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Weigh-in-Motion-Based Fatigue Damage Assessment

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Abstract
Weigh-in-motion (WIM) data provide an excellent opportunity to study the effects of actual traffic loads on bridges. Here procedures are presented for using WIM data to quantify the fatigue damage accumulated in steel bridges. These procedures allow comparisons of the impacts of truck traffic on various routes beyond simple comparisons of the numbers and gross vehicles weights of trucks in the traffic streams. The fatigue damage accumulation procedures are demonstrated using WIM traffic data collected in the state of Alabama. The results of the analysis show that approximately 20% of trucks are overloaded, that is, permit loads and illegal loads, and those trucks create more than 50% of the total damage based on the combined data from all the WIM locations in the state. The contribution of overloaded trucks to the total fatigue damage varies so that their contribution is less significant along some routes. A typical steel bridge with bottom flange coverplates was evaluated using the WIM data from 1 year for a heavily traveled route. This analysis shows that the fatigue life of the bridge was consumed at an annual rate consistent with a mean life of 100 years. These procedures have applications in planning weight limit enforcement, budgeting, and maintenance, and they have the potential for future use in planning inspection intervals.

The service life of a bridge is affected by many factors such as, but not limited to, traffic loads, natural hazards, and defects in material production. Traffic-induced loads may cause damage to a bridge by fatigue, overload, or both. Steel bridges are generally more prone to fatigue cracking compared with other types of bridges, so steel bridges are the main focus of this paper. Every passage of a truck across a bridge creates one or more stress cycles in the structural components, which results in the accumulation of fatigue damage over time. During the service life of a steel bridge located on a busy highway, the bridge experiences millions of cycles of fatigue loading from heavily loaded trucks. If these stress cycles are of sufficient magnitude and number, they will likely result in fatigue cracking. The entire fatigue process in a member includes the formation of a fatigue crack, crack growth, and final failure (*1*). The number of stress cycles required for the formation of a fatigue crack is typically much larger than the number of cycles required to subsequently grow the crack to a size that will cause failure. After formation, if a fatigue crack is not detected and properly repaired, it may lead to failure of the member. So, in broad terms, the passage of each heavy truck uses a portion of the fatigue life of a bridge. The goal here is to quantify the damage produced by an individual truck and the accumulated damage resulting from many trucks.

The AASHTO *LRFD Bridge Design Specifications* (*2*) have a design approach for fatigue. The stress range calculated for a code-specified fatigue design truck is limited to avoid fatigue cracking caused by the accumulation of damage from repetitive truck loading. The AASHTO fatigue design truck is intended to represent truck traffic. However, over the service life of a bridge, there is uncertainty in the traffic loads that the bridge experiences. The procedure proposed here addresses the fatigue damage accumulated by bridges as a result of actual heavy truck traffic recorded at weigh-in-motion (WIM) stations. Background information is provided along with a review of the state-of-the-art from literature and the practices in the other states. Further, the methodology used in this paper and the implementation of that methodology are discussed.

The study of the impacts of vehicular traffic on infrastructure has been conducted in many states. Some states have sponsored research based on an experimental approach and other states have sponsored research based on analytical studies. In this section, some of the on-site

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field measurement studies are discussed first, and later the analytical studies.

Instrumentation and field measurements of fatigue stresses in steel girder bridges date back many years. Some of those early studies are documented by Fisher (*3*). In the early 1990s Laman and Nowak carried out a study for Michigan Department of Transportation (DOT) (*4*). Nowak et al. performed both experimental and analytical studies of the Woodrow Wilson Memorial Bridge (*5*). Field measurements were also done in Michigan for five steel bridges by Laman et al. (*6*) and a state-specific fatigue truck was proposed. In the early 2000s (*7*, *8*) an analytical and experimental study was conducted on 58 simple-span rolled-girder bridges, 18 continuous-span rolled-girder bridges, and six plate-girder simple-span bridges located in Birmingham, Alabama. Based on that study (*8*), the base metal at the end of the bottom flange cover plates was identified as the most fatigue-prone detail of those existing steel girder bridges. A recent study that includes procedures for assessing fatigue damage using measured strain data is reported by Fasl et al. (*9*).

Past analytical studies looked at various ways to assess the damage caused to bridges by traffic. Many studies have used simple beam line theory to assess the damage by running a single truck on an influence line. Most of these methods result in very conservative estimates of fatigue life. In 2012, Bowman et al. (*10*) performed a study aimed at improving the methodology for fatigue evaluation of bridges contained in Section 7 of the AASHTO *Manual for Bridge Evaluation* (MBE) (*11*). Interpretation of the S-N curves, average life-time single-lane average daily truck traffic (ADTT), and simplification of the structural performance and the variation of the site-specific truck loads were considered to be sources of conservatism of fatigue life prediction methods. The cumulative effect of these conservatisms is a significant underestimation of fatigue life. As an alternative, a closed-form solution for future traffic growth, previously developed by Fu et al. (*12*), was proposed for inclusion in Section 7 of the MBE. The multiple presence effect was studied using WIM data from seven states (California, Florida, Idaho, New York, Michigan, Texas, and Vermont). The effect of multiple presence on the fatigue evaluation results caused by the actual present ADTT, span length, and number of lanes was determined. These WIM data were also used to validate the gross vehicle weight (GVW) of the 54-kip fatigue truck developed in 1978 (*13*). The proposed remaining fatigue life evaluation procedure included the traffic volume growth rate, present ADTT, current age of the structure, and importance factor of the structure.

Most of the damage to bridges is caused by overweight (OW) trucks. The availability of an enormous WIM database collected by states has made it possible to analytically evaluate the damaging effect of the OW trucks on infrastructure, and many state DOTs have sponsored studies of these damaging effects. The impact of the heavy trucks on the bridges, and the potential impacts of increasing truck weight limits was studied by Fu et al. in 2003 (*14*). South Carolina DOT sponsored a study in 2013 to analyze the impact of heavy vehicle traffic on infrastructure and develop policy recommendations. Stakeholder interviews were done as part of the study (*15*). Ghosn et al. in 2015 (*16*) investigated the effect of OW trucks on the infrastructure of New York State. In modeling the effects of OW trucks on bridges, the WIM traffic data for OW trucks was categorized to probable divisible permits, special hauling permits, and illegals. The structural response to overweight trucks was considered using the overstress of main bridge members and cyclic fatigue damage accumulation. A study initiated by the Federal Highway Administration (FHWA) within the Moving Ahead for Progress in the 21st Century Act (MAP-21) (*17*) was partially focused on the analysis of the effect of the OW and oversize (OS) vehicle operation on bridges. Fatigue analysis was performed for the various fatigue-prone details of simply supported and continuous steel bridges. It was concluded that cost-wise the fatigue induced repairs were a non-significant portion of the total bridge cost. The Texas Department of Transportation (TxDOT) and the FHWA sponsored a study of infrastructure damage caused by OS and OW trucks to provide recommendations for permit fee adjustments if required (*18*). A methodology to quantify the pavement and bridge consumption rate per mile was developed as part of the study.

Technical Approach

Procedures were developed to use WIM measurements to calculate the level of accumulated fatigue damage for a specific bridge and to compare the damage accumulated in generic bridges characterized by their span length and the location of a cross-section of interest along the span. Damage accumulation was quantified for the period of time over which the WIM measurements were made. Traffic data for 1 year were used for this paper.

General Methodology

The nominal-stress life approach from the AASHTO *LRFD Bridge Design Specifications* (*2*) was used. The fatigue resistance of the detail was defined by an S-N relationship similar to that shown in Figure 1. The resistance relates the magnitude of the applied constant amplitude stress range (S) to the corresponding number (N) of cycles to failure of the detail. A family of similar

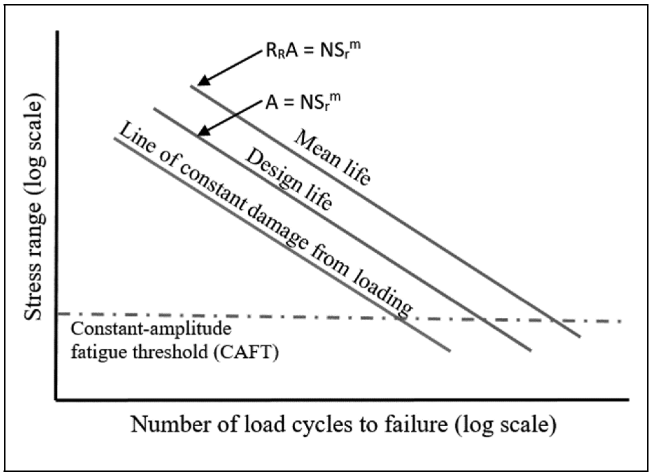


Figure 1. Fatigue life relationship.

S-N curves for different detail categories were established by extensive laboratory testing and is included in the AASHTO *LRFD Bridge Design Specifications* (*2*) for various categories of fatigue details that are commonly used in bridge construction.

The design life for a category of details is defined by the constant A, and the slope constant m which is taken as 3 for steel members according to the following relationship:

$$NS_r{}^m = A \tag{1}$$

The constant A defines a line on the log(N) versus log(S) plot that is at a constant value of fatigue damage. Values of A for the AASHTO fatigue detail categories are giving in Table 6.6.1.2.3-1 of the AASHTO *LRFD Bridge Design Specifications* (*2*). The mean life relationship can be obtained by multiplying the constant A by a resistance factor R_R as given in Table 7.2.5.1-1 of the MBE (*19*). The mean life is the statistically most likely fatigue life (*19*).

The S-N curves were developed using constant amplitude stress range test data. However, bridges are subjected to variable amplitude stress cycles. The Palmgren–Miner (*20*) rule provides a rational means to account for the cumulative damage from a spectrum of applied stress ranges of variable amplitude. Using Miner's rule, a well-known equivalent constant amplitude stress range, commonly referred to as an effective stress range S_{eff}, can be calculated as:

$$S_{eff} = \left[\sum \frac{n_i}{N} S_i^m\right]^{1/m} \tag{2}$$

where
n_i = number of cycles at the ith stress range, S_i
N = total number of cycles.

At a specific point along a bridge girder, the applied range of bending moment can be determined by multiplying the applied stress range by the section modulus. Multiplying each side of Equation 2 by the section modulus results in the effective moment range, M_{eff}:

$$M_{eff} = \left[\sum \frac{n_i}{N} M_i^m\right]^{1/m} \tag{3}$$

where
n_i = number of cycles at the ith moment range, M_i
N = total number of cycles.

Equation 3 was used to process the WIM data. Each WIM record provides the axle spacing and weights for a vehicle. These vehicles were analytically passed across a simple span to calculate the bending moment range. Each heavy vehicle (truck) in the WIM database was used for a specified span length to obtain the bending moment versus time at a specific location along the span. Some trucks in the WIM database create more than one cycle of loading as they cross a bridge. Rainflow cycle counting (*21*) was used to determine the number and magnitudes of the individual moment cycles resulting from the truck crossing. M_{eff} was then determined for the entire set of WIM records.

Two approaches are used in this paper to quantify the fatigue damage accumulated in steel bridges. One approach is a WIM site-specific damage measure, which is useful to quantify the damage in a generic girder bridge of a specified span length. The other approach is a measure of the fatigue damage at a specific fatigue-prone detail in a particular bridge. These two approaches are described in the next sections.

WIM Site-Specific Damage Ratio. This approach is useful in comparisons of the fatigue damage from truck traffic at different WIM sites. Comparisons of routes represented by those WIM sites are then possible. Also, the damage caused by different types or classifications of trucks in the traffic can be easily assessed by sorting the WIM records based on the truck configuration.

To provide comparisons in this paper using the largest amount of traffic data possible, all lanes of traffic in both directions of travel are combined. Comparisons are made between WIM sites where the number of traffic lanes is the same. Each WIM record from the traffic database is analyzed for 30, 60, 90, 120, and 200 ft span lengths. All results presented here are for a location 20% of the span length from the upstream support. This is a typical location for the end of a bottom flange cover plate. The upstream cover plate end location rather than the downstream end location was used because the damage at the upstream cover plate end is higher. Results for midspan and downstream cover plate end locations are reported by Babu et al. (*22*).

The amount of damage D_n is calculated from the effective bending moment determined from the WIM data using Equation 3 by:

$$D_n = NM_{eff}^m \tag{4}$$

It is important to note that the product of D_n and the section modulus raised to the power m is a constant that defines a line of constant damage from the applied loading as shown in Figure 1.

Damage at a Specific Fatigue-Prone Detail. Different details in a steel bridge experience stress ranges of various numbers and magnitudes, and therefore, some details accumulate fatigue damage faster than others. Based on the study performed by Franklin (*8*), the base metal at the end of a bottom flange cover plate is considered here as the most fatigue-prone detail in Alabama's steel girder bridges. The bottom flange of the girder at the upstream cover plate end is typically a Category E' detail as defined in the AASHTO *LRFD Bridge Design Specifications* (*2*).

For a specific fatigue-prone detail, an index of the accumulated fatigue damage D_m for a bridge along a route where the WIM data are recorded can be calculated using

$$D_m = \frac{N^*{S_{eff}}^3}{R_R^*A} \tag{5}$$

D_m is a Miner's fraction determined by dividing the accumulated fatigue damage caused by the trucks in the WIM data ($N{S_{eff}}^3$) by the value of fatigue damage defining the mean fatigue life (R_RA). This fraction is also equal to the fatigue life expended (N, total number of cycles) divided by the mean fatigue life ($R_RA/{S_{eff}}^3$). For example, a D_m value of 0.5 indicates that 50% of the mean fatigue life is expended by the traffic recorded in the WIM dataset. The D_m value is 1 when the mean life is expended.

The effective stress range, S_{eff}, for use in Equation 5 is

$$S_{eff} = \frac{M_{eff}^*GDF^*(1 + IM)^*P^*R_p}{S} \tag{6}$$

where

M_{eff} = effective moment range from WIM data by Equation 3

GDF = girder distribution factor for a single loaded lane

IM = dynamic load allowance

P = ratio of measured to calculated stress range

S = section modulus for the specific fatigue detail

R_p = multiple presence factor.

The effective moment range is calculated for the set of WIM records for the traffic in one lane and in one direction. This is consistent with fatigue design by the AASHTO *LRFD Bridge Design Specifications* (*2*). If WIM data are available for more than one lane in one direction, then all the records can be combined and the fraction of trucks assumed to be in a single lane can be based on Table 3.6.1.4.2-1 of the AASHTO *LRFD Bridge Design Specifications*. The girder distribution factor (GDF) and dynamic load allowance (IM) are calculated according to AASHTO *LRFD Bridge Design Specifications*, Articles 4.6.2.2.2 and 3.6.2, respectively. P is used to account for the commonly observed difference between the calculated and field-measured stress ranges. P is the ratio of the actual stress range to the calculated stress range. The ratio P is taken as 0.6 here and is based on results reported by Pearson (*7*) in which field tests were conducted on steel girder bridges representative of those studied here. This ratio may be improved in the future by more on-site field measurement studies. The multiple presence factor, $R_{p,}$ is calculated according to MBE (*19*), Equation (7.2.2.1-1).

Values of D_m described and reported in this paper are for a period of 1 year. Values of D_m (and D_n) are additive. For example, if values of D_m are calculated from WIM data each month for convenience of data processing, the D_m value for the year is the sum of the values calculated for each month. Similarly, the values of D_m for multiple years can be added together.

Finite Life Check. An important question that must be addressed in the discussion of fatigue damage accumulation is: do all trucks, or traffic-induced stress cycles, contribute to the accumulation of damage and potential formation of a fatigue crack? Current U.S. practice is that all stress cycles are considered to contribute to fatigue damage if there is a sufficient percentage of the traffic-induced stress ranges above the constant amplitude fatigue threshold (CAFT). When this is the case, the finite life relationship between stress range and fatigue life is defined by Equation 1 as is assumed in the procedures described in this paper. Otherwise, the fatigue detail has an infinite life, and the accumulation of fatigue damage is inconsequential. In this paper, AASHTO Category E' details are the primary focus for which Fisher et al. (*23*) show that only 1 in 2,000 stress cycles must be above the CAFT to result in fatigue cracking. Field measurements (*7*) have confirmed a significant number of stress cycles above the CAFT in Alabama bridges at E' details. Thus, the procedures described in this paper are judged as valid for the calculation of fatigue damage, and all vehicles in the WIM data can be considered. However, it is also common practice in the analysis of WIM data to omit light vehicles because of the extremely large number of vehicle records typically recorded, and the amount of time that is required to include the light vehicles in the analysis. Filtering the

Table 1. Number of Records in the Weigh-in-Motion Database for the Year 2014

WIM site code	County	Number of WIM records before QC	Number of WIM records after QC
911 (US280)	US280 Sosa Co.	1,092,751	357,854
931 (I65)	I65 Limestone Co.	3,655,980	1,584,347
933 (AL157)	AL157 US72 Colbert Co.	977,580	427,505
934 (US78)	US78 Walker Co.	688,388	169,407
942 (US231)	US231 Montgomery Co.	1,262,375	787,426
960 (US84)	US84 Clark Co.	521,484	305,566
961 (I65)	I65 Mobile Co.	2,136,008	851,213
964 (US231)	US231 Dothan Co.	1,217,687	642,337
Total		11,552,253	5,125,655

Note: WIM = weigh-in-motion; QC = quality control.

lightest vehicles out of the WIM data is often justified by the small contribution of light vehicles to the total damage is small. In the work reported here, vehicles with GVW less than 20 kips were omitted from the analyses. Research is ongoing to develop improved finite life check procedures to be applied to the spectrum of bending moment ranges obtained from the analysis of WIM data.

Traffic Database

To demonstrate the fatigue damage accumulation procedures described in the previous sections, traffic data collected in the state of Alabama are used. Traffic data consist of a database of the WIM records and a database of single-trip permits issued for OW vehicles. Using these databases allows comparisons of fatigue damage caused by legal loads, permit loads, and illegal loads. Additional discussion of these traffic databases is provided in the next sections.

WIM Database

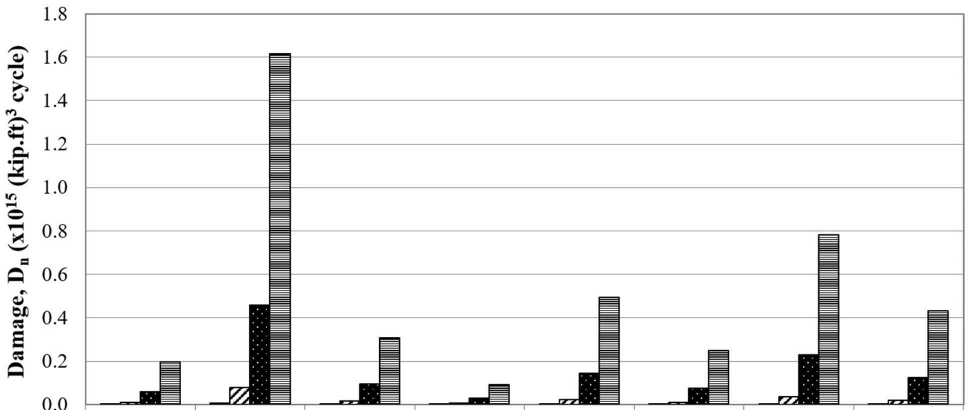
WIM data collected from eight locations around the state of Alabama are used in this study. All WIM sites except one were equipped with a permanent bending plate system consisting of two scales and inductive loops (*24*).

The WIM data for the year 2014 are considered in this paper. A summary of all available records for each WIM site is provided in Table 1. The following information was recorded for each vehicle in the WIM database: time of record, the direction of travel, GVW, vehicle type, axle spacing, axle loads, and vehicle speed. Two types of errors can occur in long-term WIM data collection: random errors (occurring once in a while), and systematic errors (occurring frequently and affecting more records) (*25*, *26*). The errors are usually associated with a malfunctioning of the WIM system, misrecording, non-typical vehicle configuration or lateral position with regard to the sensor, and other causes (*27*, *28*). A site-specific quality control (QC) algorithm was used to improve the quality of the data set by eliminating errors. A detailed description of the QC checks is provided by Babu et al. (*22*). The number of records before QC and after QC is shown in Table 1.

Permit Database

The Maintenance Bureau of the Alabama DOT (ALDOT) issues about 500–600 permits per day. About 200 of them are permits for OW vehicles. The permit data for the year 2014 are considered in this paper. The annual reports are in the form of tables, and they include the permit ID, the validity of the permit, original and final destination, authorized roads, description and FHWA class of vehicle, GVW, axle load, and axle spacing. Annual permits are issued, but each trip accomplished within the annual permit is also listed as a separate row in the database. The total number of issued permits was 123,603 for the year 2014.

To identify permitted trucks in the WIM database, the first step was the separation of legal traffic based on the truck weight regulations for the state of Alabama (*29*). The remaining database included only permit vehicles and illegal traffic, that is, overloaded trucks. Next, the database of permits issued by ALDOT for OW vehicles was processed together with the WIM database of overloaded trucks using the developed routines and Google Maps API. WIM vehicles without permits were sorted out, and the remaining vehicles were considered to be illegal traffic. A comparison of the permits identified in the traffic stream with the vehicles that required permits indicates that less than 0.5% of OW vehicles operate with an OW permit. More detailed discussion of this procedure is provided by Babu et al. (*19*)



	Site 911	Site 931	Site 933	Site 934	Site 942	Site 960	Site 961	Site 964
30 ft	0.001	0.006	0.001	0.000	0.002	0.001	0.003	0.002
60 ft	0.010	0.079	0.016	0.006	0.024	0.012	0.038	0.021
90 ft	0.059	0.458	0.094	0.031	0.145	0.075	0.228	0.125
120 ft	0.198	1.614	0.309	0.093	0.493	0.250	0.782	0.434

Figure 2. Accumulated damage, D_n at the upstream cover plate end.

Comparison of WIM Site-Specific Damage

The amount of damage ($D_n = NM_{eff}^3$) was calculated for each WIM site (911–964) for both traffic directions combined and for the year 2014 for the upstream cover plate end. All the WIM sites had two lanes of traffic in each direction (Lane 1 & 2 and Lane 3 & 4). The accumulated damage for each WIM location is calculated for 30, 60, 90, 120, and 200 ft span lengths for the upstream cover plate end at 0.2 L, where L is the span length. To cover steel girder bridges without cover plate end, similar study was performed considering steel girder bottom flange at midspan (C type detail) as the most fatigue critical (*19*).

The results for the upstream cover plate end are shown in Figure 2. The accumulated damage at WIM site 931 was greater than the accumulated damage at all the other WIM sites. WIM site 931 was followed by WIM site 961 and 942. The accumulated damage increases rapidly as the span length increases for all the WIM sites. Because of this rapid increase with span length, the results for the 200 ft span are not shown in Figure 2 so that the results for the shorter spans are more visible in the plot. Results for the 200 ft span are presented separately in Figure 3.

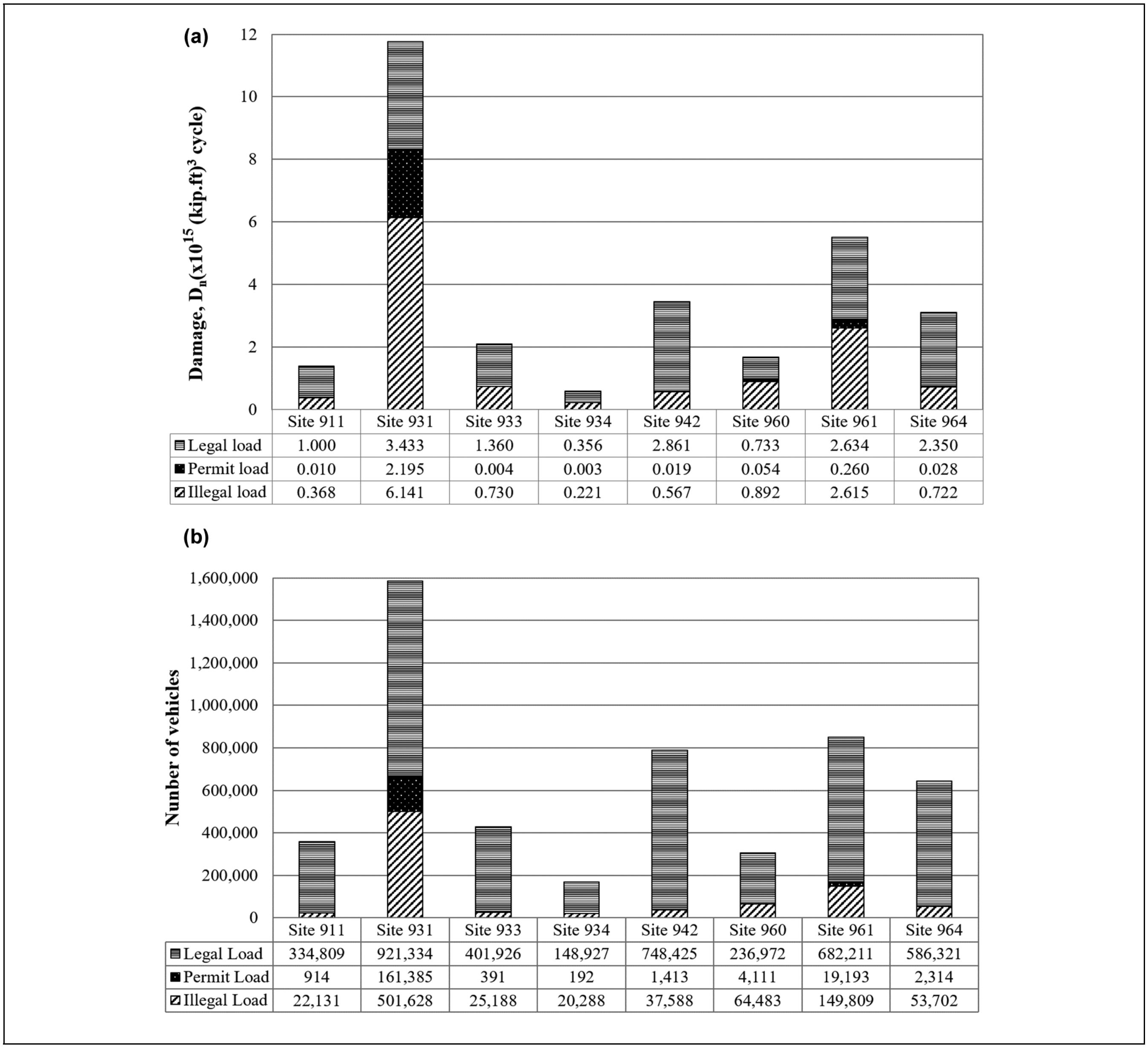
Further, the truck traffic data were sorted into legal loads, permit loads, and illegal load categories. One of the objectives for sorting the traffic data was to assess how the damage compared for these types of vehicles. The corresponding accumulated damage (NM_{eff}^3) was computed for each vehicle category.

Legal loads include vehicles that comply with Alabama's legal regulations (*29*), "grandfather exceptions," and annual permits that have a GVW less than 100 kips. The OW vehicle category covers vehicles that require an individual trip permit to travel legally because of their weight or axle load combination and annual permits that have a GVW greater 100 kips. Further, the OW vehicle category was sorted into permit loads and illegal loads. The identification of legal loads, permitted, and illegally overloaded vehicles adopted in this paper is discussed in detail by Babu et al. (*22*).

Results shown by Babu et al. (*22*) illustrate that the relative contributions of different categories of vehicles are similar for all these span lengths. The accumulated damage at the upstream cover plate end in the 200 ft span for the corresponding vehicle categories is shown in Figure 3*a*. The corresponding number of vehicles is shown in Figure 3*b* for the year 2014. The accumulated damage from illegal loads at WIM sites 931, 961, and 960 was about 50% of the total damage for those WIM sites.

Comparing the pie charts shown in Figure 4, the proportion of the vehicles in each category is significantly different from the proportion of the damage caused by those vehicles. For example, legal vehicles make up 79% of the total truck traffic, but they create only 50% of the total damage. At the same time, OW trucks (legally and illegally) are responsible for another half of the damage from the total truck traffic stream.

In Figure 4, the traffic from all WIM sites is combined for the year 2014, and it can be concluded that the 17% of trucks that are illegally overloaded (Figure 4*a*) create more than 40% of the total damage (Figure 4*b*). These results are state specific and can vary from state to state.



	Site 911	Site 931	Site 933	Site 934	Site 942	Site 960	Site 961	Site 964
Legal load	1.000	3.433	1.360	0.356	2.861	0.733	2.634	2.350
Permit load	0.010	2.195	0.004	0.003	0.019	0.054	0.260	0.028
Illegal load	0.368	6.141	0.730	0.221	0.567	0.892	2.615	0.722

	Site 911	Site 931	Site 933	Site 934	Site 942	Site 960	Site 961	Site 964
Legal Load	334,809	921,334	401,926	148,927	748,425	236,972	682,211	586,321
Permit Load	914	161,385	391	192	1,413	4,111	19,193	2,314
Illegal Load	22,131	501,628	25,188	20,288	37,588	64,483	149,809	53,702

Figure 3. (*a*) Accumulated damage, D_n at the upstream cover plate end for the 200 ft span and (*b*) numbers of vehicles.

Damage at a Specific Fatigue-Prone Detail

Studies conducted by Pearson (*7*) and Franklin (*8*) included evaluation and instrumentation of representative spans from among 58 simple-span rolled-beam bridges, 18 continuous-span rolled-beam bridges and six plate-girder simple-span bridges in downtown Birmingham. Stress ranges were calculated at the following fatigue-prone details on the bridges: transverse diaphragm connections, longitudinal cover plate fillet weld connections, shear connectors, and cover plate ends at the upstream and downstream locations. The study concluded that the base metal at the ends of the cover plates was the most critical fatigue detail of those bridges. The bottom flange of the girder at the cover plate end is a Category E' detail as defined in the AASHTO *LRFD Bridge Design Specifications* (*2*).

To demonstrate the procedures developed in this paper, the damage accumulated at this specific fatigue-prone detail, E', for the Span 86-W as described by Franklin (*8*) was selected. This bridge was selected because it is a real bridge, typical of steel girder bridges on highways in Alabama. Also, as an example, WIM data from site 931 were used. This is real traffic, and a real bridge, although this traffic does not cross this particular bridge. Damage accumulated at the upstream

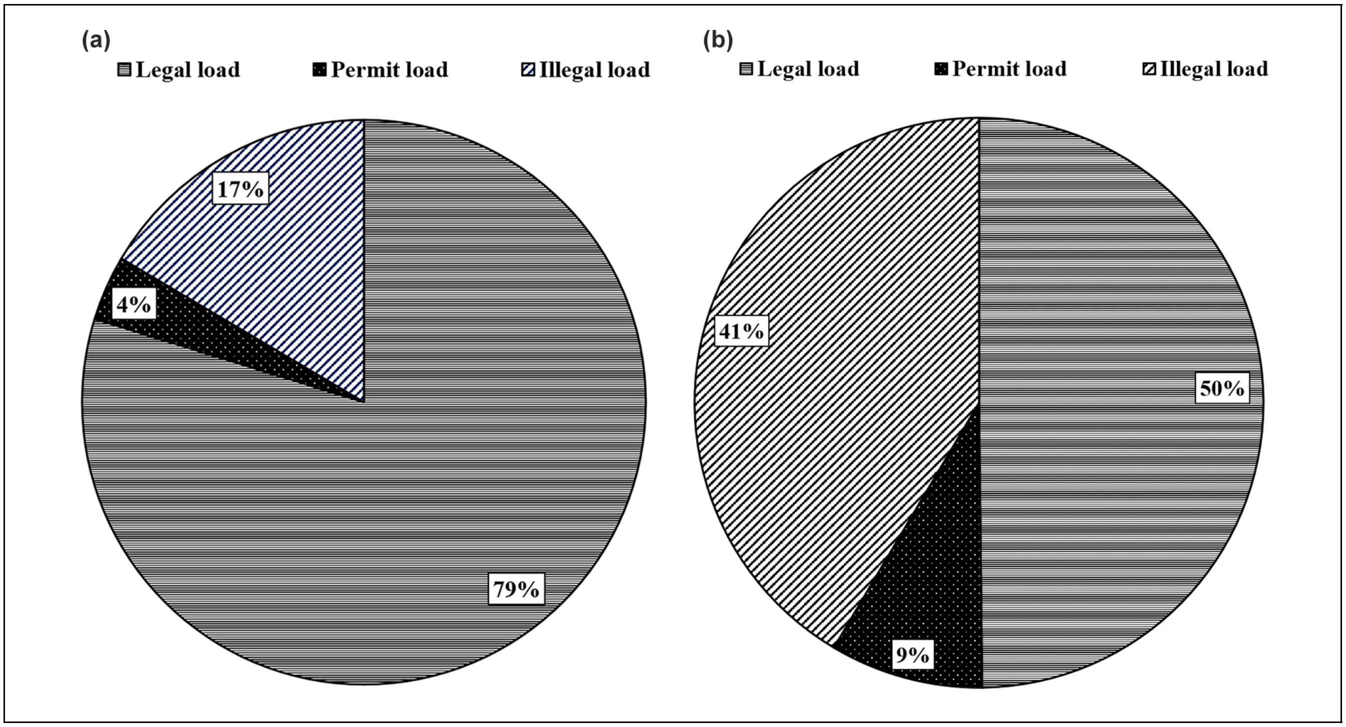


Figure 4. (*a*) The total number of vehicles and (*b*) the accumulated damage, D_n for all WIM sites combined at the upstream cover plate end for the 200 ft span.

cover plate end from the passage of traffic in one of the directions (lane 1 and 2) at WIM site 931 for the year 2014 was calculated.

Span 86-W consists of eight W36 × 150 rolled section girders spaced at 8.71 ft. On average, the total length of the beams is 66.30 ft, and the approximate average span length is 60 ft. The cover plate size is 10" × 0–15/16" × 41"–6". Based on this information, the bridge data inputs to estimate the damage accumulation were calculated and listed in Table 2.

Using Equations 5 and 6, and the bridge data inputs from Table 2, the fraction of mean fatigue life expended, D_m, at the upstream cover plate end is 0.01. Thus, this 1 year of traffic expends 1% of the mean fatigue life of this fatigue-prone detail. If this annual traffic was applied to the bridge in each year of its life, then its mean life would be ($1/D_m$) or 100 years. For comparison, the fraction of fatigue life expended, D_m, determined according to AASHTO *MBE* is equal to 0.008. Therefore, the corresponding mean fatigue life is 120 years. For this specific case, the traffic represented by the WIM data produces more damage than is accounted for by the procedure in the MBE. This will not always be true.

Table 2. Bridge Data Inputs for Span 86-W Bridge on Interstate I-59/20 in Birmingham, Alabama

Section modulus (S)	702 in.^{3}
Span length (L)	60 ft
Girder distribution factor (GDF)	0.51
Dynamic load allowance (IM)	0.15
Location of upstream cover plate end	11.0 ft
x/L of upstream cover plate end	0.2
Location of downstream cover plate end	53.6 ft
x/L of downstream cover plate end	0.2
Number of traffic lanes	4
Direction of traffic	One direction only
Fraction of truck traffic (p)	0.85
Resistance factor for mean fatigue life for E' detail (R_R)	1.9
Ratio of measured to calculated stress range (P)	0.6
Average daily truck traffic (ADTT)	2809
Number of lanes (n_L)	4
Bridge location	Birmingham, AL

Conclusion

A procedure to quantify the fatigue damage accumulated in steel bridges using WIM data was demonstrated using combined WIM data collected in 2014. The procedure can be used to assess WIM site-specific damage or bridge-specific damage. The procedure can be used for WIM traffic data from any state. In this paper, the procedure is demonstrated using WIM data from the state of Alabama. The following conclusions can be made:

1. Comparisons of accumulated fatigue damage are more direct comparisons of the effects of heavy vehicles on steel bridges than simple comparisons of GVWs and numbers of vehicles.
2. WIM site-specific damage gives a way to compare the effects of the traffic streams along different routes.
3. The use of WIM data for calculation of accumulated fatigue damage at specific fatigue-prone details avoids the use of a standard fatigue truck. This approach is best suited for bridges near the WIM site.
4. By using the procedure presented here, the damage (D_m) calculated at a specific fatigue-prone detail in a particular bridge is equal to the percentage of the mean life expended.
5. Analysis of accumulated damage caused by legal loads, permit loads, and illegal loads allows comparisons of the relative damage from different vehicle categories. This approach can also be applied to compare the effects of the vehicles broken down by other classification schemes such as the total number of axles.
6. From the analysis of WIM data from the state of Alabama, 17% of vehicles that are illegally overweight create more than 40% of the total fatigue damage.

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Author Contributions

The authors confirm contribution to the paper as follows: study conception: J.M. Stallings, A.S. Nowak; data processing: O. Iatsko, A. Ramesh Babu; analysis and interpretation of results: J.M. Stallings, A. Ramesh Babu, O. Iatsko, A.S. Nowak; draft manuscript preparation: O. Iatsko, J.M. Stallings, A. Ramesh Babu. All authors reviewed the paper and approved the final version of the manuscript.

Data Availability Statement

The traffic data that support the findings presented in this paper are available from the corresponding author on reasonable request.

Declaration of Conflicting Interests

The author(s) declared no potential conflicts of interest with respect to the research, authorship, and/or publication of this article.

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