ABSTRACT
Riveted steel deck girders comprise the majority of the deck girders on North American railroads, and they have been in service for many decades. Fatigue cycle accumulation creates concern that crack development might require repair or replacement of these spans. Given the number of spans, examination of individual locations is time-consuming and inefficient. Railroad bridge designs are standardized and the use of standard spans means a fatigue check also can be standardized. A method is presented that takes advantage of design parameters of girders for a range of span lengths designed to a specific loading to check for potential fatigue cycle accumulation. Advantages of the method are that multiple span lengths are checked for loadings from actual equipment instead of an assumed loading. Another major advantage is the ability to determine whether fatigue cycles are accumulating at the rate of one cycle per railcar or one cycle for an entire train. The method allows a quick assessment of a typical group of spans designed to a standard, pinpointing areas for more detailed analysis. An assessment of past and present railcar equipment is presented compared to standardized AREA/AREMA girder designs. The analysis explains girder performance trends that have been noted over the years. In particular, longer spans generally have not shown fatigue-related problems. The analysis also indicates that a significant portion of the railway deck girder inventory, especially the longer spans, are not likely to show fatigue-related problems for many more decades under typical traffic conditions.

INTRODUCTION
Fatigue evaluation of steel bridges on railroads is a critical task but can be complicated and time-consuming. Given the high number of steel bridges on the North American railway system, a systematic approach to fatigue evaluation can save time and effort. This paper provides a method for performing a preliminary evaluation that utilizes design information of spans along with an analysis of the railcar equipment that is in service on the railways.

The type of bridge under focus in this study is the riveted steel two-girder deck girder bridge. Population estimates of this type of span indicate they comprise up to one-third of all bridges in North America. Examination of a sample of bridge records indicates that approximately 75 percent of these bridges are at least 75 years old, with a large portion of that group exceeding 100 years old. Given these conditions, a convenient check to identify bridge spans subject to high-cycle fatigue is worthwhile.

This method provides for preliminary evaluation of girders. Older steel spans are standardized by design loading and impact over a range of span lengths, and the standardization places the same span design in multiple locations. The evaluation of a standardized design allows a check on multiple bridges across a railroad. The preliminary evaluation also determines which lengths may be prone to fatigue cycle accumulation and those that are not of concern. The major advantage of this method is determining which
The span length may be subject to one cycle per car, one cycle per train, or is not subject to any accumulative cycles. While it is a preliminary analysis, it is effective in determining which spans need further examination.

The method requires basic design data for both bridge spans and railcars. For span data, the method requires the design load, the design live-load impact, and the estimated dead load over the range of span lengths that are being investigated. For railcar data, the information needed is simply maximum axle weights and axle spacings that define the railcar layout. The assumptions for design live load are those used by the American Railway Engineering Association (AREA)/American Railway Engineering and Maintenance of Way Association (AREMA) from 1906 through the present. Design impact is associated with the time periods corresponding to the design loads. Dead load is represented by formulae available from published sources. Span lengths vary from 20 feet to 150 feet. Rail cars are represented by examples used in revenue service in North America concurrent with the spans being examined. This paper provides the methodology for the preliminary screening method applied to AREMA prototypical practice. The method is applicable to any standardized bridge design where the railcar equipment follows conventional layout.

The results of the analysis can provide dual benefit in relation to fatigue evaluation. If the purpose of the exercise is determination of spans requiring detailed inspection for fatigue-related issues, the results provide a convenient guide to the range of span lengths of a certain design that would benefit from a detailed inspection given a potentially high number of cycles to which the bridges may be subjected. At the same time, the need for detailed fatigue evaluation calculations can be validated with the same results. The method provides guidance for a number of spans instead of an evaluation on an individual basis.

**SPAN PROPERTIES**

**Live Load and Impact**

For this method, a full design of a span is not required. Instead of going through the full design procedure, a virtual span is created by calculating the section modulus needed for any span length. AREMA has always depended upon Allowable Stress Design (ASD). ASD allows straightforward calculation of required section modulus since the allowable stress is applied directly to the design moment without factoring.

For the majority of steel girder bridges, the combination of dead load, live load, and impact for vertical loading are the only necessary items to determine section modulus. Additional loads such as wind force usually do not contribute to design vertical loading since lateral forces receive an increase in allowable stress when combined with live loading.

With some assumptions, preliminary sizing of girders using AREMA criteria is simple. The two assumptions made for this method are 1) bridges are on tangent alignments and 2), the designs use open decks. The assumption of tangent alignment means that centrifugal force from live load does not have to be considered. The assumption of an open deck is more nuanced. Open decks result in a lighter dead load which decreases section modulus. Most deck girder bridges originally were built with open decks so that assumption is used for this study.

With the assumptions for the bridge spans, the spans are sized for section modulus simply by choosing a live-load level augmented by impact using a contemporaneous impact equation associated with that live-load level. With the addition of the dead load to the live load with impact, the total design moment is calculated and with an allowable stress level also associated with the live-load level.

For this study, the span lengths ranged from 20 to 150 feet in 5-foot increments. Multiple live load levels were used. AREMA has distinct periods of design level for steel bridges and has always recommended the Cooper E Load at various levels of load. The original 1906 load was the initial Cooper E40. Impact equations also evolved over time. Table 1 provides information concerning the years where Cooper design levels and impact changed in the AREMA recommendations.
TABLE 1. Time Periods for Design Levels (1-5)

<table>
<thead>
<tr>
<th>Time Period</th>
<th>Design Level (Cooper)</th>
<th>Impact Equation (Year)</th>
<th>Allowable Stress (ksi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1906-1919</td>
<td>E40</td>
<td>1906</td>
<td>16</td>
</tr>
<tr>
<td>1920-1934</td>
<td>E60</td>
<td>1920</td>
<td>16</td>
</tr>
<tr>
<td>1935-1947</td>
<td>E72</td>
<td>1935</td>
<td>18</td>
</tr>
<tr>
<td>1948-1967</td>
<td>E72</td>
<td>1948</td>
<td>18</td>
</tr>
<tr>
<td>1968- present</td>
<td>E80</td>
<td>1968</td>
<td>20</td>
</tr>
</tbody>
</table>

For each period shown in Table 1, impact formulas changed. Impact for the earlier periods were based on a theoretical approach to dynamic augment from steam locomotion. From 1935 on, bridge impact testing provided more realistic formulas. The 1948 and 1968 impact equations are actually the same, but the 1948 recommendations still showed steam locomotive dynamic augment equations as the primary impact for design with rolling impact secondary. By 1968 with the retirement of steam locomotives in North America, the rolling impact equations became primary for design impact. Figure 1 provides a graphical representation of design impact associated with the different design load levels.

FIGURE 1. Impact vs Span Length for Design Periods

The Cooper E Load levels shown in Table 1 were recommendations and the private railroad companies were not required to use them exactly as written. Prior to 1906 many railroads used their own proprietary design specifications. After the introduction of the Cooper Load as an AREA standard, some railroads opted to use a load level above the original E40 to satisfy specific needs. While differences existed between the original AREA recommendations and the private railroad design specifications, the method to perform the analysis in this study is the same.

Dead Load

Dead load of a girder bridge is a two-part effort, separated into the deck and the steel weights. Dead load of the decks can be easily determined. Girder spacing is the main variable. The girders are spaced based on stability criteria and the spacing (and deck weight) will increase when span length increases. For most railroads, deck weights are obtained from internal design data. Girder steel weight is a similar process, with preliminary design information considered adequate. Ketchum (6) provides tables and charts to determine
steel weights for girders and trusses while Hool & Kinne (7) use formulas with separate coefficients to take into account various factors.

The steel weights determined from either method can be adjusted to account for differences in design live load level or allowable stress. This may have an effect on long spans, but for short spans the ratio of live load to dead load is great and changes in dead load do not significantly alter the results.

**Total Section Modulus**
The section modulus is calculated by dividing the total combined bending moment of live load plus impact with the dead load by the allowable stress. Some adjustments and factors can be applied before determination of stresses due to railcar loading.

The calculated section modulus is that required to sustain the applied design load. This does not account for two items. The first item is corrosion, a consideration in ratings of older bridges. Corrosion can be handled by assuming a certain percentage of section loss on the girder and adjusting the section modulus by that percentage. Given the linearity of stress calculation, the stress calculated with full section modulus can be increased linearly to account for assumed section loss due to corrosion.

The second item addresses adjustment of live load stress based on research and tests. The adjustment can be approached in two ways. The first method relates to research by Sweeney (8) on span response and impact over a variety of span types and speed conditions. For girders the results showed that typical static live load response was 85 percent of the applied load while actual impact was 15 percent of the full design impact calculated from current AREMA impact equations. These factors are applied to the net section calculated for a girder for fatigue analysis. The other finding is from tests performed by Transportation Technology Center, Inc. (TTCI) (9). The tests demonstrated that a convenient method to calculate stresses due to live load plus impact is assumption of static live load without impact applied to the gross section of the girder.

The factors developed by Sweeney provide for impact and net section of a girder based on statistical analysis of data, while the assumption of static live load moment and gross section is based solely on observation of data. The results are only slightly different, but in the method described herein an assumption is made for the net/gross section ratio of 0.85. With an assumption for net section instead of a calculated result, the examples in this study use gross section and static live load moment. With accurate information on net section and corrosion, use of the factors developed by Sweeney are appropriate versus the convenience of gross section and static loads. With the use of static live load and gross section, the calculated section modulus is divided by the assumed net/gross section ratio to create gross section modulus used for the stress ranges in preliminary fatigue evaluation. Figure 2 provides a comparison of gross section modulus versus span length for the spans.

Figure 2 displays results that might be considered surprising. The curves describing the section modulus for both Cooper E40 and E80 are essentially identical. While Cooper E40 is a load system that is one-half the magnitude of Cooper E80, other factors reduce that difference. The first factor is the reduced impact in the 1968 primary equation versus the original 1906 equation. The second factor is the increase in allowable stress for 1968 E80 over 1906 E40. The final factor is the conversion of net section to the assumed gross section for the riveted E40 designs while the E80 designs are as welded (gross) sections. The largest sections in this figure are those using Cooper E60 with 1920 impact.
RAILCARS AND LOADING BEHAVIOR

Railcar Characteristics

Freight cars in North America follow familiar patterns regardless of the railroad company that may own them. Prior to the mid-1960s, boxcars were used for the majority of general traffic. Coal was shipped in small open hoppers, with liquids shipped in tank cars. Although differences in plans exist among railroads for the same type of car, the differences were minor which created little difference in dimensions. Weights of railcars in this period could vary because of the commodities being carried.

After the mid-1960s, limits on railcar weights increased in North America and this resulted in a variety of new railcar types designed to carry specific commodities. This resulted in a diversity of equipment with variations in length dimensions for railcars that would load bridges. While the length dimensions are variable for different railcars, the axle weights are similar since one of the purposes of design for specific commodities is maximization of axle weights.

Discrete axle loadings for railcars are not used for fatigue evaluation in the AREMA recommendations, instead relying upon the use of the Cooper E Load. The Cooper Load provides an accurate representation of steam locomotive loading effects but does not reflect current diesel or electric locomotives. The uniform load does not accurately represent actual railcar behavior. A relationship between axle spacings and span length in the creation of stress ranges was noted from previous testing results (10). While suitable for strength design and rating, the Cooper Load does not reflect fatigue behavior of railcars on railroad bridges. Figure 3 displays the dimensions for calculation of bending moments.
From Figure 3, the following definitions are provided for the dimensions shown:

- \( L_O \) – Overall length of railcar over couplers
- \( S_I \) – Inboard axle spacing between trucks
- \( S_O \) – Outboard axle spacing between rail cars
- \( S_T \) – Truck wheelbase between adjacent axles
- \( N \) – Number of axles

Table 2 provides typical dimensions and weights for the railcars used in this study. The vast majority of railcar equipment follows this railcar pattern. The railcars listed were/are heavily utilized so their examination provides a synopsis of most traffic during the life of current steel bridges.

### TABLE 2. Railcars Examined in this Study (12)

<table>
<thead>
<tr>
<th>Railcar Type</th>
<th>Year</th>
<th>N</th>
<th>( L_O ) (ft.)</th>
<th>( S_I ) (ft.)</th>
<th>( S_O ) (ft.)</th>
<th>( S_T ) (ft.)</th>
<th>Gross Rail Load (lbs.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>50-Ton 40-Foot Boxcar</td>
<td>1920</td>
<td>4</td>
<td>44.75</td>
<td>25.50</td>
<td>8.25</td>
<td>5.50</td>
<td>169,000</td>
</tr>
<tr>
<td>55-Ton Open Hopper</td>
<td>1900</td>
<td>4</td>
<td>34.50</td>
<td>16.50</td>
<td>7.00</td>
<td>5.50</td>
<td>169,000</td>
</tr>
<tr>
<td>70-Ton Open Hopper</td>
<td>1920</td>
<td>4</td>
<td>44.25</td>
<td>26.08</td>
<td>6.83</td>
<td>5.67</td>
<td>210,000</td>
</tr>
<tr>
<td>Heavyweight Passenger</td>
<td>1910</td>
<td>6</td>
<td>85.00</td>
<td>48.50</td>
<td>14.50</td>
<td>5.50</td>
<td>190,000</td>
</tr>
<tr>
<td>110-Ton Unit Coal Hopper</td>
<td>2000</td>
<td>4</td>
<td>53.08</td>
<td>34.67</td>
<td>6.75</td>
<td>5.50</td>
<td>286,000</td>
</tr>
<tr>
<td>110-Ton Grain Hopper</td>
<td>2000</td>
<td>4</td>
<td>60.00</td>
<td>39.92</td>
<td>8.42</td>
<td>5.83</td>
<td>286,000</td>
</tr>
<tr>
<td>110-Ton Cement/Sand Hopper</td>
<td>2000</td>
<td>4</td>
<td>42.00</td>
<td>23.67</td>
<td>6.67</td>
<td>5.83</td>
<td>286,000</td>
</tr>
<tr>
<td>Intermodal/Autorack Flat</td>
<td>1965</td>
<td>4</td>
<td>94.00</td>
<td>60.33</td>
<td>22.33</td>
<td>5.67</td>
<td>190,000</td>
</tr>
<tr>
<td>GE 6-Axle AC Locomotive</td>
<td>2000</td>
<td>6</td>
<td>73.17</td>
<td>33.08</td>
<td>13.75</td>
<td>6.58</td>
<td>432,000</td>
</tr>
</tbody>
</table>

### Bending Moment Behavior

As with any moving load system, bending moment varies over time as a train passes over a bridge. The shape of the time history depends upon the span length of the bridge that is being traversed compared to the length of the rail cars that are loading the bridge. This action is related directly to the number of fatigue cycles that may be accumulated during a train passage. This behavior is illustrated with an example of a train of two locomotives and ten cars crossing span lengths with distinct length characteristics. The loading characteristics of the train are a unit train of identical cars all equally loaded, with axle loadings of the locomotives matching the railcars.

Assume a span length \( (L_S) \) has a length with the relationship \( (S_O/2 + 2 \times S_T) < L_S < S_I \). For the railcars conforming to Figure 3, the result is where the span is loaded with four axles providing the maximum moment. The span is then completely unloaded so that dead load is the only load present. Figure 4 provides a computed time history at midspan for this condition.
The time history in Figure 4 shows a potential for one accumulative cycle per car if the stress produced by the maximum bending moment is sufficient. Maximum stress and stress range are synonymous for this span length. This is typical when the span length is less than any of the axle spacings.

The next assumption of span length follows the relationship $L_S > S_i$. With this relationship span length is greater than any individual axle spacing. The bridge span will have some live load on it at all times. An example of this condition is in Figure 5. The variation in live load moment shown in Figure 5 is significant and may be sufficient to develop an accumulative cycle for each railcar exceeding the applicable fatigue limit for the detail category. The time history also shows that some live load moment is being generated at all times while the train is on the span.

The final assumption for span length is $L_S >> L_O$. The bridge span length is much longer than the railcar length. This creates a large overall maximum moment with multiple rail cars on the span. The bending moment time history is displayed in Figure 6.
Figure 6 shows a different result from Figures 4 and 5. While variation in moment occurs during the train passage, the variation in moment is small compared to the maximum moment. The moment range from the railcars is insufficient to generate an accumulative fatigue cycle, meaning that the critical check is the stress range from the maximum moment from the locomotives and railcars. Assuming that the maximum moment is sufficient, it counts as only one accumulative cycle for the entire train versus for each railcar.

Figures 4, 5, and 6 display basic generalized bending moment behavior for railroad loadings. Research (13) and testing results (14) have shown that the variation in moment for unit-train railcar loadings, the
moment range of the railcars, is periodic in nature and is predictable given the ratio of the span length to the railcar length, \( L_s/L_o \). Additionally, moment range for railcars in a unit train has an absolute maximum value which initially occurs at \( L_s/L_o = 1.0 \), and reoccurs at each whole value of \( L_s/L_o \). Maximum moment will continue to increase as span length increases but moment range remains in a defined range.

Since an absolute maximum exists for moment range, two items create conditions where fatigue on a per railcar basis diminishes as span length increases. The first item is that with an absolute maximum for moment range, the proportion of moment range to the total maximum moment decreases which will decrease the stress ranges due to moment range. The second item is that as span length and maximum moment from the design Cooper Load increases, the overall section size increases serving to reduce the stress ranges.

**DETERMINATION OF ACCUMULATIVE CYCLES**

**AREMA Fatigue Criteria**

AREMA fatigue criteria follows North American practice similar to highway bridge design as far as individual fatigue details are concerned. Riveted railroad bridges have a separate rating fatigue curve in AREMA that includes multiple slopes for cycle life depending upon stress range level. Ultimately, high-cycle fatigue cycles are deemed accumulative if the stress range exceeds 6 ksi, the Variable Amplitude Fatigue Limit (VAFL) for riveted members coinciding with 20 million cycles. Depending upon the magnitude of stress ranges, fatigue category C or D is used to assess fatigue life.

Bridge fatigue follows the same criteria as design using AREMA Chapter 15, but little guidance is available for determining accumulative cycles under typical loads in a train passage. It is the responsibility of the engineer to determine the number of accumulative cycles.

**Evaluation of Railcars for Fatigue Stresses**

The critical item is determining if the spans are subject to one cycle per railcar or one cycle per train. For each span length the maximum moments and maximum moment ranges were calculated under static loading for each railcar type. The bending moments were calculated with an influence line analysis algorithm. The following figures illustrate examples of the results.

Figure 7 displays the current standard unit-train coal hopper against the range of span designs using Cooper E40 Load and 1906 impact. The figure shows the stress ranges for maximum moment and moment range. Figure 7 provides railcar information along with the plot of the stress ranges. The stress ranges are displayed by the solid line for maximum moment and the dashed line for moment range. A reference line shows 6 ksi for the VAFL. The graph includes reference lines for \( S_i \) and \( L_o \) to display salient points.
The line representing the stress range from the moment range for each railcar diverges from the line for stress range due to maximum moment, occurring at the point representing the span length $S_i$, the length where the minimum live-load bending moment is no longer zero. This indicates the point where the stress from maximum moment is no longer equal to stress range for an individual railcar. The stress range diminishes as span length increases and crosses the 6 ksi VAFL reference line close to the span length representing the overall length of the railcar. The graph displays that the unit-train coal hopper will generate one accumulative cycle per car for spans designed to E40 with 1906 impact up to a span length of approximately 55 feet. Beyond that span length, the railcar will generate one accumulative cycle for each train of these railcars, not for every railcar.

Figure 8 displays the opposite behavior where the stresses generated by the 50-ton 40-foot boxcar are insufficient for any accumulative fatigue cycles on an E60 design with 1920 impact. This preliminary check demonstrates that checking actual equipment in railroad operations can eliminate some railcars and save time when performing fatigue evaluations by eliminating certain equipment from consideration.
The same data can be displayed in a different format easier to interpret than what is shown in Figures 7 and 8. Figures 9 and 10 display the same results as Figures 7 and 8 with highlights presented in colors to easily differentiate the potential for fatigue cycle accumulation.
The use of colors allows for quick recognition of the four different zones that can be identified with rail-based bridge fatigue. The red region shows that stress ranges are sufficient to accumulate one cycle per railcar for the combination of railcar type and span design criterion. The yellow region shows that the stress ranges from each railcar diminished and only one cycle per train is now accumulative. The green region represents cycles per car that are not accumulative while the white region is for one cycle per train that is not accumulative. For Figure 9, the presence of both red and yellow regions demonstrates that in the shorter spans, one cycle per railcar is the accumulation rate for cycles, transitioning to one cycle per train at the same location as shown in Figure 7. Figure 10 demonstrates that since no red or yellow regions are present that no accumulative cycles are being generated for the boxcar.

The same analysis was performed for the other railcars listed in Table 2. The results of the analysis are summarized in Table 3. The results displayed are quite simple in this example. For analysis performed by the practicing engineer, the critical function is the calculation of the stresses for both maximum moment and moment range for the range of span lengths used for design criteria of bridges that are on any particular rail line. The graphical representation of the data allows for convenient reference when comparing the variety of railcar types. Table 3 provides a simple set of results divided between span lengths either less than or longer than L0. If the range of span lengths is susceptible to fatigue from the railcar being analyzed, it is marked with a “C” for railcar susceptibility, “T” for train susceptibility, or “--” for no susceptibility. The results for E80 are not included since E80 spans are welded or bolted with different fatigue criteria than riveted spans.

The use of L0 as a delineator in Table 3 is based on the results as illustrated in Figure 7. The current maximum axle loading used for unrestricted interchange in North America generates a stress on lighter design levels that creates the potential condition of one cycle per railcar until the span length is approximately the same as the railcar overall length. Beyond that, one cycle per train is the case. The location of the transition from one cycle per railcar to one cycle per train will change for different axle load levels or for different bridge design loads. Its use provides a convenient reference in this study when tabulating the results. Various methods can be implemented to display the results for the convenience and need of the bridge engineer. More detailed results can be obtained by displaying the results entirely by graphics which provides more nuance and detail in results than the simplicity of Table 3.
TABLE 3. Results of Fatigue Stress Preliminary Analysis

<table>
<thead>
<tr>
<th>Railcar</th>
<th>Design Loading</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>E40 1906</td>
</tr>
<tr>
<td></td>
<td>&lt; Lo &gt; Lo</td>
</tr>
<tr>
<td>50-Ton 40-Foot Boxcar</td>
<td>--</td>
</tr>
<tr>
<td>55-Ton Open Hopper</td>
<td>T T</td>
</tr>
<tr>
<td>70-Ton Open Hopper</td>
<td>C T</td>
</tr>
<tr>
<td>Heavyweight Passenger</td>
<td>--</td>
</tr>
<tr>
<td>110-Ton Unit Coal Hopper</td>
<td>C T C T</td>
</tr>
<tr>
<td>110-Ton Grain Hopper</td>
<td>C T C T</td>
</tr>
<tr>
<td>110-Ton Cement/Sand Hopper</td>
<td>C T C T C T</td>
</tr>
<tr>
<td>Intermodal/Autorack Flat</td>
<td>-- -- -- -- --</td>
</tr>
<tr>
<td>GE 6-Axle AC Locomotive</td>
<td>C T C T C T</td>
</tr>
</tbody>
</table>

C – One cycle per car  T – One cycle per train  -- No accumulative cycles

**CORROSION EFFECTS ON RESULTS**

The results shown in Table 3 present the case for stresses upon pristine sections that have not been subjected to the effects of corrosion. Corrosion will reduce the section modulus which will increase the maximum stresses and stress ranges from live load. The increase in live load stress ranges can have effects on fatigue life. For spans already subject to accumulative cycling, the stress increases will either increase the stress range for railcar cycling, or may change train cycling to car cycling. Also, for spans that with full section that might not be subject to cycle accumulation, corrosion can reduce enough section to create cycle accumulation.

Another outcome of corrosion on the bridge is that since the stresses increase from loss of section, the span length at which the stress ranges will intersect the 6 ksi reference line will also increase. As the stresses increase, the portion of the curve shown in Figures 7 and 8 that represents stress range due to each car will be shifted to the right which increases the span length at which that intersection occurs. The point at which the curve representing the stress range due to a single railcar deviates from the curve representing the stress range due to maximum moment, however, will remain the same. That point is defined by the S; dimension of the railcar.

**CONCLUSION**

The purpose of this research was to develop a method providing preliminary fatigue evaluation for a number of riveted steel girder spans instead of examining spans individually. A primary interest was the ability to use actual railcar configurations instead of design loads or assumed fatigue loads to examine the spans, with those spans sized according to historical design loads representing typical in-service bridges. Recent field testing of revenue service trains provided insight into expected stress levels so assumptions for analysis could be applied with confidence.

The process allows application of actual railcar configurations along with examining girder spans by design load criteria. This allows for evaluation of both the bridges from a fatigue viewpoint with determination of the effects of actual railcars on the bridges. The evaluation of railcar equipment includes review of past and present configurations, either removing or highlighting specific railcar types for consideration of fatigue cycle accumulation on any particular bridge span. The process also allows for examination of future proposed equipment that may be contemplated for use determining potential fatigue effects prior to placement in revenue service.

The method can be modified to include deductions for corrosion or evaluation of anticipated bridge span designs for fatigue. The data also can be displayed in any fashion that is appropriate for the assessment method used. The critical portion of the method is developing section moduli for the assumed design loads.
and calculation of maximum bending moments and moment ranges for the assumed rail car. This is usually a straightforward process that is within the scope of any engineer.

ACKNOWLEDGMENTS

Dr. Dick wishes to gratefully acknowledge the support of the Association of American Railroads/Transportation Technology Center, Inc. for production of this work. This work was funded under the AAR Strategic Research Initiative (AAR SRI) program. The author thanks TTCI for the opportunity to produce this work during his employment at the Transportation Technology Center.

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FIGURE 10. Stress Range vs Span Length for the 50-Ton 40-Foot Boxcar (alternate Figure 8)
Fatigue Screening of Railway Deck Girder Bridges

Stephen Dick, Senior Research Engineer, Purdue University
Duane Otter, Scientist, Transportation Technology Center, Inc.
Reasons for preliminary evaluation

• Many old bridges and many potentially accumulative cycles from service
• Bridge designs are convenient; standard designs
• Predictable railcar dimensions and axle loads
Reasons for preliminary evaluation

• Preliminary evaluation allows for elimination
• Pinpoints those which may have issues
• Determine whether the bridge accumulates fatigue cycles on the basis of:
  • one cycle per railcar
  • one cycle per train
  • not at all
Background for steel deck girder bridges

- Approximately 1/3 of all are steel deck girders
- Typically two girders per track for older spans
- Approximately 75% are 75 years old or older
- Approximately 50% are 100 years old or older
- Large majority are riveted
- Fatigue category D
Background for North American steel deck girder bridges

• Before 1906, bridge design specifications were proprietary by railroad or by design engineer
• After 1906, AREA published design recommendations for steel bridges using the Cooper E Load
• Use of Allowable Stress Design continues
• Since ASD is standard, comparison is straightforward
• Bridges designed as standards/classes
## AREA / AREMA design eras

<table>
<thead>
<tr>
<th>Time Period</th>
<th>Design Level</th>
<th>Impact Equation</th>
<th>Allowable Stress</th>
</tr>
</thead>
<tbody>
<tr>
<td>1906-1919</td>
<td>E40</td>
<td>1906</td>
<td>16</td>
</tr>
<tr>
<td>1920-1934</td>
<td>E60</td>
<td>1920</td>
<td>16</td>
</tr>
<tr>
<td>1935-1947</td>
<td>E72</td>
<td>1935</td>
<td>18</td>
</tr>
<tr>
<td>1948-1967</td>
<td>E72</td>
<td>1948</td>
<td>18</td>
</tr>
<tr>
<td>1968- present</td>
<td>E80</td>
<td>1968</td>
<td>20</td>
</tr>
</tbody>
</table>
Design Impacts

![Graph showing the impact of different span lengths for various hammer blows and rolling impacts.](Image)
Girder span data

• Section moduli for chosen range (20 – 150 ft)
• Designs for different design load levels
• Section modulus calculated from dead load plus live load plus impact
  • Dead load information available from older sources
Section modulus comparison
Section modulus comparison

• E40 and E80 curves are almost identical
• E40 – E72 spans are riveted, E80 welded
• Section modulus increased for E40 – E72 spans to account for gross section (15%)
• Differences in impact and allowable stresses make spans similar in size
Recent research

Sweeney (2018) published findings showing stresses from testing well below calculated values. Statistics from the data provide factoring for impact and span response.
Recent research

TTCI (2018) published independent results showing that live-load stresses can be closely estimated. Appropriate section and live-load assumed for preliminary analysis.
Railcar data

- Maximum and minimum live-load bending moments for the range of span lengths.
- Assumed unit train loadings, no live-load impact.
## Typical railcar dimensions

<table>
<thead>
<tr>
<th>Railcar Type</th>
<th>Year</th>
<th>n</th>
<th>(L_O) (ft)</th>
<th>(S_I) (ft)</th>
<th>(S_O) (ft)</th>
<th>(S_T) (ft)</th>
<th>GRL (lbs)</th>
</tr>
</thead>
<tbody>
<tr>
<td>50-Ton 40-Foot Boxcar</td>
<td>1920</td>
<td>4</td>
<td>44.75</td>
<td>25.50</td>
<td>8.25</td>
<td>5.50</td>
<td>169,000</td>
</tr>
<tr>
<td>55-Ton Open Hopper</td>
<td>1900</td>
<td>4</td>
<td>34.50</td>
<td>16.50</td>
<td>7.00</td>
<td>5.50</td>
<td>169,000</td>
</tr>
<tr>
<td>70-Ton Open Hopper</td>
<td>1920</td>
<td>4</td>
<td>44.25</td>
<td>26.08</td>
<td>6.83</td>
<td>5.67</td>
<td>210,000</td>
</tr>
<tr>
<td>Heavyweight Passenger</td>
<td>1910</td>
<td>6</td>
<td>85.00</td>
<td>48.50</td>
<td>14.50</td>
<td>5.50</td>
<td>190,000</td>
</tr>
<tr>
<td>110-Ton Unit Coal Hopper</td>
<td>2000</td>
<td>4</td>
<td>53.08</td>
<td>34.67</td>
<td>6.75</td>
<td>5.83</td>
<td>286,000</td>
</tr>
<tr>
<td>110-Ton Grain Hopper</td>
<td>2000</td>
<td>4</td>
<td>60.00</td>
<td>39.92</td>
<td>8.42</td>
<td>5.83</td>
<td>286,000</td>
</tr>
<tr>
<td>110-Ton Cement/Sand Hopper</td>
<td>2000</td>
<td>4</td>
<td>42.00</td>
<td>23.67</td>
<td>6.67</td>
<td>5.83</td>
<td>286,000</td>
</tr>
<tr>
<td>Intermodal/Autorack Flat</td>
<td>1965</td>
<td>4</td>
<td>94.00</td>
<td>60.33</td>
<td>22.33</td>
<td>5.67</td>
<td>190,000</td>
</tr>
<tr>
<td>GE 6-Axle AC Locomotive</td>
<td>2000</td>
<td>6</td>
<td>73.17</td>
<td>33.08</td>
<td>13.75</td>
<td>6.58</td>
<td>432,000</td>
</tr>
</tbody>
</table>

\(n\) - number of axles

\(L_O\) - Overall length of railcar

\(S_I\) - Inboard axle spacing

\(S_O\) - Outboard axle spacing

\(S_T\) - Truck axle spacing

GRL - Gross Rail Load
Bending moment time histories

Bending moment time history for
\[(S_O + 2 \times S_T) < L_S < S_I\]

Maximum moment and moment range are the same

One cycle per railcar
Bending moment time histories

Maximum moment and moment range are not the same

Possibly one cycle per railcar/train

Bending moment time history for $L_S > S_I$
Bending moment time histories

Bending moment time history for $L_S \gg L_O$

Moment range insignificant
Possibly one cycle per train
Determination of accumulative cycles

- Maximum moment over 6 ksi
- Divergence of maximum moment and moment range at $S_1$
- One cycle per car up to $\sim L_0$
- One cycle per train all span lengths
Determination of accumulative cycles
Determination of accumulative cycles

- Maximum moment stress nor moment range stress exceeds 6 ksi
- No accumulation of fatigue cycles
Determination of accumulative cycles

- Stress Range, ksi
  - 6 KSI VAFL Reference
  - E60 1920

- Span Length, ft
  - 50-ton 40-foot boxcar
  - Maximum Moment
    - 1 cycle per train
    - Non-Accumulative

- Moment Range
  - 1 cycle per car
  - Non-Accumulative
## Determination of accumulative cycles

<table>
<thead>
<tr>
<th>Railcar</th>
<th>Design Loading</th>
<th>E40 1906</th>
<th>E60 1920</th>
<th>E72 1935</th>
<th>E72 1948</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>&lt; L₀</td>
<td>&gt; L₀</td>
<td>&lt; L₀</td>
<td>&gt; L₀</td>
</tr>
<tr>
<td>50-Ton 40-Foot Boxcar</td>
<td>--</td>
<td>--</td>
<td>--</td>
<td>--</td>
<td>--</td>
</tr>
<tr>
<td>55-Ton Open Hopper</td>
<td>T</td>
<td>T</td>
<td>--</td>
<td>--</td>
<td>--</td>
</tr>
<tr>
<td>70-Ton Open Hopper</td>
<td>C</td>
<td>T</td>
<td>--</td>
<td>--</td>
<td>--</td>
</tr>
<tr>
<td>Heavyweight Passenger</td>
<td>--</td>
<td>--</td>
<td>--</td>
<td>--</td>
<td>--</td>
</tr>
<tr>
<td>110-Ton Unit Coal Hopper</td>
<td>C</td>
<td>T</td>
<td>C</td>
<td>T</td>
<td>T</td>
</tr>
<tr>
<td>110-Ton Grain Hopper</td>
<td>C</td>
<td>T</td>
<td>C</td>
<td>--</td>
<td>C</td>
</tr>
<tr>
<td>110-Ton Cement/Sand Hopper</td>
<td>C</td>
<td>T</td>
<td>C</td>
<td>T</td>
<td>C</td>
</tr>
<tr>
<td>Intermodal/Autorack Flat</td>
<td>--</td>
<td>--</td>
<td>--</td>
<td>--</td>
<td>--</td>
</tr>
<tr>
<td>GE 6-Axle AC Locomotive</td>
<td>C</td>
<td>T</td>
<td>C</td>
<td>T</td>
<td>C</td>
</tr>
</tbody>
</table>

C – One cycle per car  T – One cycle per train  -- No accumulative cycles
Effects of Corrosion

• Corrosion results in loss of section
• Stresses will increase with corrosion
• Length of affected span lengths may increase
• Point where cycling per car diverges is the same
Conclusions

• A convenient method to determine fatigue cycling for railroad loadings was developed
  • Uses basic design and loading criteria
  • Preliminary results but reduces further effort
  • Can eliminate span types/lengths
  • Can eliminate railcar types
Dr. Dick wishes to thank the Transportation Technology Center, Inc. (TTCI) for the opportunity to develop this work during his employment at TTCI. This work was funded under the Association of American Railroads Strategic Research Initiative (AAR SRI).
Thank you!

Any questions?