The railcars are a mixture of types which may be either loaded or empty. Loads do not always reach full car capacity. Trains from these cars create a stochastic process in loading a bridge and the moment time-history resulting from this has no regular pattern. The analysis for bending moments and cycles must be calculated with the loading pattern matching the actual train creating a virtual time history. The time history is then analyzed by the rainflow method for cycles.

Heavy Load and Other Special Train Types

Heavy loads, such as oversize electrical transformers, large pressure vessels, etc. constitute a special train where the movement is made under special conditions. These occur, but are infrequent and do not represent typical traffic experienced on a daily basis. Occasional occurrences such as these have a negligible effect on fatigue life and can be dismissed in consideration of fatigue loading. They will not be included for further discussion in this document.

FUNDAMENTAL BEHAVIOR OF RAILWAY LOADINGS

The behavior of railcar loadings for fatigue on a simply supported span has previously been studied on a unit-train basis (*20*). The train types in this analysis include more than unit trains but the principles learned from the study of unit trains are applicable in a general sense. As has been discussed, the evaluation of the fatigue stresses is more complicated than a simple evaluation of the maximum applied live load plus impact. The nature of a train crossing a bridge produces different results for its loads depending on a small number of conditions:

- the magnitude of the axle loads
- the spacings of axles related to the span the axles are loading
- the length of any given span and its relationship to the length of the railcar loading it

Varying these parameters, a bending moment time-history can be created and an effective moment-range histogram can be calculated for any configuration of train. The third item requires an examination of the railcar dimensions shown in Figure 5 and an explanation of loading configurations based upon the ratio of L_S, the span length, divided by L_O, the overall or "coupled" length of the railcar (L_S/L_O). A value of $L_S/L_O < 1.0$ indicate loading by no more than one equivalent railcar at any time whereas $L_S/L_O > 1.0$ indicates that the span can simultaneously carry loads from more than one railcar. The manner that a train loads a bridge can be broken down into two general categories of loading segments. These are based upon the dimension S_I; either a value less than or greater than that dimension. The dimensions described in Figure 5 provide guidance for the following discussion.

Loading Segments

$L_S < S_I$

This group of span lengths also includes those less than S_T and S_O, along with most combinations of the two that may fit on a given span length. This group includes span lengths from 0 to approximately 60 feet according to Table 3. This behavior depends upon the dimensions of the railcar. The axle loads at the ends of the rail cars are tied to the coupled length and the two adjacent trucks comprise the complete Railcar Load Group shown in Figure 5. Generally, the length of the load group, $S_O + (n-2) \times S_T$ for any railcar, is less than S_I. In MRE 15, the load group is graphically represented by the Alternate Load (Article 15-1.3.4). This set of span lengths also includes other subsets which are not critical for this discussion, but can play a part in cycle generation of non-obvious cycles. Figure 7 displays the features of bending moment over time for this set of loading segments, showing the time history for five railcars cross the bridge where each truck produces a separate cycle, or two cycles per railcar. This loading pattern is binary; either axles are or are not on the span and this occurs throughout the train passage.

For the example in Figure 7, two axles could fit on the span at any one time. This resulted in the span being loaded by one truck at a time. Other example span lengths will occur where only one axle may fit, creating one cycle per axle, or the entire load group fits onto the span length creating one cycle per railcar. In these cases, S_I is sufficiently long relative to L_S that the live load bending moment returns to zero before the next set of axles starts to load the span. For these conditions, the number of potential cycles will be high. For the condition of Figure 7, it is two cycles per railcar. This condition is simple to calculate in that the moment range per cycle is simply the maximum moment due to live load plus impact for the set of loads that are on the span. The same effect also results for certain truss members supporting short segments (e.g. stringers, floorbeams, and floorbeam hangers). The critical relationship is the span length compared to the S_I railcar dimension.

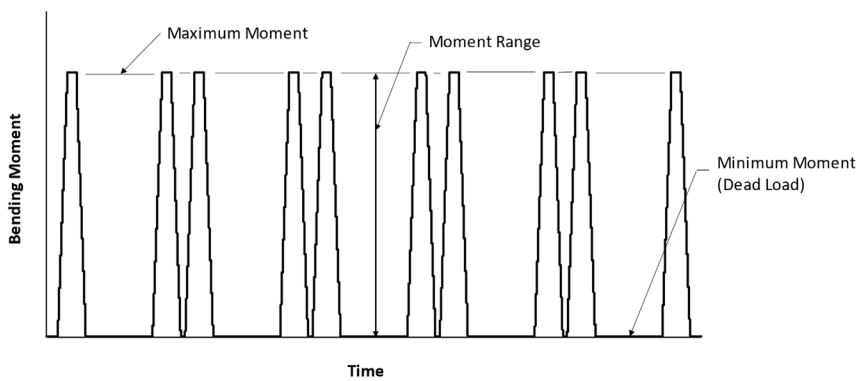


FIGURE 7. Bending moment over time for span lengths where $L_S < S_I$.

$L_S > S_I$

Once span lengths exceed S_I in length, cycling of the bending moments undergoes a transformation. Table 3 shows that this behavior starts at a span length of roughly 18 feet for the shortest car and 60 feet for long railcars. Variation in bending moment will occur depending upon the location of the axles, but the span is not completely unloaded to the dead load condition during a train passage that occurs for the shorter spans where $L_S < S_I$. For this loading condition, live-load moment range requires calculating both the maximum and minimum bending moments due to the live load plus impact not returning to zero until the train has completely passed over the span. Figure 8 displays an example condition for bending moment where $L_S > S_I$. The figure displays five railcars crossing a span, the same number of railcars used in Figure 7

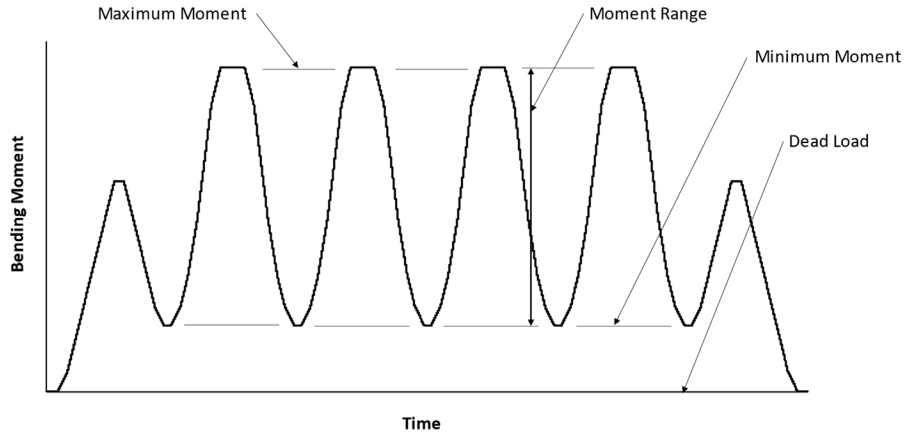


FIGURE 8. Bending moment over time for span lengths where $L_S > S_I$.

In this case, the Engineer must determine the magnitude of the moment range and examine it to understand the basic behavior of a unit train load on a longer span length. The magnitude of the moment range varies with span length and location on the span. Its value requires solution in addition to the overall maximum moment.

Moment Range

Moment range is the algebraic difference between the maximum and minimum moments due to live load plus impact. In the example of Figure 7, $L_S<S_I$, the maximum moment range per cycle is the maximum moment (M_{MAX}) generated by the axles on the span. Figure 9 displays the maximum and minimum bending moment envelopes for four example railcars on a span length exactly four times the length of the railcar ($L_S = 4L_O$, $L_S >> S_I$). The x-axis of the figure displays the quarter points of the span referenced to both railcar and span length.

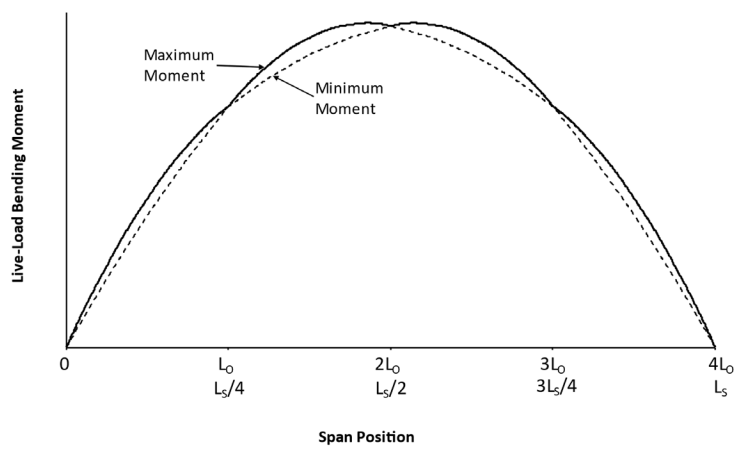


FIGURE 9. Maximum and minimum bending moment envelope for a span loaded by four railcars where $L_S = 4L_O$.

Figure 10 displays the maximum moment range across the span from the maximum and minimum bending moment envelope shown in Figure 9. Figure 10 displays unique features for moment range. This moment range pattern is only displayed when values of L_S/L_O are integers. The pattern displays the absolute maximum moment range (M_{RAM}) at its peak while showing that the maximum and minimum moments are equal at the quarter points and midspan; i.e. a location of constant moment under moving loads. Not coincidentally, the number of times the pattern occurs is equal to the value of the integer in the ratio L_S/L_O. For any railcar design, a unique value of M_{RAM} exists. M_{RAM} only exists at certain points, while most other locations on the span will have a maximum moment range less than M_{RAM}.

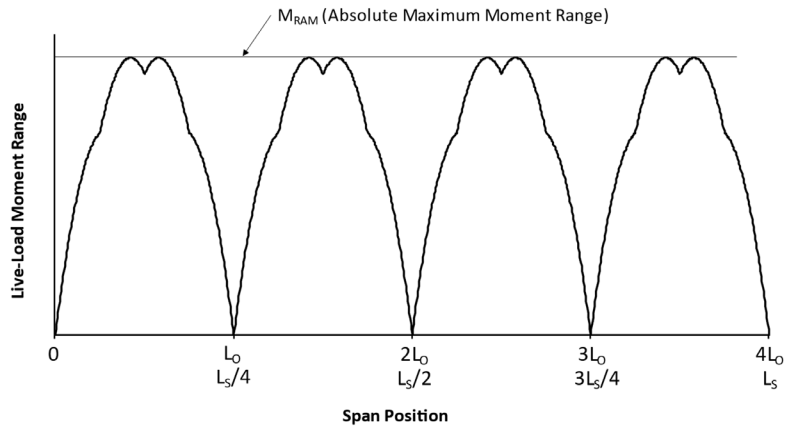


FIGURE 10. Moment range for the span loaded in Figure 9.

M_{RAM} represents the maximum moment range that can be generated by a single railcar in a unit train. A set of loads on a span (whether a portion of one railcar or multiple railcars) will have a maximum bending moment (M_{MAX}) which also must be considered in the fatigue calculations. M_{RAM}, however, provides the guidance for knowing what maximum moment range is expected from a single car in a unit train. Its stress range magnitude is useful to determine when fatigue cycles are being accumulated on the basis of one cycle per railcar or one cycle per train on a span. For values of L_S/L_O not equal to an integer value,

The occurrence of the maximum and minimum bending moment being equal (moment range = 0) can be demonstrated as an intrinsic function of the influence lines for the span and an integer value for the ratio of L_S/L_O. Given that the moment range reaches an absolute maximum value (M_{RAM}) on the same span length where null points are observed indicates that railcars and the bridge span are resonant when the ratio of L_S/L_O is an integer value.

The values for moment range where L_S/L_O is not an integer, by far the more common occurrence, is between zero and M_{RAM}. As an expedient initial check of fatigue susceptibility, M_{RAM} can be calculated by formula for any railcar that conforms to the basic layout shown in Figure 5. The formula for absolute maximum moment range is:

$$M_{RAM} = \frac{nP}{4}\left[S_I - S_O + \frac{S_O^2}{L_O}\right] \qquad (8)$$

For this case of the integer value of L_S/L_O (Figure 10) and the maximum moment range of the railcar equal to M_{RAM}, the location of M_{RAM} always occurs at a distance $S_O/2$ away from the center point of the M_{RAM} curve. Equation 8 uses dimensions of the railcar without regard to span length. The formula represents the maximum potential energy of the railcar to develop a bending moment range; the difference between the maximum and minimum bending moments of the railcar. Because the formula depends upon car dimensions, each set of car dimensions will have a unique value of M_{RAM}. At that same time the lengths at which M_{RAM} occurs are a function of the railcar length.

A comparison of Figures 9 and 10 also show that for the case of $L_S/L_O = 4$, M_{RAM} is small in comparison to M_{MAX}, the overall maximum bending moment due to the live load. For lesser integer values of L_S/L_O, M_{RAM} can be a significant portion of the maximum bending moment due to the live load when L_S/L_O is equal to 1 or 2. This demonstrates that the moment range due to any one particular railcar in the train may or may not create an active fatigue cycle on a bridge. That depends upon L_S/L_O for that particular railcar.

When L_S/L_O increases, the section size required to carry the maximum design moment gets so large that the moment range due to one railcar is insufficient to create an active stress range of concern. The consequence of this evaluation is that a unit train will only create active cycles for each railcar over a finite range of span lengths. As L_S (and L_S/L_O) further increases Tc the section size will increase to the point that M_{MAX} will be insufficient to generate any fatigue damage.

Load Group Behavior

General Railcar Load Group

An examination of the railcar load group for hopper cars in Figure 11 provides a graphical representation of bending moment behavior. The standard load group for both open and covered hopper railcars used standardized dimensions. Those include an S_T distance of 5 feet, 10 inches and S_O distance of 6 feet, 9 inches. While not used for all hoppers, its use is common among railcars of many different lengths. Figure 10 displays maximum moments and moment ranges for the three railcars in Table 5 that have those dimensions; the Cement/Sand car, the Unit Coal car, and the DDG Hopper car.

Figure 11 displays the maximum moments (M_{MAX}) and absolute maximum moment range (M_{RAM}) for the three railcars. In the range of span lengths from 0 to 60 feet, the load group of four axles provides the same maximum bending moment for all three railcars. Just beyond 60 feet, M_{MAX} for the cement/sand car diverts from that line since at that span length, axles from the adjacent cars contribute to a greater maximum moment beyond the four-axle group. At that point the load includes axles from the opposite end of the railcar, across S_I. By 84 feet of span length, two complete cement/sand railcars are constantly loading the span. The same phenomenon occurs for the other cars at the appropriate lengths.

For moment range, Figure 11 shows that maximum moment and moment range are the same value until the span length reaches 23.58 feet for the cement/sand railcar. Beyond that length the minimum live-load moment is no longer zero, and separate values occur for maximum moment and moment range. Moment range on this figure is shown at its absolute maximum (M_{RAM}) for each railcar, but the actual value of moment range on any span length will vary according to its position on the span, as shown for the example in Figure 9.

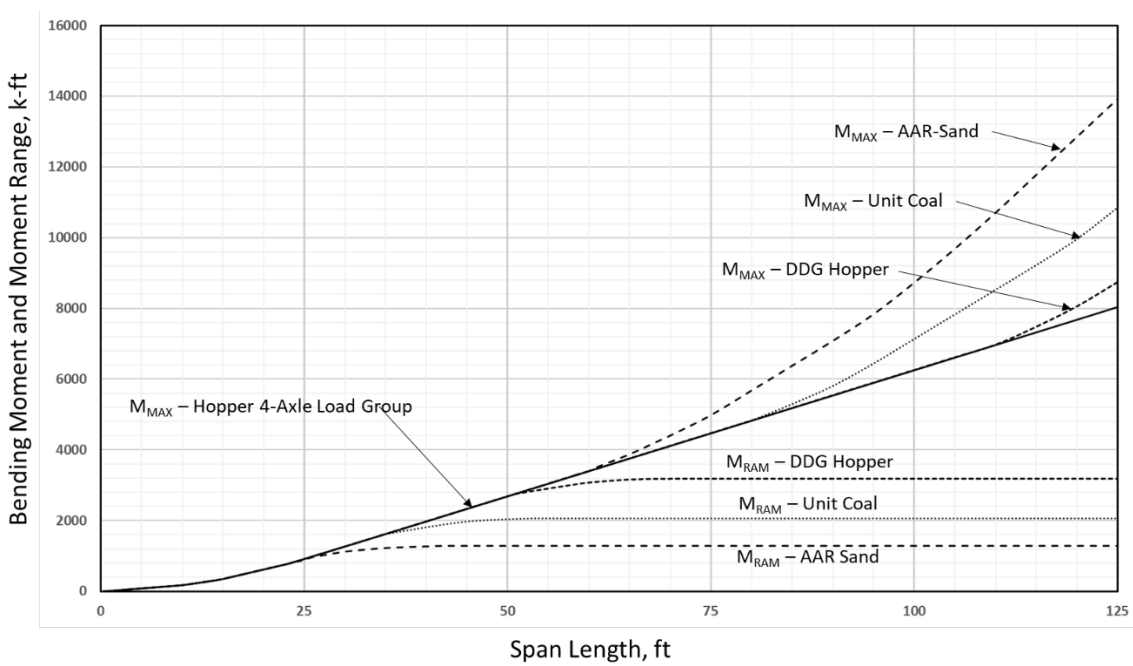


FIGURE 11. Bending moments and moment ranges for railcars using the Hopper Load Group

Articulated Intermodal Load Group

Articulated patterns are similar to the general behavior, but an examination is worthwhile for some fundamental behavior of trucks and load groups including some effect of axle loads. Figure 12 displays the bending moment behavior for articulated equipment based upon full axle loads. The two-axle load group has axle loads of 78,750 pounds while the four-axle group has axle loads of 55,000 pounds. The two-axle group controls on very short spans while the four-axle group becomes dominant once the fourth axle is on the span and contributing to bending moment. This is typical behavior in how moments are generated by load groups. The figure demonstrates that the bending moment curve by span length is a collection of linear segments versus a true curve. This is typical no matter how many axles are loading the span.

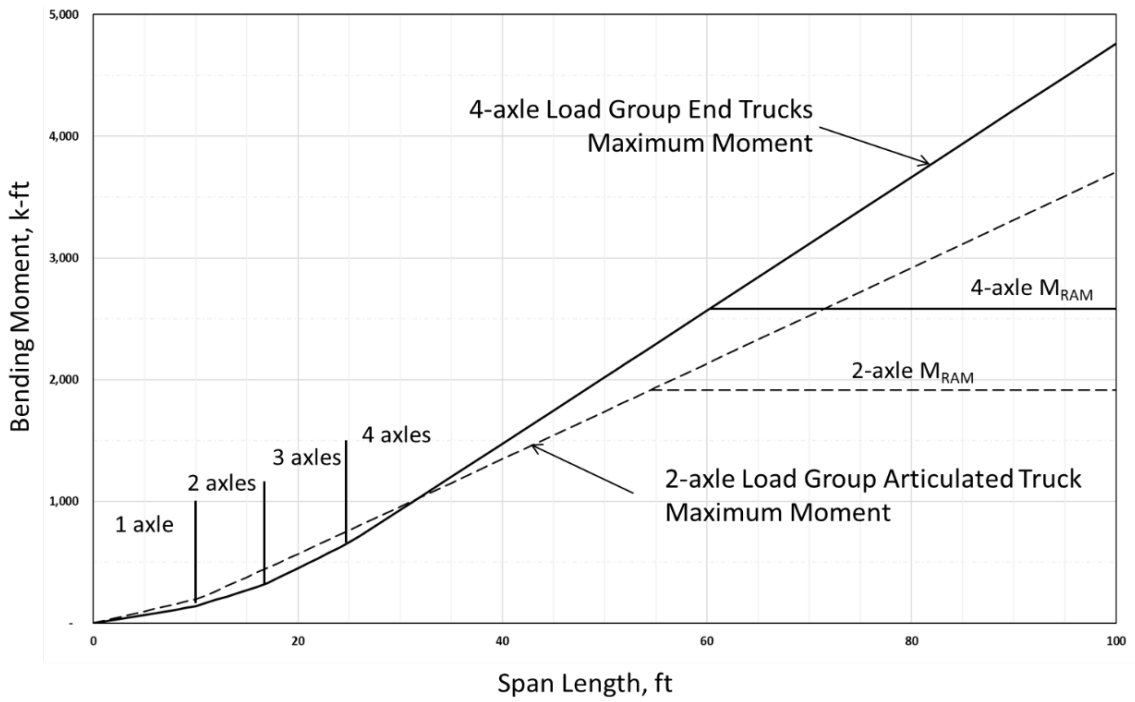


FIGURE 12. Bending moment versus span length for articulated railcars.

FATIGUE ANALYSIS

General Discussion

The research so far has discussed the equipment used and span sizing and development. One of the four items listed as necessary data for fatigue analysis was the knowledge of the fatigue category so that a fatigue life can be calculated in terms of the number of available cycles. For modern construction, either welding or high-strength bolts are used, generally in conjunction with rolled shapes for some members. This construction usually results in Categories A, B, or C'. If the design is carefully detailed, Category C' details can be excluded from the fabrication. For riveted construction, rivets in a specific detail are generally classified into Category D for initial or first cracking. However, due to the inherent internal redundancy of riveted built-up members, cracking in one component does not result in complete member failure. Therefore, for evaluation, the overall life of a riveted structure is based upon Category C in some specifications (*6*). It is worth mentioning that although punched holes may result in fatigue resistance less than Category D, it does not affect development the load model since a conservative baseline design was used (Cooper E40-1906) in combination with a low truncation level (i.e., 3 ksi).

Rivets also display fatigue characteristics under variable amplitude loading of having a fatigue limit of 6 ksi. The 6 ksi limit is known as the Variable Amplitude Fatigue Limit (VAFL), which is similar to the Constant Amplitude Fatigue Limit (CAFL). These fatigue limits are such that if none of the active fatigue cycles exceed the VAFL, the structure will not develop fatigue cracks. If only a small portion of the cycles exceed these limits (approximately one in one 10,000 cycles), fatigue crack initiation is assumed to occur and the stress range cycles below the fatigue limits are assumed to be accumulative (i.e., result in crack propagation). In general, stress ranges of approximately ¼ to ½ of the VAFL are included for counting against the life of the detail or structure.

For new designs using Categories A and B, the CAFL values are 24 ksi and 16 ksi respectively. For all but the shortest of spans, those levels of stresses and stress ranges are rarely seen on railroad bridges that support loads in bending. That level of stress is reserved for movements of very heavy loads moving under special operating conditions. The deflection requirements in design preclude this stress level on an everyday basis.

For evaluation of existing structures, especially older riveted structures, stress ranges above 6 ksi are common on an array of span lengths. The VAFL for riveted connections is 6 ksi for AREMA-based evaluations. Steam or heavy diesel

locomotives produce stress ranges above 6 ksi and the frequency of the applied stress ranges is such that initiation of fatigue cracking at the rivet holes is assumed to occur (i.e., the fatigue limit exceedance rate is far greater than 1 in 10,000 since every train will included a locomotive.). With that, a bending moment and stress time-history needs to include stress ranges sufficiently below the VAFL to account for all cycles that may contribute to initiation of the crack. This is usually assumed to be ¼ to one-half of the value of either the CAFL or VAFL. Such a truncation or cutoff is necessary (as discuss below) and was incorporated for this analysis. The tables displaying the RMC bending moment ranges and the number of cycles were based upon ignoring all stress ranges less than 3.0 ksi calculated for Cooper E40-1906 designs. The use of Cooper E40-1906 as the design basis ensured capturing all damaging cycles that for all future designs and is conservative. It is important to note that Cooper E40-1906 is the base line design while the actual fatigue analysis performed herein used modern loading.

Computer Algorithm for Data Generation

Data generation took the concept of estimating section modulus (assuming the Cooper E40-1906 design level) for the bending members and creating a virtual train to pass over any span length. The bending moments are calculated at short intervals moving the loads over the span length and recording the bending moment time history. With the section modulus available the stresses are easily calculated so the stress time-history is available. From that, rainflow analysis counts the cycles, eliminating less cycles less than a given truncation stress range of 3ksi.

The program, called CyclRR, uses the AREA/AREMA design criteria to generate girder section size, and various Cooper E loadings can be specified as needed. Any of the five impact regimes can also be selected. The program can be modified to use any design load and impact making it useful for proprietary design loads. A practical maximum span length is 200 feet, close to current maximum bending member length in railroad service. The analysis is limited to simply supported spans, the standard for railway bridges.

CyclRR has a library of railcars available and new cars can be added. The program calculates the necessary dimensions shown in Figure 5, creating the series of loads into a train. Typical influence line analysis is used to calculate the bending moments. Any point on the span can be examined and multiple points can be calculated in a single operation.

Fatigue Load Development and Monte Carlo Simulation for Mixed Train Analysis

As has been well discussed in the literature, a variable-amplitude stress-range (or moment-range) spectrum can be represented by an effective constant-amplitude stress range equal to the cube root of the mean cube (RMC) of all stress ranges (*21*). In highway bridges, this RMC concept has been applied to the variable-amplitude load spectrum obtained from weigh-in-motion data to determine a single equivalent truck capable of producing the same cumulative fatigue damage (stress/*moment range and number of cycles*) as the variable series of trucks. The resulting truck is referred to as the "fatigue truck" in design and considerably simplifies fatigue evaluation by representing the variety of trucks of different weights and types found in actual traffic (*22*).

It is important to recognize that the effective "fatigue truck" is only a useful concept for the finite-life fatigue design approach, (i.e., design for a specific number of cycles using the S-N curves). Generally speaking, older railway bridges will likely fall into the finite life category as they were not designed for the fatigue limit state and because modern equipment is heavier than considered in the original design. This creates the need for a simple load model for fatigue evaluation. Also, the percentage of exceedance of either the CAFL or VAFL of a detail for railway bridges has been found in this study to be sufficiently large (greater than 1%) to assume a finite-life for the structure. Given these conditions, the "fatigue truck" concept can be extended to development of a "fatigue train."

One major drawback of the finite life approach is that an accurate estimate of the effective stress range and the number of cycles expected over the design life of the structure must be known to estimate remaining fatigue life. Fortunately, most railroad owners can make a reasonable estimate of the number of trains per day that may have crossed the bridge during its life. The variety of trains found on North American railroads today, in terms of magnitude and consist, is included in the proposed fatigue load model. As seen in Table 2, early equipment was not comparable in weight to current equipment and likely did not consume much of the fatigue life of the bridge. This is especially true for longer bridges. For example, pre-1965 equipment is a type of lighter equipment that had an effect on span lengths less than 50 feet long. Generally, speaking however, equipment, in the post-1965 era can be shown to have produced most of the fatigue damage on typical railway girder spans.

Development of the Fatigue Load Models

Ideally, the consist of every train that has ever crossed a given bridge would have been recorded and subsequently available determine how much of the fatigue life has been consumed over the service life. Each such train would be "run" across the bridge analytically, and the effective fatigue damage combined. Obviously, this is not practical for a number of reasons, one of which is that the data in that level of detail from historical consists no longer exists.

While historical data is rare, it is sensible to assume that each train that crosses the bridge consumes some portion of the available fatigue life. Further, as this train continues along a line and crosses girders of different span and design, the train itself does not change. However, the number and the magnitude of the stress range (or moment range) cycles produced in each span at various locations (0.25L, 0.375L, 0.50L) will be different.

The train types described in this research can all have fatigue load models developed for them. The unit train is simple, with all railcars identical in length dimensions and axle weights. Only one run over any span length is needed for them since they will produce the same action for each passage over any bridge. The assumptions used for the intermodal trains in this research are the same, although complicated by having two separate loading groups to include in development of an RMC moment range and number of cycles. The third train model is the mixed train, which provides random application of loaded and empty railcars, grouped into a train in a random fashion. The bending moment effects and number of cycles produced requires analysis for each train type and span length. While the unit trains and double-stack intermodal train needed only one run per span length to determine RMC moment range and number of cycles, the mixed train is more complicated given its random nature.

To illustrate the basic approach, assume that a random 100-car train of mixed freight (including empty and full cars) crosses a bridge and continues on a given line. With this data, one can could develop the stress time-history at any given point on each span (effectively the influence lines for the real train). The data is then converted into a stress range histogram using a cycle-counting method which summarizes the number of cycles of each magnitude of stress or moment range. The rainflow cycle-counting method was used in this research (*23*). For example, for a given train passage, it may be determined that 2,500 cycles are produced, with magnitudes ranging from 0.5 ksi to 13 ksi. The histogram contains a tabulation of how many cycles were between 0.5-1.0 ksi, 1.0-1.5 ksi, etc. for this train and that it produced a total of 2,500 cycles.

Once the histogram for this train is obtained, an effective stress range is then calculated using a root mean cube approach. This process could then be repeated assuming some other 100-car "random" train and the results compared with those from the first train. The stress-range histograms from these two trains could also be combined in order to calculate an effective constant amplitude stress range as well as the cumulative damage (*21, 22*). Dividing the number of cycles by two, since two unique trains produced the cycles, would result in an effective constant amplitude stress range and equivalent number of cycles for a single 100-car train.

Obviously, such an exercise could be performed for thousands of simulations where various trains made up of various car types that vary in terms of their placement in the train is also random. Further, the distribution of full, partially full, and empty cars could also be varied. The effective moment (or stress) range and number of associated cycles could be calculated for each train. Such a simulation is commonly referred to as a Monte Carlo approach. The Monte Carlo approach is well established and is effective in estimating the probability of different outcomes (for example the effective moment range and number of cycles) as specific inputs are varied either randomly or within certain bounds. In this study, this approach was used to determine if there is a significant influence of the important parameters (car length and type, distribution of full and empty cars, and location of the cars within a train) for a given span on the calculated *effective moment range* and resulting number of cycles for mixed trains.

Variables in the Monte Carlo Simulations for Mixed Trains

One of the first steps in performing a Monte Carlo simulation is to determine what parameters will be varied in the analysis. For this study, the following were assumed based on a review of the more common railcars used to transport freight on North American Railroads. For pre-1965, the chosen railcars were the boxcar and the two main varieties of coal hopper cars. This was due to the overwhelming majority use by the railroads of these railcars. The post-1965 railcars were chosen for those regularly seen in service along with choosing a series of railcars that provided a distribution of lengths of the cars, a significant change from pre-1965 equipment. The range of railcar lengths were those that provided a sampling of lengths between the shortest railcars (Cement/Sand) to the longest (Autorack/TOFC) that are commonly used in interchange between different railroads. All equipment used in this analysis are listed in Table 2. The conditions of the Monte Carlo simulations are:

- Consistent train size; 100 railcars long led by three diesel electric locomotives
- Span length was varied from 20 feet to 120 feet
- Railcar types were those listed in the post-1965 era from Table 2
- Distribution of full, empty, and partially full cars. Various trains were evaluated in which:
  - All cars were randomly chosen and located in the consists and assumed fully loaded
  - All cars were randomly chosen and located in the consists as both empty and loaded cars
  - All cars were randomly located in the consists with both empty and full cars. Loaded cars were assumed at defined percentages of 100%, 75%, 50%, and 25%.

The number of simulations to run in order to reach convergence of a solution was next determined. This is to reach a consistent effective moment range and equivalent number of cycles per "fatigue" train simulation. During the research, studies were conducted where 10, 50, 100, 250, and 500 one-hundred-railcar trains per span length were passed over various span lengths to determine the number of simulations required to be used for all future evaluations. It was found that a near steady-state value is reached around 25 to 50 trains. For certainty of representative and convergent data, it was decided that all simulations would be run using one hundred 100-car trains for each type of simulation. While the calculations are extensive, each simulation of one hundred 100-car trains took less than 30 seconds to run and produced the effective moments and equivalent cycles per 100-car train at 0.25L, 0.375L, and 0.50L.

Figure 13 presents the midspan effective moment range from a typical simulation in which one hundred 100-car trains crossed various simple spans. A stress-range truncation of 3.0 ksi was selected in order to remove small non-damaging cycles as stated. In this simulation, the cars were randomly assumed to be either full or empty with equal probability of each. In other words, there were one hundred, 100-car trains with randomly placed types of cars that were either full or empty. The distribution in this case assumes the probability of loaded and empty cars is equal, or 50% (*this proportion of full and empty cars is simply selected for this illustrative example. However, other proportions were also studied*). Thus, there is the probability of a given train where all cars are full, all cars are empty, 20% full and 80% empty, etc.

Plotted in the figure are the calculated effective moment ranges for each simulation for each span length. Added to the plot are curves showing the mean (red line) of the data as well as the mean plus two standard deviations (green line). The green line represents the effective moment range that has a 2.5% probability of being exceeded for this simulation. As can be seen, the individual data are very tightly banded. This indicates that no matter what train passes, no major difference in the calculated effective moment range for each mixed train will occur regardless of the distribution of loaded and empty cars. This is important as it suggests that the cumulative damage from random trains of this particular distribution and configuration can be represented by a single effective moment range or an "effective fatigue mixed train moment".

The number of damaging cycles for each of the above simulations for each span length are plotted in Figure 14. As can be seen, the number of damaging cycles per 100-car mixed train exhibited greater variability. Beyond the one lower outlying set of data, the data are still tightly banded. The mean (red line) and one standard deviation (green line) of the data are also plotted in the figure. A statistically based estimate of the number of cycles produced by the gamut of different trains can be made.

Monte Carlo simulations were performed for complete set of different types of random simulated mixed trains on each span length. Ultimately, thousands of simulations for each span length were subsequently performed. Combining the results allows for the development of 1) a single effective moment range and 2) equivalent number of cycles that are produced per 100 railcar trains that can be used for fatigue evaluation of a given span length. This "fatigue train" will result in the same cumulative fatigue damage as the broad range of mixed trains that could be expected over the life of the bridge. The major advantage is that the user simply needs to have an estimate of the number of mixed trains per given time period or total over the life that the bridge was subjected. Once this has been estimated, it is a simple matter to estimate the fatigue damage on a given span at the quarter, three-eights, and midspan locations for any span and design.

Stress Range Truncation

The following discussion illustrates the importance of 1) truncating stress-range histogram data and 2) how to select an appropriate truncation level. The data used in the example were collected during the field monitoring on a highway bridge as part of a separate study and are used for illustrative purposes only. Note, they are NOT intended to represent railway traffic but only to illustrate how varying the truncation level affects the calculation of the effective moment (or stress) range and the importance of selected the proper truncation level. While from a highway bridge, the data can be used for railway bridges as the concepts are similar. In this research, the variable amplitude spectrum was obtained from the simulations from random mixed trains crossing a set of varying span lengths.

Previously it was shown that a variable-amplitude stress-range spectrum can be represented by an equivalent constant-amplitude stress range equal to the cube root of the mean cube (RMC) of all stress ranges (*21*) (i.e., $S_{Reff} = [\Sigma\alpha_i S_{ri}^3]^{1/3}$). Several methods can be used to convert a random-amplitude stress-range response into a stress-range histogram. The rainflow cycle counting method is widely used and accepted for use in most structures (*23*). In the example below, the rainflow analysis algorithm ignored any stress range less than 0.2 ksi. It is common to ignore very small cycles from the start of the cycle-counting process. For the current research, an initial cutoff of 0.5 ksi was used.

However, as stated, for an actual fatigue evaluation, the truncation level should be greater than a very low nominal value (e.g., 0.5 ksi) to ensure meaningful results are obtained. Previous research has demonstrated that stress ranges less than ¼ to ½ of the CAFL have little effect on the cumulative damage at the detail (*24*). It has also been demonstrated that as the number of random variable cycles of lower stress range levels are considered, the predicted cumulative damage provided by the calculated effective stress range becomes asymptotic to the applicable S-N curve. The calculations become consistent in terms of the estimated fatigue damage when in the truncation levels is a slope that is consistent with the S-N curve. A similar approach of truncating cycles of low stress range is accepted by researchers and specifications throughout the world (*25*).

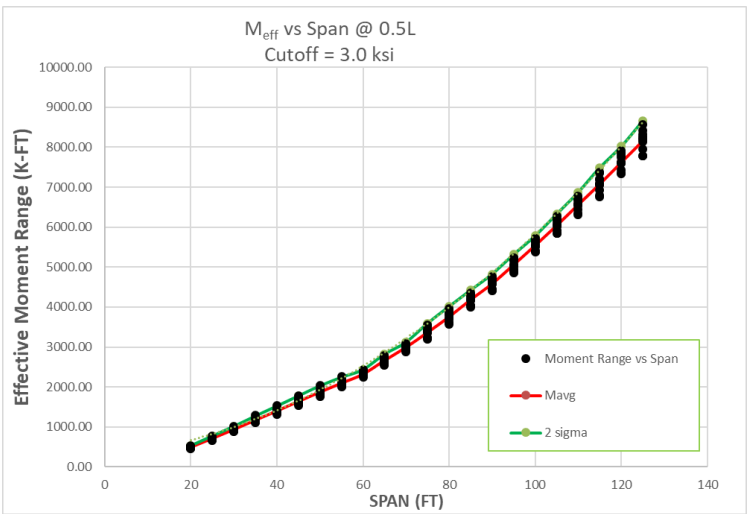


FIGURE 13. Calculated effective moment range vs span length for 100-car train assuming equal probability of either loaded or empty cars

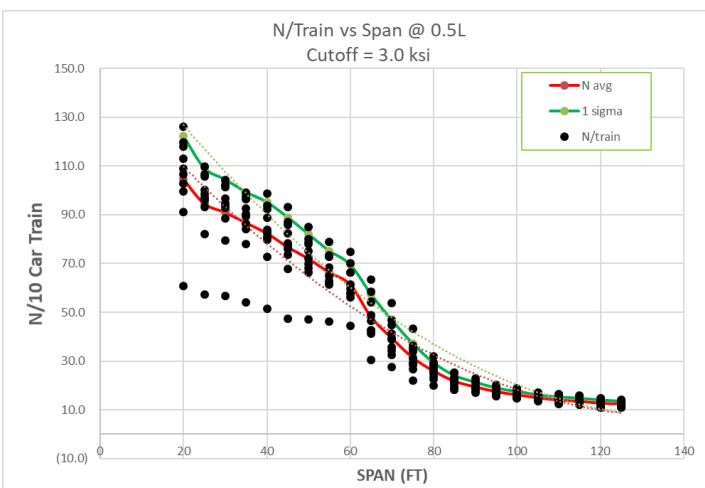


FIGURE 14. Number of damaging cycles vs span length for 100-car train assuming equal probability of either loaded or empty cars

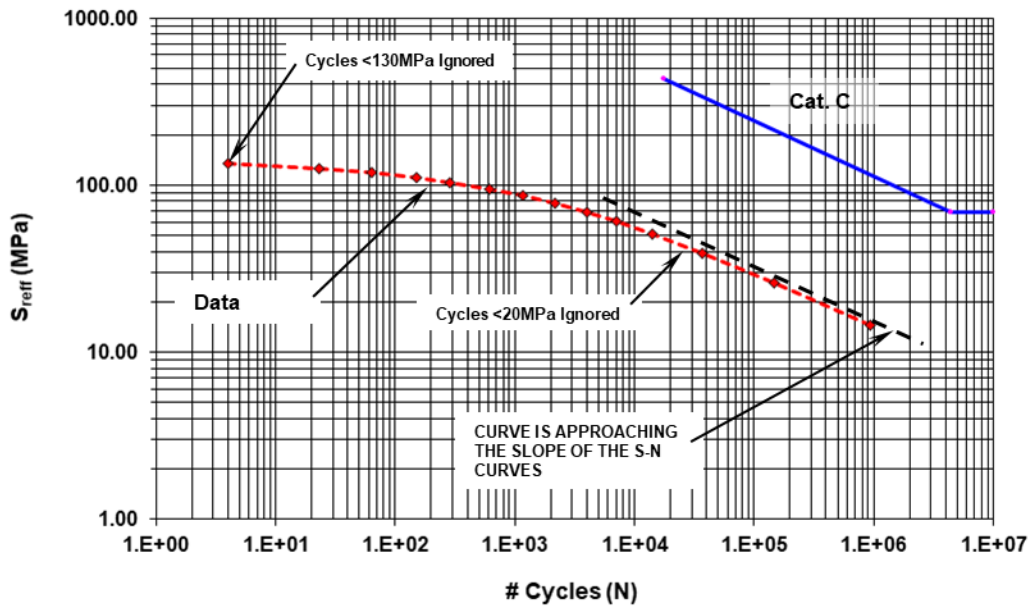


FIGURE 15. Plot showing effect of varying the truncation limit vs calculated S_{Reff}

Figure 15 shows the effect of calculating the effective stress range for several levels of truncation using test data collected from a strain gage. As can be seen, as the truncation level decreases, the effective stress range and corresponding number of cycles plotted approaches the slope of the S-N curve for Category C, which is also plotted in Figure 15 (i.e., a slope of –3 on a log-log plot). As long as the cut off level selected is consistent with the slope of the fatigue resistance curve, considering additional stress cycles at lower truncation levels does not improve the damage assessment and can therefore be ignored.

For this research, a truncation of 3 ksi was used for all evaluations. It is noted that this truncation level was also studied to ensure that it was appropriate regardless of the year of design. By comparing the cumulative damage at various levels of truncation, it was found that values ranging from 2.0 to 4.0 ksi for example, made no difference on the resulting effective fatigue damage (*i.e., the resulting* $N \times S_{Reff} = A$ *resulted in a constant value of A)* The value of 3.0 was also selected as it is ½ of the VAFL for riveted members and hence relatable to most engineers and was found to work very well for designs that were made after the base line case. For example, it is acceptable for bridges being evaluated that were designed according Cooper E-40 1906 design basis specifications as well as those designed in 2006. This is a very important finding and greatly simplifies the development of the effective fatigue train. Since the truncation is applicable to all designs, a single effective moment range can be used for all spans independent of the design year.

It is that if the evaluator had additional knowledge of the type of trains that a certain line was subjected, a more tailored evaluation could be made. For example, if an owner knew that 40% of the traffic was due to unit coal, 20% was from unit grain trains, and 40% was from mixed freight, the evaluator could combine the effective moment ranges and number of cycles per 100 car train to arrive an effective moment for that specific line. Since effective moment ranges have been developed for each type train (i.e., intermodal, unit, mixed freight, etc.), this more detailed evaluation is now possible.

The following examples and discussion present the results of the research in more detail. The effective moment ranges and equivalent number of cycles per 100 car train can be used directly or in combination to complete a fatigue evaluation. The examples in the next section illustrate the ability for either an individual train or combining a series of trains.

EXAMINATION AND APPLICATION OF RESULTS

The following examples use the data from CyclRR where fatigue lives are calculated for two 50-foot spans of different span design levels. The calculated section moduli for the different design levels are included in this section for reference. The examples start with the basic calculations and following examples demonstrate the effects of other train types and the

sensitivity of the calculations to changes in bending moment and stress range magnitudes. Included with the examples is a comparison of pre-1965 and post-1965 trains to display the difference in calculated fatigue lives between the two eras. For these examples, live-load impact has not been included. For either design or evaluation, the appropriate impact should be applied to the RMC moment ranges based upon MRE 15, Article 1.3.13 or Article 7.3.3.2, respectively

Basic CyclRR Output

The results of CyclRR program provide the RMC bending moments and number of cycles for the span length(s) defined by the user. For unit trains and articulated intermodal trains, only one solution needed to be generated. For mixed trains, the Monte Carlo simulation resulted in mean statistics as well as examination of the variance in the results. The train model used in these examples is presented with the mean values for moments and cycles of the simulation.

The train size is based on 100 railcars with either 4 locomotives for pre-1965 trains or 3 locomotives for post-1965 trains. The locations on the span are also definable by user. For this research, the three points chosen for examination were 0.25L, 0.375L, and 0.50L. Table 4 provides the tabulated version for a typical 100-car Unit Coal train, providing the RMC bending moment ranges and the associated cycles at each point ($N_{0.25}$, $N_{0.375}$, $N_{0.50}$ for 0.25L, 0.375L and 0.50L respectively). The RMC moment ranges are in kip-feet, with the magnitudes based on static values without impact applied.

For these tables, the section modulus at 0.25L was assumed to be 75 percent of the full section modulus needed under the maximum moment. This was typical of steel girder design for railway bridges with riveted design and the assumption of cover plates. For fabricated girders, this practice continues in an effort to reduce steel weight. If the fabrication includes a full section at 0.25L, or a rolled beam is used instead of a fabricated girder, the RMC moment and number of cycles are still appropriate, but the overall life will be increased due to the reduced stress range that will be calculated by the larger section modulus. The overall effect will be that if the quarter point has a full section, the life in trains will be lower than the either 0.375L or 0.50L.

The results show that as span length increases, the potential for fatigue damage from a Unit Coal train decreases. The results also show that the number of cycles is stable for a range of span lengths and then abruptly reduces to one cycle per train. Also, maximum fatigue cycling is occurring at locations other than the midspan region. The quarter points display active cycling for longer span lengths is consistent with other railcars, but as discussed, this is a combination of the train loading pattern and the reduced section used to calculate the stress ranges.

Table 5 provides the table from the Monte Carlo analysis for the mixed train. This table shows that a mixed train is more likely to generate cycles on longer span lengths when loaded and empty railcars are placed adjacent to each other in a train. The unit coal train from Table 4 produces more cycles when the stress range for each railcar is sufficient to produce a damaging fatigue cycle. Once the stress ranges from the unit coal train are insufficient, the stress from the maximum moment of the entire train is the only cycle able to inflict fatigue damage. The additional cycles from the mixed train are due to the combination of empty and loaded railcars which create greater differences in bending moment than if all railcars were loaded. .

For almost all of the tables (4, 5, 16-21) the minimum life was found to be controlled by the quarter points. Those that were not quarter points occurred at 0.35L. If a continuous section is used for the full length of the tension flange, the minimum life calculated will occur either at 0.375L or at midspan.

Application of the RMC Moment Range and Cycle Count

Tables 4 and 5 provide the RMC moment range and cycle count data for fatigue life calculation for the span lengths considered for study for Unit and Mixed train consists, respectively. The other data needed is the section modulus, and the fatigue category coefficient. Table 6 provides the gross section moduli for the AREA/AREMA design levels as calculated in CyclRR using the process outlined in this research. The section modulus at 0.25L (quarter points) was assumed to be 75 percent of the section modulus needed for maximum moment. Use of the section moduli and loading data are needed to demonstrate the method for example solutions for fatigue life. For these example problems, impact is not included. These solutions are based strictly upon using the static bending moments to demonstrate the method. In the context of the examples shown, 25 trains per day is 1,000,000 trains in approximately 110 years. Maximum train capacity per track is generally assumed to be 50 trains per day in signaled mainline territories.

TABLE 4. Unit Coal train RMC bending moment ranges (k-ft) and number of cycles per train
The trains are composed of 3 six-axle locomotives and 100 unit-coal railcars. Live load impact is not included.

Span	0.25L		0.375L		0.50L	
Length	M_{RMC}	$N_{0.25}$	M_{RMC}	$N_{0.375}$	M_{RMC}	$N_{0.50}$
5	66.7	418	83.8	418	88.9	418
10	114.9	418	119.0	418	142.7	218
15	238.5	212	343.9	107	350.4	107
20	469.5	107	562.3	107	617.1	106
25	676.8	106	888.4	106	882.1	106
30	939.6	106	1,218.7	106	1,231.8	106
35	1,202.5	104	1,548.5	104	1,580.2	104
40	1,381.8	104	1,748.9	104	1,759.0	104
45	1,482.7	104	1,875.7	104	1,940.7	104
50	1,532.1	104	1,959.0	104	1,995.4	104
55	1,547.0	103	1,989.1	103	2,012.7	103
60	1,540.4	103	1,935.1	102	2,020.1	103
65	1,503.5	102	1,858.8	102	1,882.8	102
70	1,506.3	102	1,719.4	102	1,754.0	102
75	1,558.6	102	3,690.7	3	3,737.9	3
80	1,627.7	101	4,533.6	2	4,622.4	2
85	4,003.5	2	6,093.3	1	6,273.0	1
90	5,374.9	1	6,608.3	1	6,813.0	1
95	5,848.3	1	7,105.9	1	7,391.1	1
100	6,387.7	1	7,681.0	1	8,026.5	1
105	6,987.7	1	8,422.3	1	8,656.5	1
110	7,599.7	1	9,189.8	1	9,439.5	1
115	8,211.7	1	9,956.7	1	10,249.5	1
120	8,868.0	1	10,753.2	1	11,059.5	1

EXAMPLE 1. Life in trains for any type of train.

A new service is placing coal trains on a line where 50-foot spans of the E40-1906 design and E60-1920 designs are common. The riveted spans are assumed to use Category C for fatigue life. With no assumption of past fatigue performance or concern about other trains, the bridge engineer wishes to know how many trains a bridge of that design will be able to support before the calculated end of the fatigue life. The current traffic consists of mixed trains and both lives for current and proposed traffic values are desired for comparison of service life. All trains are 100 railcars with 3 locomotives as shown in Tables 4 and 5. Live load impact is not included in this example.

From MRE 15, Article 9.7.3.3.2, the number of cycles at any stress range that a riveted connection will endure prior to initiation of a crack at the edge of the rivet hole is:

$$N = A \times S_r^{-3.0} \qquad (9)$$

where:

N = the number of cycles to crack initiation of the rivet hole
A = fatigue category coefficient
S_r = tensile stress range at the connection under investigation, ksi

For fatigue category C, A = 4.446×10^9. Category C is the fatigue category for life of the structure.

TABLE 5. Mixed train RMC bending moment ranges (k-ft) and number of cycles per train
The train is composed of 3 six-axle locomotives and 100 mixed cars from Monte Carlo simulation.
Live load impact is not included.

Span	0.25L		0.375L		0.50L	
Length	M_{RMC}	$N_{0.25}$	M_{RMC}	$N_{0.375}$	M_{RMC}	$N_{0.50}$
5						
10						
15						
20	376.8	115	456.3	108	490.9	105
25	538.4	99	671.9	95	710.6	95
30	711.1	93	887.1	91	935.0	91
35	893.3	88	1,117.1	86	1,170.1	87
40	1,077.2	84	1,345.5	81	1,400.2	83
45	1,252.1	78	1,561.3	76	1,638.4	77
50	1,425.1	74	1,788.5	71	1,872.2	72
55	1,608.0	68	2,015.4	65	2,096.0	66
60	1,795.7	62	2,254.9	58	2,320.6	62
65	2,027.2	53	2,572.9	47	2,664.4	48
70	2,288.4	44	2,900.7	38	2,993.3	40
75	2,569.1	38	3,280.7	31	3,368.5	32
80	2,893.7	32	3,672.0	26	3,755.6	26
85	3,222.2	28	4,078.4	22	4,186.4	22
90	3,567.2	24	4,504.7	20	4,588.6	20
95	3,931.4	22	4,942.7	18	5,062.0	18
100	4,296.6	20	5,387.3	17	5,546.1	16
105	4,678.6	19	5,881.4	16	6,047.3	15
110	5,068.9	18	6,368.5	15	6,568.2	14
115	5,481.6	17	6,875.4	14	7,085.6	14
120	5,891.3	16	7,395.4	13	7,627.5	13

TABLE 6. Total section moduli demand (in^3) for span lengths using AREA/AREMA design criteria
Section modulus value is for one track (two rails).

	AREA/AREMA Cooper Design Level and Impact				
Span	E40	E60	E72	E72	E80
Length	1906	1920	1935	1948	1968
5	89.3	134	143	125	94.2
10	203	306	325	285	214
15	450	680	724	634	477
20	741	1,124	1,199	1,049	789
25	1,095	1,664	1,781	1,555	1,168
30	1,475	2,243	2,373	2,097	1,574
35	1,882	2,862	2,996	2,679	2,009
40	2,361	3,588	3,718	3,362	2,520
45	2,886	4,379	4,494	4,108	3,077
50	3,434	5,198	5,285	4,882	3,653
55	4,039	6,096	6,143	5,731	4,285
60	4,705	7,078	7,071	6,660	4,974
65	5,430	8,139	8,065	7,665	5,719
70	6,201	9,255	9,096	8,720	6,500
75	6,980	10,368	10,109	9,771	7,275
80	7,871	11,634	11,255	10,963	8,153
85	8,793	12,929	12,411	12,177	9,071
90	9,766	14,281	13,603	13,437	10,044
95	10,779	15,671	14,810	14,723	11,060
100	11,845	17,120	16,052	16,052	12,134
105	13,021	18,709	16,972	16,972	13,324
110	14,313	20,447	18,504	18,504	14,641
115	15,657	22,236	20,105	20,105	16,013
120	17,035	24,049	21,749	21,749	17,421

The number of years of service for these bridges depends upon the annual use. The results (Table 7) show that for the same design era a difference in RMC stress of 7.5 percent results in a difference of over 24 percent for total cycles. The difference in cycles per train for the same number of railcars in the train means the coal train is 1.75 times as damaging in fatigue life as a mixed train for this span length.

TABLE 7. Estimated fatigue lives for 50-foot spans in Example 1.

Design Level	Train Type	RMC Stress	Total Cycles	Cycles/ Train	Total Trains	Location
E40-1906	Mixed	6.64	15,191,064	74	205,285	0.25L
	Coal	7.14	12,225,394	104	117,552	0.25L
E60-1920	Mixed	4.39	52,684,465	74	711,952	0.25L
	Coal	4.72	42,399,160	104	407,684	0.25L

EXAMPLE 2. Life in years for a mixture of trains

A rail line has a variety of trains with a weekly volume of 80 trains. The pertinent information on the trains is shown in Table 8. The bridge engineer wants to know the estimated fatigue life of the bridges (in years) used in Example 1 under the trains listed in Table 8. Intermodal trains are broken into Z and DS components. DS trains are double stack container railcars while Z trains are semi-trailers only on autorack-style platforms. Live load impact is not included.

TABLE 8. Train information for fatigue life for 50-foot spans in Example 2.

	RMC Stress		Cycles/	Trains/	Cycles/
Train Type	E40-1906	E60-1920	Train	Week	Week
Mixed	6.64	4.39	74	30	2,220
Coal	7.14	4.72	104	5	520
Vehicle	4.55	3.01	104	10	1,040
Intermodal Z	4.55	3.01	104	5	520
Intermodal DS	6.66	4.40	103	20	2,060
Ethanol	8.15	5.39	104	10	1,040

Using the Palmgren-Miner Rule for the information in Table 8, the effective RMC stresses for all trains is calculated for each of the design levels for the bridges in Example 1. The equation for combining the cycles using Palmgren-Miner is:

$$S_{Reff} = \sqrt[3]{\frac{\sum(m_i \times n_i \times S_{RMC(i)}^3)}{\sum(m_i \times n_i)}} \qquad (10)$$

where:

S_{Reff} = effective RMC stress range for the trains
i = number of different train types in the analysis
m_i = number of trains for each train type i
n_i = number of cycles per train for each train type i

TABLE 9. Combined life for train frequency in Table 8.

Design Level	Effective Stress	Trains/ Week	Cycles/ Week	Span Life Weeks	Span Life Years
E40 - 1906	6.63	80	7,400	2,059	39
E60 - 1920	4.38	80	7,400	7,142	137

Both Tables 7 and 9 demonstrate the sensitivity of the calculations to stress range magnitude. Manipulation of the stress range in equation (9) is such that accuracy of the data greatly affects the potential results. A difference of 8 percent in stress results in 24 percent difference in cycles.

A source of variability is axle weights that are not accurate for the assumed load. This discrepancy will create inaccuracy in bending stresses affecting the calculated cycle life. The values shown in Table 3 are based on full GRL. While that represents a maximum operational capacity, that does not mean that the railcar will be loaded to full GRL for each shipment. This is common for intermodal equipment, autoracks, and covered hoppers which carry multiple commodities such as different grains, or chemical products in tank cars and covered hoppers.

Records of the actual weights are confidential to the railroads but information is occasionally published that allows a comparison to actual loaded weights of certain railcars. For Example 2, comparisons are possible for automobiles and intermodal traffic. This information was reported in 2004 (*26*) which is in the time period of allowable railcar weights of 286,000 pounds. This represents current equipment in terms of expectations for axle weights. The available data allowed for calculation on a unit weight basis. This is easily calculated for the railcar equipment chosen for this research. Table 10 displays the differences between maximum allowable axle weights and actual axle weights experienced in service.

TABLE 10. Comparison of actual unit train weights to maximum unit train weights from (*20*).

	Unit Weight (plf)	
Train Type	**Capacity**	**Actual**
Intermodal Z	1,915	1,407
Intermodal DS	2,625	1,779
Vehicle	2,625	1,462

EXAMPLE 3. Life in years for a mixture of trains from Example 2 using Table 10 train weights

This example uses the same mix of trains used for Example 2 and uses the actual train weights shown in Table 10 and applying those values to the specific train types in Table 10. The other trains in Table 8 use the values shown in Table 8 based upon loading to capacity. This reduction in for the train types in Table 10 was done by using a ratio of the actual unit weight to the capacity unit weight applied to the RMC bending moments for those train types. Table 11 provides the results for the E40 and E60 design spans used in the examples. Live load impact is not included.

TABLE 11. Combined life for train frequency in Table 7 with reduced train weights from Table 10.

Design	Effective	Trains/	Cycles/	Span Life	
Level	**Stress**	**Week**	**Week**	**Weeks**	**Years**
E40 - 1906	6.04	80	7,400	2,730	52
E60 - 1920	3.99	80	7,400	9,470	182

Comparing Table 11 with Table 9, the fatigue lives of both design levels increased approximately 33 percent by the use of actual train weights for the intermodal and vehicle trains versus the full capacity train weights. This is a substantial increase in fatigue life given that the train types that were modified for their lighter actual weights (vehicles and intermodal) are among the lightest of the types used in this research. This is an indication that a general weight survey of loaded railway equipment can be critical in attempting to estimate fatigue lives for the steel railway bridge inventory.

EXAMPLE 4. Life in years for a mixture of trains from Example 2 using Table 10 and pre-1995 Gross Rail Load.

Prior to 1995, the GRL on four-axle cars was limited to 263,000 pounds versus the current 286,000 pounds. The fatigue life calculations for using the pre-1995 GRL combined with the actual train weights in Table 9 provides information on the estimated fatigue lives under that load level. The results are shown in Tables 12 and 13. Live load impact is not included.

TABLE 12. Combined life for train frequency in Table 7 with train weights from Table 10 and lower GRL.

Design Level	RMC Stress	Trains/ Week	Cycles/ Week	Span Life Weeks	Span Life Years
E40 - 1906	5.62	80	7,400	3,377	65
E60 - 1920	3.72	80	7,400	11,712	225

Table 12 shows that when compared to the maximum load capacities used in Table 12, fatigue life increases by 25 percent. Comparing Table 12 to Table 8, the increase in fatigue life is approximately 65 percent between the two load levels shown. Increases in axle weights can create a stark difference in calculated fatigue lives.

Table 13. Results of Table 7 using GRL of 263,000 pounds

Design Level	Train Type	RMC Stress	Total Cycles	Cycles/ Train	Total Trains	Location
E40-1906	Mixed	6.11	19,535,253	74	263,990	0.25L
	Coal	6.56	15,721,490	104	151,168	0.25L
E60-1920	Mixed	4.03	67,750,644	74	915,549	0.25L
	Coal	4.34	54,524,050	104	524,270	0.25L

Compared to the results in Table 7, the bridge fatigue life for these trains are 28 percent greater based upon the lower GRL. This is directly related to the reduced stress level which is 8 percent lower for the trains prior to the increase to 286,000 pounds.

EXAMPLE 5. The equipment shown in Table 3 provides four different railcars for hauling coal. The 55- and 70-ton cars plus the two levels of GRL for the Unit Coal railcar. With a capacity of 110 tons for a GRL of 286,000 pounds, that is 11,000 tons of coal as cargo in that train of 100 cars. If the other coal trains were used to deliver the same 11,000 tons of coal, which train would provide the least fatigue cycle accumulative damage to the bridges used in the example problems. Assume that the additional cars are added to the trains and use the same RMC bending moment as displayed for a 100-railcar train. Table 14 provides the results of this analysis. Live load impact is not included.

TABLE 14. Comparison of coal railcars and expected life in trains for equal coal tonnage

Design Level	Train Type	Railcars per Train	L_S/L_O	RMC Stress	Total Cycles	Cycles per Train	Total Trains
E40-1906	Unit Coal 286,000 lbs	100	0.94	7.14	12,225,394	104	117,552
	Unit Coal 263,000 lbs	110	0.94	6.56	15,721,490	114	137,908
	70-Ton Hopper	158	1.13	3.97	71,290,122	158	451,203
	55-Ton Hopper	200	1.37	6.64	15,162,319	1	15,162,319
E60-1920	Unit Coal 286,000 lbs	100	0.94	4.72	42,399,160	104	407,684
	Unit Coal 263,000 lbs	110	0.94	4.34	54,524,050	114	478,281
	70-Ton Hopper	158	1.13	2.62	247,242,854	158	1,564,828
	55-Ton Hopper	200	1.37	4.39	52,584,774	1	52,584,774

Table 14 demonstrates the effects of increased equipment weights and lengths of railcar used for transporting coal. The pre-1965 hoppers show that increased weight per car has a negative effect upon the total life of the 50-foot spans. In the case of the 55-ton hopper, the short length of the car is such that it is incapable of generating more than one cycle per train while the other trains are capable of one cycle per car. This is not just a function of railcar weight, but car length as well. The current emphasis for MRE 15 is on railcar weights without consideration of railcar lengths. The short length of the 55-ton hopper in this example is a factor in reducing the cycle count for that train.

A PROPOSED FATIGUE DESIGN LOAD

A specific design load for railway bridge fatigue is not provided in MRE 15. The current method uses the Cooper E80 Load for fatigue design which was originally created to address overall strength for bridge design with no consideration for fatigue. The Cooper E80 Load is used with an adjusted number of constant-amplitude cycles for fatigue design. This load is not a convenient load to use since the Cooper Load does not represent current locomotives and the uniform load does not properly represent actual freight railcar loads. It is also difficult to develop a rating system for railway bridge fatigue.

Figure 14 shows the proposed fatigue load developed from the results of the analysis on the railcars shown in Table 3. The load was developed by examining the results for unit trains along with the Monte Carlo mixed train analysis. For fatigue design, the design load needs to provide an upper bound of expected moment range that can be used for design of new railway bridges. Additionally, the number of cycles needs to be such that the calculated life for the design load is less than the any of the equipment currently (or projected in the future) in use on an open interchange basis.

Cycling on a per-railcar basis is extended to longer spans with longer cars and heavier axle weights as evidenced by the results of Table 14. This is opposite of the bending moment behavior of the Cooper E80 Load. The Cooper E80 Load does not provide sufficient axle spacing to provide any meaningful cycling on its own for span lengths beyond 20 feet. The uniform load representing the railcars behind the locomotives do not produce any cycles other than the maximum moment. Current design in MRE 15 assumes the number of cycles based upon span length while basing the design moment range magnitude on overall maximum moments experienced in service.

The behavior of actual railcar loads is closer to that of the Alternate Load. The Alternate Load uses 100-kip axle loads and controls maximum bending moment up to a span length of 50 feet when compared to Cooper E80. The Alternate Load was originally included in MRE 15 in recognition that existing railway traffic at the time of introduction was very close to the Cooper E80 design moment levels. The Alternate Load uses 100-kip axle loads to provide margin between actual loads and design load levels for shorter spans since the shorter spans and floor systems were experiencing fatigue damage issues noted and cited in inspection reports.

The proposed fatigue design load is termed the F80 Load. It takes advantage of the axle spacings of the Alternate Load with an axle of spacing for dimension S_I of 60.0 feet. The axle load was reduced to 80 kips versus the 100 kips of the Alternate Load. The S_I axle spacing is roughly that of the autorack/TOFC and the Long Flat railcars (66.0-foot truck centers.) This matches the current longest S_I of any railcar currently in use. Table 15 displays the RMC bending moment ranges and number of cycles produced by the F80 Load for various span lengths and locations on the spans.

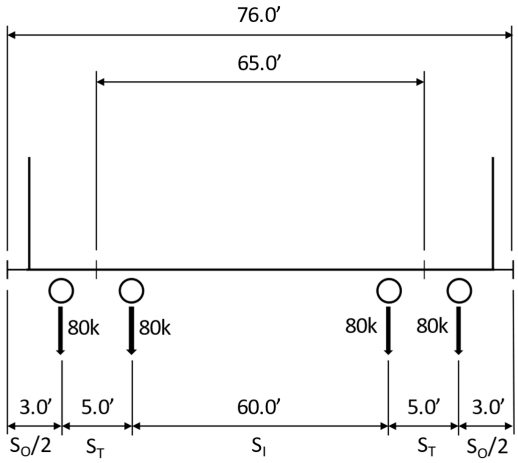


FIGURE 14. Proposed F80 Load

These and the other similar tables that were developed during the research were used to develop Figures 15 and 16 (also see Tables 16 – 21). These figures display the fatigue life for E40-1906 (Figure 15) and E60-1920 (Figure 16) spans under the selected train types. The maximum life of 1,000,000 trains represents 25 trains per day for approximately 110 years (109.6). For clarity in creating these tables and figures, no impact was included. Under normal conditions performing an evaluation, the appropriate impact should be included. The railcars in these figures all are based upon unit trains of each type of car and the mixed train was from the Monte Carlo simulation was used for that train. A successful design load will

have a fatigue life, in trains, that is less than any of the actual railcars used in service. For Figures 15 and 16, the F80 Load was run over the same locations as the virtual trains (0.25L, 0.375L, and 0.50L). The overall minimum life is plotted for all train types in Figures 15 and 16.

Figure 15 displays the susceptibility of E40-1906 designs to fatigue cycle accumulation from trains that have been or are currently used in railroad service. The proposed F80 load is also displayed. For each train type, the shorter spans are more prone to fatigue while increases in span length and section properties associated with longer spans will not be subject to as many cycles. As expected, the pre-1965 50-Ton boxcar trains provide the least amount of fatigue accumulation, while the introduction of the unit coal train produced accumulative fatigue cycles into longer span lengths. The mixed train, using current railcars, produces a fatigue life curve that is not significantly greater than the curve shown for the unit coal train. This demonstrates the effect of mixing loaded and empty railcars and the stress ranges caused by them. For railcars with similar weights, different curves are apparent. This reflects the importance of axle spacings in determination bending moment ranges on spans and how those moment ranges relate to fatigue life.

Figure 16 displays the sample trains against an E60-1920 design load. Figure 1 showed that section modulus for this design era was greatest compared with the other design levels, save for very short span lengths where E72-1935 was slightly greater. The increase in section modulus causes a shift in the range of span lengths that are affected under the trains. Increased section modulus will decrease the range of span lengths that are affected by fatigue under a train type in addition to increasing the life of the span. The proposed F80 fatigue load also still provides a minimum compared to the other train types, remaining consistent for use as a reference.

Both figures demonstrate that the F80 Load provides an estimated fatigue life in trains less than any of the railcars used in this analysis. The railcar with the life in trains closest to the F80 Load is the Long Flat, a railcar with an S_I dimension just above 60 feet and a GRL of 286,000 pounds. For the current railcar fleet, this car is the longest railcar available at the current GRL.

In addition to evaluating the results from the perspective of span length, location along the span is also critical for fatigue analysis. Bending moment cycling is more active near the quarter points and through the cover plate cutoff regions for riveted girders. This is a function of the bending moment influence line without regard to the section properties of a span. Depending upon the section properties of the span, fatigue cracking may manifest itself in this region before the midspan region. If the girders follow typical practices with a reduced section, fatigue cracking is likely to manifest itself in this region and at cover plate cutoffs. If the section is full size throughout the length, the critical location may shift.

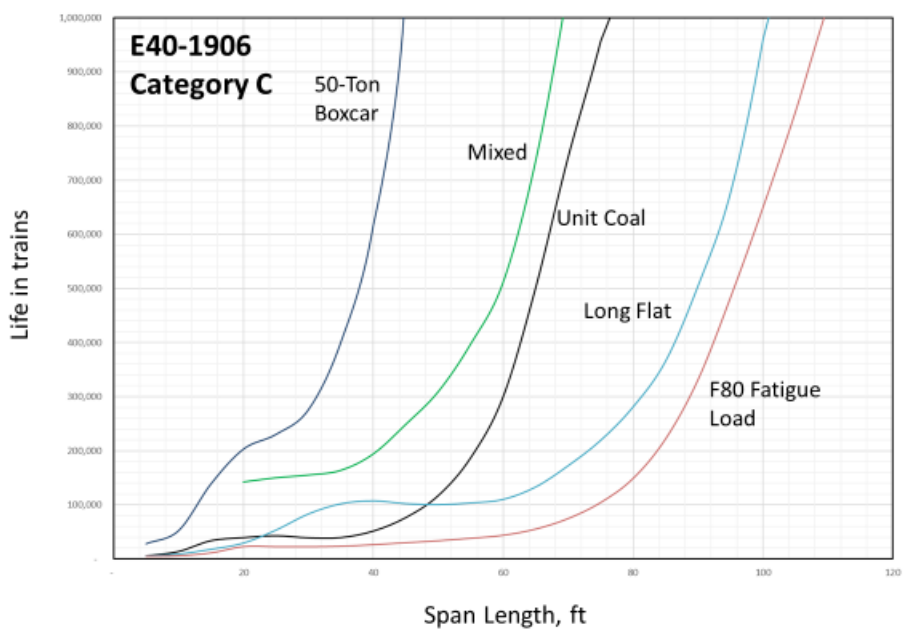


FIGURE 15. Fatigue life, in trains, for E40-1906 designs under sample trains

The proposed F80 fatigue load is meant to serve as a design load and may also be used as a reference for fatigue evaluation. For the shortest spans (< 20 feet) the difference is minimal since the level of axle load is of main importance. The Alternate Load magnitude for short spans can be proportioned to an axle load level and the decisions on desirable stresses for fatigue in connections can be based upon that magnitude. The effect of length assumes control beyond short span lengths. Axle loads may determine magnitude, but the axle spacings in relation to the span length are critical in shaping the curves seen in Figures 15 and 16 and the number of cycles that the railcars will produce. The Cooper E Load does not provide sufficient axle spacing to emulate actual railcar equipment for fatigue cycles.

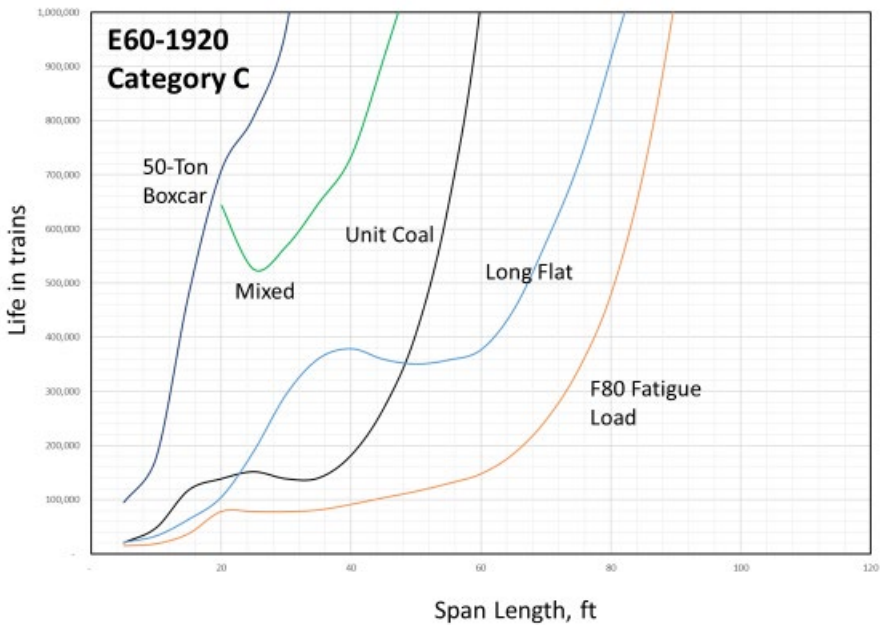


FIGURE 16. Fatigue life, in trains, for E60-1920 designs under sample trains

TABLE 15. RMC moment ranges (k-ft) and number of cycles for the F80 fatigue load (100 railcars)
The train is composed of 100 F80 fatigue load cars only. Live Load impact is not included.

Span	0.25L		0.375L		0.50L	
Length	M_{RMC}	$N_{0.25}$	M_{RMC}	$N_{0.375}$	M_{RMC}	$N_{0.50}$
5	75.0	400	93.8	400	100.0	400
10	200.0	200	225.0	200	200.0	200
15	355.0	200	413.8	200	460.0	200
20	580.0	100	740.0	100	760.0	100
25	860.0	100	1,115.0	100	1,120.0	100
30	1,160.0	100	1,490.0	100	1,520.0	100
35	1,460.0	100	1,865.0	100	1,920.0	100
40	1,760.0	100	2,240.0	100	2,320.0	100
45	2,060.0	100	2,615.0	100	2,720.0	100
50	2,360.0	100	2,990.0	100	3,120.0	100
55	2,660.0	100	3,365.0	100	3,520.0	100
60	2,960.0	100	3,740.0	100	3,920.0	100
65	3,161.0	100	3,966.6	100	4,122.1	100
70	3,263.3	100	4,094.4	100	4,324.4	100
75	3,286.9	100	4,173.2	100	4,329.6	100
80	3,291.3	100	4,134.5	100	4,335.6	100
85	3,218.1	100	4,028.3	100	4,225.1	100
90	3,126.8	100	3,856.0	100	4,038.9	100
95	3,042.5	100	3,648.7	100	3,741.4	100
100	3,029.7	100	3,446.9	100	3,377.9	100
105	3,073.1	100	7,115.0	1	7,520.0	1
110	3,147.5	100	7,490.0	1	7,920.0	1
115	3,286.9	100	7,891.3	1	8,320.0	1
120	3,485.6	100	8,450.0	1	8,720.0	1

SUMMARY AND FINDINGS

The purpose of the research was twofold. The primary purpose was development of a fatigue loading fur use in design and rating or railway beam and girder bridge spans. The second purpose was development of a computer program to provide a tool for enhanced and improved fatigue analysis for railway bridges. The computer program provided the data for the development of the proposed fatigue load. The data also provide additional knowledge to engineers related to fatigue design and analysis.

1. Section size for bending members of railway bridges can be estimated easily allowing analysis of loads on a system basis for the design criteria. The live load has a much higher magnitude than the dead load so that differences in assumption of dead load do not create large errors in the estimation. This is the case for standard designs on tangent alignments. Curved bridges or other special conditions will result in different section design criteria not included in the design equations.

2. The use of open-deck deck girder spans provides the lowest steel weight of any type of steel girder. Ballasted decks on deck girders increase the dead load which offsets any reduction allowed in live load impact. Through girder spans will have heavier main girder sections due to the increase in dead load from the floor system. Fatigue can be an issue on the floor systems in the floorbeams, stringers, and stringer connection angles on through girder bridges, but main girders will generally have higher fatigue resistance when compared to a deck girder of the same length.

3. The method of section estimation can be used for any railway bridge design load that uses Allowable Stress Design for a design method. It can be used for Load and Resistance Factor Design methods only where deflection controls the size of the section instead of the nominal moment capacity to assess service load stress magnitudes. This method depends upon linearity between load and defection.

4. Live load design level is not an indicator of the fatigue resistance of the bridge. Overall load capacity depends upon section size and material strength. Fatigue stresses from actual loads depend only upon section size and the overall condition of the bridge considering those factors that may alter section size. Material strength is not a factor for fatigue resistance for bridge steels. This research demonstrated that older design load levels with lower allowable stresses and higher impacts produced larger sections. The gross section of an E40-1906 design is roughly equivalent to an E80-1968 design for maximum size.

5. Chronological age is not an indicator of remaining fatigue life of a bridge. Age and the environment around the bridge will have an effect on its condition, but the true “age” of a railway bridge is defined by its usage.

6. Fatigue cycling is related both to axle loads and axle spacings. Pre-1965 railcar dimensions created potential fatigue issues for spans less than 50 feet. Evidence of this is that many truss bridges on the North American system have had floor systems replaced because of heavy use and light designs and large numbers of cycles due to the short loaded lengths of stringers and floorbeams. Cracked connections were common. The longer railcars introduced post-1965 have increased the length of span that is subject to a shorter life due to fatigue cycle accumulation. The increases in axle loads have the same effect.

7. The results show that the loads as arranged in a railcar fashion produce more cycling in the region of the quarter points and adjacent cover plate cutoffs than at midspan. Riveted structures were designed and constructed with decreasing section modulus from the midspan to the supports. This increases fatigue susceptibility in the region of the quarter points and cover plate cutoffs, where the maximum effects of fatigue cycling are going to occur for simple spans.

8. Railcar capacities are known and length dimensions can be obtained. Surveys and records of actual weights of loaded equipment are rarely reported on a basis applicable to performing fatigue calculations. Accuracy of axle loads would reduce error in life calculations which are most sensitive to the stress ranges. Further research to gather actual weight data is needed for more accuracy in the fatigue calculations.

9. Additional analysis of actual mixed trains is highly desirable to determine the accuracy of the models produced in this research. Although CyclRR includes a variety of railcar data to reflect the length spectrum, the entire railcar inventory is diverse in all of its length dimensions and weights. Any additional effort for determination of weights and cycling

should be based upon actual train consists versus abstracted modeling. This is possible with current waybill train data and the use of CyclRR for production of the time-histories and rainflow analysis of cyclic behavior. Accuracy of axle loadings is a critical step for this and requires access to actual data. This will provide the Engineer data for a more accurate estimate of fatigue life and a better grasp of the magnitude of loading that is being placed on the bridges currently.

10. Increased axle loads have the potential reduced the fatigue lives of the existing inventory of railroad girder bridges. Additionally, the increased length of railcars has increased the number of bridges subjected to potentially damaging fatigue especially for lower bridge design levels.

11. The introduction of the number of cycles per train based upon the span length is a major step forward in performing fatigue analysis, especially for the mixed train.

12. The number of damaging fatigue cycles per train is a relationship between the railcar equipment and its axle loads and spacings, the length of the span, and the section size of the span. Regardless of the ability to pinpoint all information for a completely accurate fatigue analysis, the proposed F80 fatigue load and use of CyclRR for an owner's inventory provides a clearer view of the cumulative fatigue damage occurring or that will occur on their bridges.

13. The Cooper E Load remains useful for overall maximum design loads and sizing of span, but does not represent the actions and behavior of actual railcars in service for fatigue cycling. Factoring of the Cooper E Load for expected bending moments and cycles may not account for all cycles at the proper stress range experienced from the variation in bending moment as a train traverses a bridge.

ADDITIONAL TABLES

TABLE 16. 50-Ton Boxcar (pre-1965) RMC moment ranges (k-ft) and number of cycles per train
The train is composed of 4 four-axle locomotives and 100 boxcars. Live load impact is not included.

Span	0.25L		0.375L		0.50L	
Length	M_{RMC}	$N_{0.25}$	M_{RMC}	$N_{0.375}$	M_{RMC}	$N_{0.50}$
5	40.4	416	50.7	416	53.8	416
10	93.0	216	102.0	216	96.4	216
15	150.3	209	209.2	108	202.4	105
20	272.1	108	322.8	105	342.2	105
25	389.3	105	492.8	105	496.1	105
30	493.6	105	617.9	105	588.8	105
35	558.0	104	700.7	103	698.9	103
40	608.2	103	755.1	102	736.4	103
45	618.0	100	799.0	100	749.2	100
50	1,426.0	1	1,845.1	1	1,860.0	1
55	1,661.6	1	2,135.7	1	2,170.0	1
60	1,951.5	1	2,426.4	1	2,480.0	1
65	2,237.4	1	2,717.1	1	2,862.9	1
70	2,569.9	1	3,105.7	1	3,255.0	1
75	2,921.8	1	3,543.9	1	3,642.5	1
80	3,267.4	1	3,977.7	1	4,061.0	1
85	3,660.3	1	4,484.0	1	4,577.2	1
90	4,065.7	1	4,995.2	1	5,115.0	1
95	4,495.0	1	5,526.4	1	5,666.2	1
100	4,960.0	1	6,109.5	1	6,237.5	1
105	5,425.0	1	6,693.5	1	6,901.9	1
110	5,928.8	1	7,277.6	1	7,583.3	1
115	6,440.3	1	7,937.9	1	8,318.8	1
120	6,975.0	1	8,598.8	1	9,034.5	1

TABLE 17. 55-Ton Hopper (pre-1965) RMC moment ranges (k-ft) and number of cycles per train
The train is composed of 4 four-axle locomotives and 100 55-ton hopper railcars. Live load impact is not included.

Span	0.25L		0.375L		0.50L	
Length	M_{RMC}	$N_{0.25}$	M_{RMC}	$N_{0.375}$	M_{RMC}	$N_{0.50}$
5	40.7	416	51.1	416	54.2	416
10	89.4	216	97.6	216	111.8	117
15	179.3	110	209.6	108	212.2	105
20	250.1	108	289.0	105	297.1	105
25	296.7	105	376.0	104	354.9	104
30	343.1	104	430.9	104	412.5	104
35	550.6	5	704.8	4	753.4	4
40	689.4	4	946.3	3	902.4	4
45	1,193.5	1	1,548.5	1	1,566.0	1
50	1,426.0	1	1,845.1	1	1,860.0	1
55	1,661.6	1	2,135.7	1	2,170.0	1
60	1,951.5	1	2,426.4	1	2,480.0	1
65	2,237.4	1	2,717.1	1	2,862.9	1
70	2,569.9	1	3,105.7	1	3,255.0	1
75	2,921.8	1	3,543.9	1	3,642.5	1
80	3,267.4	1	3,977.7	1	4,096.6	1
85	3,660.3	1	4,484.0	1	4,590.4	1
90	4,065.7	1	4,995.2	1	5,124.4	1
95	4,495.0	1	5,526.4	1	5,692.6	1
100	4,960.0	1	6,109.5	1	6,335.7	1
105	5,425.0	1	6,693.5	1	7,008.9	1
110	5,928.8	1	7,277.6	1	7,701.9	1
115	6,440.3	1	7,937.9	1	8,375.2	1
120	6,975.0	1	8,612.3	1	9,090.2	1

TABLE 18. 70-Ton Hopper (pre-1965) RMC moment ranges (k-ft) and number of cycles per train
The train is composed of 4 four-axle locomotives and 100 70-ton hopper railcars. Live load impact is not included.

Span	0.25L		0.375L		0.50L	
Length	M_{RMC}	$N_{0.25}$	M_{RMC}	$N_{0.375}$	M_{RMC}	$N_{0.50}$
5	49.3	416	62.0	416	65.8	416
10	85.8	416	133.7	117	131.9	117
15	217.4	110	254.2	108	285.9	105
20	359.2	108	454.1	105	479.1	105
25	528.0	105	694.9	105	699.3	105
30	682.8	105	880.6	105	882.3	105
35	783.2	105	1,002.3	104	1,016.4	104
40	838.2	103	1,080.1	103	1,111.0	103
45	849.1	100	1,125.9	100	1,125.9	100
50	851.2	100	1,082.0	100	1,131.3	100
55	830.7	100	1,795.5	2	1,868.9	2
60	2,051.0	1	2,459.6	1	2,563.2	1
65	2,346.3	1	2,830.9	1	2,925.9	1
70	2,676.1	1	3,248.7	1	3,288.8	1
75	3,003.1	1	3,659.2	1	3,690.9	1
80	3,373.1	1	4,080.0	1	4,135.9	1
85	3,815.6	1	4,556.2	1	4,621.4	1
90	4,260.2	1	5,099.8	1	5,186.0	1
95	4,708.6	1	5,656.5	1	5,776.5	1
100	5,152.2	1	6,221.2	1	6,375.6	1
105	5,602.9	1	6,781.7	1	7,054.1	1
110	6,092.4	1	7,342.2	1	7,717.1	1
115	6,589.7	1	7,937.9	1	8,440.6	1
120	7,110.2	1	8,598.8	1	9,187.1	1

TABLE 19. Double Stack Intermodal (post-1965) RMC moment ranges (k-ft) and number of cycles per train
The train is composed of 3 six-axle locomotives and 20 five-platform railcars. Live load impact is not included.

Span	0.25L		0.375L		0.50L	
Length	M_{RMC}	$N_{0.25}$	M_{RMC}	$N_{0.375}$	M_{RMC}	$N_{0.50}$
5	67.9	258	84.9	258	90.6	258
10	163.3	118	177.1	118	181.0	118
15	309.1	106	357.5	106	337.8	106
20	453.6	106	534.7	106	533.3	106
25	604.5	106	725.1	106	731.7	106
30	763.2	106	933.6	106	936.4	106
35	902.7	103	1,133.0	103	1,152.1	103
40	1,086.4	103	1,339.3	103	1,362.2	103
45	1,255.7	103	1,544.6	103	1,576.1	103
50	1,430.2	103	1,764.7	103	1,788.3	103
55	1,561.6	103	1,923.2	103	2,069.9	103
60	1,644.0	103	2,003.8	103	2,038.2	103
65	1,647.8	103	2,006.8	103	2,094.4	103
70	1,649.2	103	1,987.0	103	2,029.1	103
75	1,644.4	103	2,712.6	23	2,760.4	23
80	2,256.2	23	2,750.0	23	2,849.5	23
85	2,353.2	21	2,848.5	21	2,957.7	21
90	3,025.1	21	6,519.0	1	6,876.0	1
95	5,737.3	1	6,979.6	1	7,253.7	1
100	6,237.3	1	7,500.1	1	7,837.5	1
105	6,841.9	1	8,242.6	1	8,465.0	1
110	7,480.1	1	9,043.2	1	9,290.3	1
115	8,123.2	1	9,849.4	1	10,142.0	1
120	8,810.8	1	10,676.6	1	10,983.8	1

TABLE 20. Ethanol/Crude (post-1965) RMC moment ranges (k-ft) and number of cycles per train
The train is composed of 3 six-axle locomotives and 100 Ethanol/Crude tank cars. Live load impact is not included.

Span	0.25L		0.375L		0.50L	
Length	M_{RMC}	$N_{0.25}$	M_{RMC}	$N_{0.375}$	M_{RMC}	$N_{0.50}$
5	66.6	418	83.7	418	88.8	418
10	117.1	418	122.6	418	146.5	218
15	242.5	212	279.2	206	326.5	107
20	453.7	107	539.0	107	584.8	106
25	652.6	106	840.1	106	849.9	106
30	908.2	106	1,169.1	106	1,169.1	106
35	1,178.4	104	1,510.5	104	1,528.1	104
40	1,444.4	104	1,841.6	104	1,879.9	104
45	1,631.7	104	2,058.4	104	2,084.0	104
50	1,749.7	104	2,199.2	104	2,263.7	104
55	1,831.6	104	2,295.1	104	2,352.0	104
60	1,847.3	103	2,374.2	102	2,369.9	103
65	1,865.8	102	2,328.2	102	2,388.5	102
70	1,808.8	102	2,252.6	102	2,328.0	102
75	1,775.1	102	2,150.9	102	2,189.9	102
80	1,774.9	102	4,019.5	3	4,094.0	3
85	1,841.0	101	6,093.3	1	6,273.0	1
90	5,374.9	1	6,608.3	1	6,813.0	1
95	5,848.3	1	7,105.9	1	7,391.1	1
100	6,387.7	1	7,681.0	1	8,026.5	1
105	6,987.7	1	8,422.3	1	8,656.5	1
110	7,599.7	1	9,189.8	1	9,439.5	1
115	8,211.7	1	9,956.7	1	10,249.5	1
120	8,868.0	1	10,753.2	1	11,059.5	1

TABLE 21. Autorack/TOFC (post-1965) RMC moment ranges (k-ft) and number of cycles per train
The train is composed of 3 six-axle locomotives and 100 autorack/TOFC railcars. Live load impact is not included.

Span	0.25L		0.375L		0.50L	
Length	M_{RMC}	$N_{0.25}$	M_{RMC}	$N_{0.375}$	M_{RMC}	$N_{0.50}$
5	43.9	418	55.2	418	58.6	418
10	106.5	218	118.0	218	112.8	218
15	192.7	212	227.4	206	218.6	206
20	281.2	206	337.0	206	337.6	206
25	358.9	206	431.5	205	434.9	205
30	418.1	205	505.7	205	517.9	205
35	567.7	104	698.8	104	723.0	104
40	679.9	104	810.8	104	834.2	104
45	811.0	104	950.3	104	958.0	104
50	977.3	104	1,154.3	104	1,118.5	104
55	1,145.5	104	1,367.6	104	1,290.6	104
60	1,316.2	104	1,585.2	102	1,508.6	104
65	1,451.3	101	1,735.8	101	1,654.3	101
70	1,534.6	101	1,840.5	101	1,786.8	101
75	1,609.1	101	1,917.7	101	4,209.1	2
80	1,684.1	101	4,533.6	2	4,622.4	2
85	4,003.5	2	6,093.3	1	6,273.0	1
90	5,374.9	1	6,608.3	1	6,813.0	1
95	5,848.3	1	7,105.9	1	7,391.1	1
100	6,387.7	1	7,681.0	1	8,026.5	1
105	6,987.7	1	8,422.3	1	8,656.5	1
110	7,599.7	1	9,189.8	1	9,439.5	1
115	8,211.7	1	9,956.7	1	10,249.5	1
120	8,868.0	1	10,753.2	1	11,059.5	1

PLANS FOR IMPLEMENTATION

Implementation has three separate objectives. These are:

- Proposed fatigue load inclusion into design recommendations
- Presentation and dissemination
- Use of CyclRR for operational purposes and future research

The proposed fatigue loading and the data presented in this report provide a level of detail for railway bridge analysis that has not been previously available. The introduction of the number of cycles per train based upon equipment axle spacings and span length is a major step forward in performing analyses, especially for the mixed train.

PROPOSED FATIGUE LOAD INCLUSION IN DESIGN RECOMMENDATIONS

The obvious location for the proposed fatigue design load (F80) is inclusion in the design recommendations for steel railway bridges. This is in Chapter 15 of the AREMA MRE. This will be facilitated by the PI and co-PI of this research as they are both members of Committee 15 of AREMA, the committee responsible for Chapter 15. The co-PI is chairman of the subcommittee responsible for ratings and bot PI and co-PI are members of the subcommittee responsible for design. A ballot proposal with presentations to the entire committee will be necessary for this material. Committee 15 will convene an ad hoc working group to review the material and determine what portion of it should be included in the main portion of the chapter and the associated commentary. Other data generated in this research may be included as commentary material, or other methods of distribution of the material will be developed.

PRESENTATION AND DISSEMINATION

Presentation and dissemination are important for this research. This is possible in multiple methods including presentations at conferences, association meetings, and seminars (including all-day) associated with continuing education. Presentation to public agencies operating commuter lines or owning leased freight lines are appropriate. This can be accomplished through the listed methods or to the agency directly if sufficient staff need to attend. The results affect not only design and rating, but are critical for proper inspection and budgetary priority on the assets for repair and replacement.

Another critical group for outreach is short lines and regional railroads. Engineering functions, expenses, and upgrades are incrementally more expensive for these sizes of railroads, and accurate information on their bridges is important to make informed decisions. Results of this research is very useful to this group of owners and operators. Many of them are maintaining some of the oldest infrastructure.

USE OF CYCLRR FOR OPERATIONAL PURPOSES AND RESEARCH

The software was developed for use in the research but its usefulness in modeling trains allows running any number of train compositions or concepts for future operations. A straightforward application of the software is use by a railroad with a link to the waybill information and routing, and a separate link to the UMLER (Universal Machine Language Equipment Register) administered by Railinc (part of the Association of American Railroads). The UMLER link provides the car dimensions and weight data while the waybill information for the train has the car identity, load/empty status, and actual weight (if loaded). A virtual train can be constructed in real time and the cycles calculated over the necessary span lengths. This allows the bridge engineer to monitor the cycling along a particular route or at a specific bridge. It allows the bridge engineer to monitor in real time and assess the data as seen fit.

That same use is a research project on its own by analysis of the data that is being generated. The example calculations displayed a sensitivity to the magnitude of stress range since it is a cubic function in the calculations. Fatigue analysis needs accurate load information to obtain accurate estimates. The use of the CyclRR allows this type of analysis of train weights with development and potential revision to this research. The data is easily sortable by railcar type, and procedures to protect sensitive information are simple given the database architecture.

CONCLUSIONS

A critical step forward in railway bridge fatigue design and evaluation has been made with the completion of this research. Prior to the work performed for this project:

- the complete knowledge of cyclical behavior for train sets had not been available.
- without the knowledge of number of cycles per train for the various trains shown in this report, the basis for determination of fatigue cycling was subjective with the potential for each user having a different interpretation of the results
- subjectivity arriving at an answer of a fatigue life estimation that is quite different from someone else performing the same analysis.
- more information was available that could be applied to this issue than had been applied, leaving many unanswered questions relating to bridge fatigue.

The main purpose of this research was to take available knowledge on a high-level basis, and using the tools from that knowledge, create additional tools for use on a lower level where the end user can progress from subjective assessment about cyclic behavior to produce results in a quicker and more objective fashion. The result is moving beyond "guesswork" on the effects of bridges subjected to trains in fatigue to producing more consistent results and allowing those results to be used in a more confident and useful fashion for inspections, ratings, and budget planning. The methods and knowledge from this research allow a more comprehensive examination with confidence of potential issues surrounding railway bridge fatigue and the steps that need to be taken to ensure the future of the structure, whether that be retention on a longer term, making necessary repairs and strengthening to extend its use, or the realization that replacement may be in order.

The results of the research allow users to more quickly determine how long a railway bridge is expected to last in service, whether it is a new design, or by knowledge of past use, how long an existing railway bridge may continue in use. This has not been easily achievable up to this point since the data concerning the effects of a train passage had not been fully defined. The research in this project defined missing information and fills major gaps in the ability of the bridge engineer to quickly and confidently produce life calculations that can be more meaningful to all parties who may need this information.

This is especially useful for public agencies and short lines or regional railroads. The tools from this research allow them the same information for discussions and decision-making on projects that may represent a significant portion of their infrastructure budgets compared to a larger railroad. An example is a proposed bridge replacement where the chronological age of a bridge may seem significant but its actual usage may point to only limited loss of life when examined from a bridge fatigue point of view. This is an area that can result in better economics on infrastructure spending which is often under close scrutiny for both public agencies and small railroads.

The ultimate result of the research is that the railway bridge fatigue can now move from a position of incomplete information with some left to subjective evaluation using more objective measures with less concern about the validity of the calculations. The tools, methods and software produced in this research follow accepted practices for fatigue evaluation and analysis and will serve to be a part of the solution versus an issue of concern without objective measures.

INVESTIGATORS' PROFILES

ROBERT CONNOR, PhD, PE – PRINCIPAL INVESTIGATOR

Dr. Connor is the Jack and Kay Hockema Professor in Civil Engineering and Director of CAI and S-BRITE at Purdue University. He has 30 years of experience with bridge behavior, laboratory testing, field evaluation, inspection, long-term performance, design, and analytical modeling of steel bridges. Over his career, he has conducted field evaluations of bridges throughout the United States and in bridge evaluations internationally. He has researched fabrication flaws, fatigue cracking, and failures and developed repair strategies for structures for a variety of agencies including state DOT, rapid transit authorities, construction companies, and structural consultants. He has developed and is currently developing fatigue design specifications for highway bridge structures and bridge expansion joints for NCHRP and state agencies. In addition, he has developed a two-day short course focused on fatigue and fracture design for steel bridge structures geared to toward practicing engineers. He has led efforts in developing the current criteria used to assess the vulnerability of selected welded details to constraint induced fracture through research sponsored by FHWA. The work served as the basis for the provisions present found in the AASHTO LRFD Bridge Design Specifications. Dr. Connor is a member of a number of technical committees including AREA Committee 15, Steel Structures.

STEPHEN DICK, PhD, SE – CO-PRINCIPAL INVESTIGATOR

Dr. Dick is a Senior Research Engineer at Purdue University Bowen Laboratory. He has 42 years of experience in the railroad industry and research. His professional experience is extensive in the fields of railroad bridge design and rating, track and alignment design, and facilities including operating facilities and mechanical facilities. His work has included operations analysis including timetable development, velocity profiling, and train performance. He also has extensive construction experience in all phases of railway engineering. His experience includes freight rail, heavy-rail passenger and commuter rail, and light rail. His major expertise is in bridge fatigue where his research has been fundamental in describing the behavior of railroad loadings on bridges. His teaching experience includes graduate level instruction in railroad track design and bridge design. He is a member of AREMA Committee 15, Steel Structures. He is past chairman of Subcommittee 1, Design and is currently chairman of Subcommittee 5, Ratings.

CEM KORKMAZ, PhD

Dr. Korkmaz is a Research Engineer at Purdue University Bowen Laboratory. He received his BS from Middle East Technical University and his PhD from Purdue University. His research interests include redundancy of composite steel bridges with fracture-critical members and fatigue assessment with finite element analysis.

MYRIAM SARMENT, MSCE, EI

Myriam Sarment, MSCE, EI is a PhD student at Purdue University Lyles School of Civil Engineering. She received three bachelor's degrees in Civil Engineering, Environmental Engineering and Mathematics with honors from Michigan State University. She received her Master of Science in Civil Engineering with concentrations in Structural Engineering and Computational Engineering from Purdue University. She was in a cooperative education program with Michigan Department of Transportation Bureau of Bridges and Structures, where she gained field work and design experience for emergency bridge repairs. This translated to her research interests including rehabilitation of steel bridges and structures by experimentation and computational approaches.

CECELIA MAGINOT, BSCE

Cecelia Maginot is a recent graduate in Civil Engineering at Purdue University. Her research interests include civil and structural design for sustainable energy facilities. She is currently an Assistant Civil Engineer with Burns & McDonnell. She is currently working in the areas of transmission and distribution specializing in high-voltage substation design.

REFERENCES

1. American Railway Engineering and Maintenance-of-Way Association. *Manual for Railway Engineering*, Chapter 15 Steel Structures. AREMA, Lanham, MD, 2022.

2. J. W. Fisher. *Evolution of Fatigue-Resistant Steel Bridges.* Journal of the Transportation Research Board, Vol. 1594, Issue 1, January 1997, pp. 5-17.

3. A. M. Rakoczy, D. Otter, S. Dick. *Testing of 155-Foot Girder Span for Fatigue Loading and Railcar Length Effects.* Technology Digest 17-029, Transportation Technology Center, Inc., Pueblo, Colorado, 2017.

4. D. Otter, S. Dick, and C. Johnson. *Fatigue Screening for Steel Deck Plate Girder Bridge Spans*. Railway Track & Structures, Vol. 117 no. 9, September 2021, pp. 4-6.

5. J. W. Fisher, P. A. Albrecht, B. T. Yen, D. J. Klingerman, and B. M. McNamee. *Fatigue Strength of Steel Beams with Welded Stiffeners and Attachments.* National Cooperative Highway Research Program Report 147, Transportation Research Board, 1974.

6. J. W. Fisher, B. T. Yen, D. Wang, and J. E. Mann. *Fatigue and Fracture Evaluation for Rating Riveted Bridges.* National Cooperative Highway Research Program Report 302, Transportation Research Board, 1987.

7. Y. Zhou. *Fatigue Strength Evaluation of Riveted Bridge Members.* PhD Dissertation, Lehigh University, Bethlehem, Pennsylvania, 1994.

8. J. W. Fisher, G. L. Kulak, I. F. C. Smith. *A Fatigue Primer for Structural Engineers.* National Steel Bridge Alliance, Chicago, Illinois, 1998.

9. J. E. Greiner. *Rolling Loads on Bridges*. American Railway Engineering Association Bulletin 139, September 1911, pp.45-54.

10. American Railway Engineering and Maintenance of Way Association. *General Specifications for Steel Railroad Bridges, Part First – Design.* Proceedings of the Seventh Annual Convention of the American Railway Engineering and Maintenance of Way Association, Volume 7, 1906, pp. 185-194.

11. American Railway Engineering Association. *General Specifications for Steel Railway Bridges for Fixed Spans Less than 300 Feet in Length.* Proceedings of the Twenty-First Annual Convention of the American Railway Engineering Association, Volume 21, 1920, pp. 493-511.

12. American Railway Engineering Association. *Specifications for Steel Railway Bridges for Fixed Spans Not Exceeding 400 Feet in Length.* Proceedings of the Thirty-Sixth Annual Convention of the American Railway Engineering Association, Volume 36, 1935, pp. 632-689.

13. American Railway Engineering Association. *Report of Committee 15 – Iron and Steel Structures.* Proceedings of the Forty-Seventh Annual Convention of the American Railway Engineering Association, Volume 49, 1948, pp. 200-206.

14. American Railway Engineering Association. *Specifications for Steel Railway Bridges.* Proceedings of the American Railway Engineering Association, Volume 70, Bulletin 617, 1969, pp. 242-254.

15. G. A. Hool and W. S. Kinne. *Steel and Timber Structures.* McGraw-Hill Book Company, New York, 1942.

16. M. S. Ketchum. *Structural Engineers' Handbook.* McGraw-Hill Book Company, New York, 1924.

17. E. C. Dilworth. *Steel Railway Bridges – Designs and Weights*. D. Van Nostrand Company, New York, 1916.

18. J. B. Hunley. *Impact in Railway Bridges of Simple Span.* Proceedings of the Thirty-Seventh Annual Convention of the American Railway Engineering Association, Volume 37, 1936, pp. 741-939.

19. Association of American Railroads. *AAR Manual of Standards and Recommended Practices – Design, Fabrication, and Construction of Freight Cars,* M-1001 Chapter 2. Association of American Railroads, Washington, D.C., 2007.

20. S. M. Dick. *Bending Moment Approximation Analysis for Use in Fatigue Life Evaluation of Steel Railway Girder Bridges.* PhD Dissertation, University of Kansas, Lawrence, Kansas, 2002.

21. M. A. Miner. *Cumulative Damage in Fatigue.* Journal of Applied Mechanics, Vol. 1, No.1, Sept., 1945.

22. F. Moses, C. G. Schilling, K. S. Raju. *Fatigue Evaluation Procedures for Steel Bridges, NCHRP Report 299.* National Cooperative Highway Research Program, Washington, DC, 1987.

23. S. D. Downing, D. F. Socie. *Simple Rainflow Counting Algorithms.* International Journal of Fatigue, January 1982.

24. J. W. Fisher, A. Nussbaumer, P. B. Keating, and B. T. Yen. *Resistance of Welded Details Under Variable Amplitude Long-Life Fatigue Loading, NCHRP Report 354*. National Cooperative Highway Research Program, Washington, DC, 1993.

25. C. G. Schilling. *Variable Amplitude Load Fatigue, Task A - Literature Review: Volume I - Traffic Loading And Bridge Response.* Publication No. FHWA-RD-87-059, Federal Highway Administration, Washington, DC, July 1990.

26. F. W. Frailey. *24 Hours at Mountainair*. Trains Magazine, Volume 64 Issue 10, October 2004, pp. 44-45.

APPENDIX: RESEARCH RESULTS

Sidebar Info

Program Steering Committee: Rail Safety IDEA Program Committee

Month and Year: September 2022

Title: DEVELOPMENT OF A FATIGUE LOAD FOR RAILWAY BRIDGES

Project Number: RS-45
Start Date: October 15, 2020
Completion Date: September 7, 2022

Product Category: Steel Railroad Bridge Safety

Principal Investigator: Robert Connor, PhD, Professor, Purdue University
rconnor@purdue.edu
Phone: 765-496-8272

TITLE:
A Fatigue Load for Railway Bridges

SUBHEAD:
A new loading for fatigue design was created to provide a loading recreating the actual behavior of railway cars on bridges.

WHAT WAS THE NEED?
The inventory of steel railway bridges on the freight railroads have been in service in some cases over 120 years. They still provide adequate service and are in good condition, but fatigue of the steel in the bridges always needs to be addressed. The current design rules for railway bridge fatigue do not have a reference loading that can be easily compared.

WHAT WAS OUR GOAL?
The goal was development of a fatigue loading for North American steel railway bridges that could be used for design of new bridges and serve as a basis for comparison and a common rating system for existing bridges.

WHAT DID WE DO?

The effort required development of computer software, called CyclRR, that creates virtual trains and passes them over a bridge of any given length. The software was built to allow examination of conditions at any point on the bridge, loaded under trains of typical railcars in service railway service. CyclRR provided the structural analysis calculations, time-histories, stress ranges, and rainflow analysis for number of cycles under each train.

The data generated by the output allowed compilation of tables for typical trains along with development of a "mixed" train (loaded and empty cars in a random arrangement) using Monte Carlo simulation. The mixed train data is a major step in railway bridge fatigue evaluation.

From the tables, the behaviors of trains on bridge life calculations were studied and the design fatigue load was developed from that analysis. The load is based upon one loading also used in railway bridge design and emulates rail equipment providing similar behavior in loading the bridge.

WHAT WAS THE OUTCOME?

The research results have been enlightening in understanding the actions of the train loadings and how a bridge may be stressed during the train passage. General design has concentrated upon maximum conditions at specific locations, but fatigue behavior behaves differently. This provides useful information for knowing where to inspect for potential damage while knowing other areas are not as concerning.

The analysis included examination of bridges on a general basis instead of individual basis. This general basis used was the level of design load used for the bridges using the five historical design levels used in North America. The sensitivity of the design levels to fatigue was examined by this process. This provides categorization of the inventory aiding in assessment of potential fatigue issues.

The software is available for continued research and can also be modified to monitor cycling on a bridge or a railroad territory with appropriate data connections.

WHAT IS THE BENEFIT?

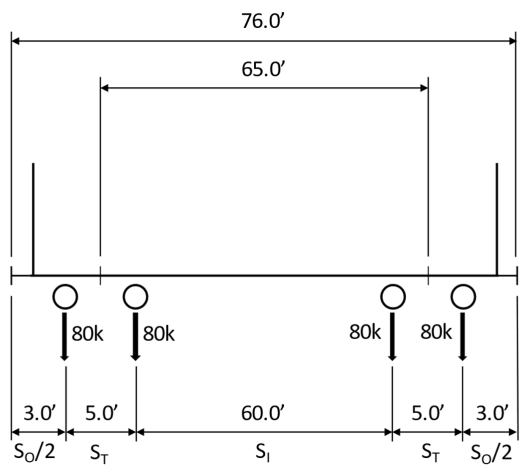
The research, fatigue tables, and proposed fatigue load provide a common basis for examination of any steel railway bridge for fatigue. It provides a convenient reference point such that subjectivity can be reduced in decision-making for repairs or replacement. The question of age of a bridge can be estimated in terms of usage instead of strictly age with conjecture of its usage.

This is beneficial for large railway operations simply in terms of maintaining the entire inventory and budgeting for repairs or replacements. This is even more beneficial for public agencies and short lines where infrastructure capital expenditures represent a significant decision in terms of amount and ability to finance. The age of a railway bridge has another tool to determine its life in usage. A chronological age of a railway bridge may contribute to its overall condition, but its age in usage is important as another indicator of its useful future.

LEARN MORE

Please find the project final report posted on the Rail Safety IDEA website.

IMAGES



The F80 Fatigue Load is displayed in the figure. The load was developed to have the same characteristics as a standard freight railcar emulating the same behavior.