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A REVIEW OF THE HL-93 BRIDGE TRAFFIC LOAD MODEL USING AN EXTENSIVE WIM DATABASE

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Abstract
HL-93, the current bridge traffic load model used in the United States, is examined here. Weigh-in-motion from 17 sites in 16 states, and containing 74 million truck records, is used to assess the level of consistency in the characteristic load effects implied by the HL-93 model. The load effects of bending moment and shear force are considered on single-lane and two-lane same-direction slab and girder bridges with a range of spans. It is found that the ratio of WIM-implied to HL-93 load effect varies considerably from one load effect to another. An alternative model is proposed which achieves improvements in consistency in this ratio for the load effects examined, especially for the single-lane case. The proposed model consists of a uniformly distributed load, whose intensity varies with bridge length.

Subject Headings: Bridges; traffic; loads; highway bridges; trucks.

1. Introduction
Accurate representations of imposed loads are needed for the efficient design of bridges. They ensure that new bridges are safe while eliminating the waste associated with over-design. Load models are used for everyday design and assessment of bridges and different models exist. The AASHTO model (HL-93) consists of a load per unit bridge length plus a three-axle truck or tandem axle load, whichever gives the greater load effect (AASHTO 2010). The basic load model used for bridge design in Europe consists of a uniformly distributed load (UDL) plus a concentrated tandem axle load (EC1 2003). Previous European models have also used knife edge loads (BS5400 2006).

The AASHTO LRFD Bridge Design Specifications (AASHTO 2010) are used for bridge design in the United States. Traffic loading for standard and routine permit vehicles is represented by the HL-93 load model. This is intended to generate the 75-year return period load effect for “normal vehicular use of the bridge” (AASHTO 2010). NCHRP report 683

(Sivakumar et al. 2008) interprets this as “all legal trucks, illegal overloads and un-analyzed permits (all routine permits)” as they represent normal service traffic. These trucks are referred to in the specifications as “Strength I” loading. A 75-year mean maximum is used as bridges are designed for a 50- or 75-year design life (Nowak & Szerszen 1998). The load model is valid for two lanes; multiple presence adjustment factors are specified for bridges with other numbers of lanes (Nowak 1999).

Load models are ideally calibrated using measured truck weights from Weigh-In-Motion (WIM) sites. WIM data was used to calibrate the US and European bridge design load models (Nowak 1999; O’Connor et al. 2001). NCHRP Report 683 details a method to adjust the US HL-93 model using site-specific WIM data to obtain a site-specific load model for design. Other work which develops load models using WIM data includes that of Crespo-Minguillón & Casas (1997) who describe a general model applicable to a broad range of applications and Getachew & OBrien (2007) who expand upon the idea of Turkstra’s Rule to produce a simple model for the assessment of two-lane bidirectional bridges. Researchers in Hong Kong have developed a city-specific load model containing a family of trucks (Miao & Chan 2002). The need for accurate WIM data when calibrating a load model is shown by a study of Wisconsin WIM data (Zhao & Tabatabai 2012). The study examines the permit vehicle load model which is used for permit evaluation in Wisconsin. It shows that 0.1% of short five-axle vehicles produce load effects that exceed those of the permit vehicle model.

The HL-93 load model used in the United States was calibrated using a database of 9250 trucks weighed in Ontario, Canada in 1975 (Nowak 1999; Nowak 1993). The far greater quantities of WIM data that are available today (Fu & You 2011; Nowak & Rakoczy 2012) facilitate a critical review of the model. This study uses an extensive WIM database containing 74 million trucks from 17 sites across the US. The large database used allows more accurate estimates of characteristic load effects and an analysis of the variation of characteristic load effects from site to site. The estimated load effects are compared with those of HL-93 to examine the level of consistency provided by the current load model. An alternative model is proposed which provides a more accurate representation of actual bridge loading. It aims to achieve a consistent level of conservatism for all traffic induced load effects at a given site. This facilitates more efficient design of bridges, as it reduces over-design for certain load effects and limits under-design for other load effects.

1.1 Weigh-In-Motion Data

Single-lane loading is assessed using data from the United States Federal Highway Administration’s Long-Term Pavement Performance (LTPP) Program. Under the program, data is collected on the slow lane, in one direction only, at each site. The data is collected under strict guidelines to ensure it is of “research-quality” (Walker & Cebon 2012; Walker et al. 2012). The data was collected in the period 2005-2011 and contains 65.7 million trucks from 17 sites in 16 states (Table 1). The first site came online in 2005 and by July 2008 the WIM systems at all the sites were collecting data.

Table 1. Details of the single-lane sites in WIM database.

Site	No. Weekdays	Average Trucks Per Weekday	MMW GVW[1]	No. Trucks > 100t	No. Trucks ≥10 Axles	Permit Threshold (t)[2]
1. AZ, I-10 East	996	4988	93	62	1264	48.3
2. AR, I-30 North	972	5526	100	171	3532	46.3
3. CA, SR-99 North	970	5939	68	8	131	54.4
4. CO, I-76 East	1020	1473	73	38	227	57.6
5. IL, I-57 North	1008	3139	98	142	2210	45.4
6. IN, US-31 North	870	1489	69	1	140	49.0
7. KS, I-70 West	1005	1851	81	26	406	43.1
8. LA, US-171	993	506	77	23	166	49.0
9. ME, I-95	915	835	67	1	120	59.0
10. MD, US-15 North	979	1030	51	0	10	49.9
11. MN, US-2	1016	316	67	10	119	41.7
12. NM, I-25 North	929	716	61	1	49	47.2
13. NM, I-10 East	861	2934	89	50	793	47.2
14. PA, I-80 West	973	5315	90	66	1249	52.6
15. TN, I-40 West	994	5474	89	41	1248	45.4
16. VA, US-29 South	998	1082	63	3	152	49.9
17. WI, US-29 South	1004	987	73	10	240	45.4

[1]MMW GVW = mean maximum weekly gross vehicle weight

[2]Above this state-specific threshold, five axle trucks are required to have special permits.

The LTPP data contains only single lanes in one direction at the sites. At some sites data is also gathered for the other lanes, although it is not subject to the same quality control measures as the LTPP lanes. This latter data is used, along with the LTPP data, to examine loading for two-lane same-direction traffic. As some quality assurance issues were identified for certain periods with some of the non-LTPP lanes, not all available data was used. With the questionable data removed, 8.4 million non-LTPP truck records were available for the two-lane analysis – see Table 2.

Table 2. Details of the two-lane same-direction WIM sites.

Site	Location	Years of Good Data	Trucks/Day Slow Lane	Trucks/Day Fast Lane
1[1]	IN, US-31 North	1.0	1489	158
2a	TN, I-40 East	2.4	5626	1225
2b	TN, I-40 West	5.4	5474	1107
3	VA, US-29 South	6.0	1082	134

[1] The southbound direction (non-LTPP lane) was also available for site 1 but it was not used.

After cleaning of the WIM data at this site some very heavy tandem weights were discovered which seemed unrealistic in the context of the other axles on the truck.

Most WIM systems register a certain number of erroneous records (Sivakumar et al. 2008). To allow accurate statistical analysis and simulation, these records must be removed. Different anomalies occur at different WIM sites and data cleaning rules must be adjusted depending on the data. The rules used in this work are based on a previous set of cleaning rules used to prepare European WIM data for long run simulations (Enright & OBrien 2013; Enright & OBrien 2011).

2. Methodology

2.1 Filtering permit trucks

To examine the HL-93 load model, a method is needed to identify Strength I trucks in the WIM data. Strength II trucks (special permit) are not of interest here as HL-93 does not cover these vehicles. Classification schemes do not generally distinguish between standard and permit trucks (COST323 2002; FHWA 1995). Some bridge assessment guidelines (Highways Agency, 2011; O'Connor & Enevoldsen, 2009) do make this separation but do not describe a method for identifying these trucks in the WIM data. NCHRP Report 683 (Sivakumar et al. 2008) notes that it is difficult to identify Strength I and Strength II trucks when examining WIM data. It makes the simplifying assumption of putting everything with 6 axles or less in the Strength I (normal traffic) category and everything else in Strength II (special permit). This approach misclassifies some routine permits, trucks with grandfather rights (USDOT 2000), specialized haulage trucks (Sivakumar et al. 2008) and Long Combination Vehicles (Schulman 2003), all of which can have more than 6 axles but which are in the Strength I category (see Figure 1). The report does however conduct a sensitivity analysis to assess the importance of correctly identifying these truck groups. It finds that the two-lane case is not as sensitive as the single-lane case as it is examining two trucks side-by-side. It concludes that the most precise and rational method for identifying Strength I and Strength II trucks, is to use state-specific permit and weight regulations. This is the approach that is adopted here.

The filtering of Strength I trucks contains two steps. Rules previously developed by some of the authors (OBrien, Enright, et al. 2013; Leahy 2014) are used to separate the trucks into standard and permit trucks (marked with dash-dot lines in Figure 1). This first step is based solely on axle configuration; weight is not used initially as illegally overloaded standard trucks should be included in the standard truck category. These rules, however, do not separate the permit trucks into routine and special (see Figure 1) and a further weight-based filtering step is required to do this. The routine permit weight limits in the Comprehensive Truck Size and Weight Study (USDOT 2000) are used to determine if the permit trucks are routine or special permits. Unfortunately the study only provides the state-by-state upper weight limit for routine permits with five axles. This weight limit is applied here to all permit trucks to determine if they are routine or special permit. Although this method is not perfect, it avoids some of the misclassifications in the method used in NCHRP Report 683 (Sivakumar et al. 2008).

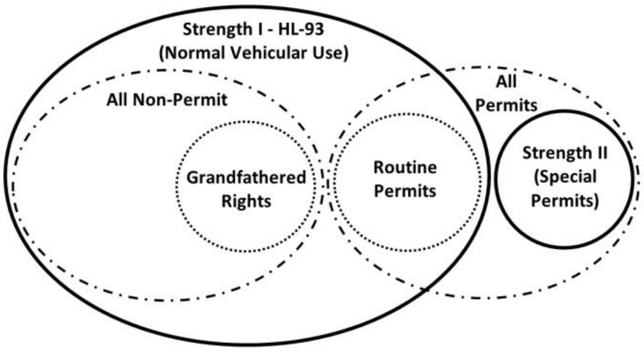


Figure 1. Different vehicle categories. Non-permit trucks include regular trucks below the legal limits and those allowed to exceed those limits through 'grandfather rights' in some states. HL-93 is deemed to include all of these non-permit trucks plus those with routine permits. It excludes those with special (non-routine) permits.

2.2 Single-Lane Load Modeling

For a single-lane bridge, three load effects are examined on 20, 30, 40 and 50 m total bridge lengths – mid-span bending moment on a simply supported bridge (LE1), shear at the end support on a simply supported bridge (LE2) and hogging moment over the central support of a two-span continuous bridge (LE3). Simple beam influence lines are used as they are considered to be a sufficiently accurate model of the transverse distribution of load on a single-lane bridge. The accuracy of the HL-93 load model is assessed by first calculating the 75-year characteristic load effects at all sites. The 75-year values are obtained by extrapolation from the observed load effect values. A Weibull distribution is fitted to the top $2\sqrt{n}$ (Castillo 1988) maximum daily load effects and extrapolated to the 75-year level (OBrien et al. 2010) – see Figure 2. The 75-year load effects for each site are then normalized by dividing them by the corresponding HL-93 load effects. As is the nature of Extreme Value Statistics, the characteristic value is found by extrapolation beyond what is measured so it may correspond to an illegally overloaded Strength I vehicles.

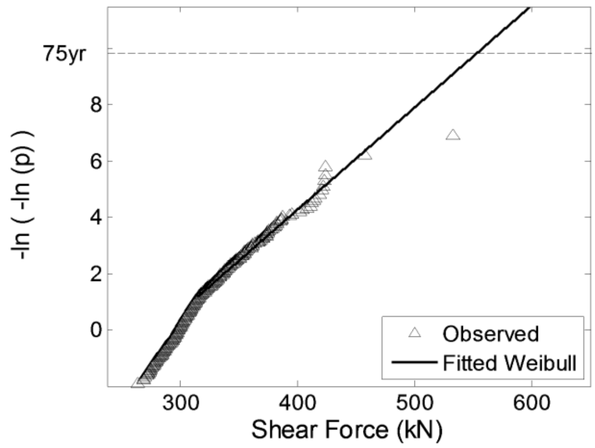


Figure 2. Probability paper plot of maximum daily shear force effects for a 30 m simply supported bridge for the Maryland WIM site (p = probability of non-exceedance).

2.3 Two-Lane Same-Direction Load Modeling

Critical loading on two-lane same-direction bridges results from two types of loading events – single-lane and side-by-side events. At a given site certain events types will govern for different load effects and bridge lengths. Typically side-by-side events are more important for girders/sections located where the lanes meet while single-lane events have more influence on edge girders/sections. However, this can vary depending on the transverse stiffness – girder bridges are much less stiff than solid slabs, for example. Single-lane events and side-by-side events are treated separately when extrapolating from the observed two-lane load effects to the characteristic values. A Weibull distribution is fitted (as described in Section 2.2) to the single-lane events. Side-by-side truck loading events however require a more complex process than the single-lane case as the correlations of truck weights and inter-truck gaps which occur in these events (OBrien & Enright 2011) must be accounted for. 'Scenario Modeling' is used here to simulate a stream of two-lane same-direction traffic. Scenario Modeling (OBrien & Enright 2011; OBrien, Leahy, et al. 2013) is a method which simulates new traffic by perturbing the measured traffic using a smoothed bootstrap approach. The measured traffic is divided into scenarios and kernel density estimators are used to vary the important characteristics of each scenario, such as truck weight and inter-truck gaps. To reduce the variability associated with estimating the 75-year load effect, 300 years of traffic is simulated using Scenario Modeling. The characteristic value is then calculated by fitting a Weibull distribution to the data, effectively interpolating between the simulated load effects. This also helps to smooth the variability in the tail of the simulation. The extrapolation process is shown in Figure 3. The calculation of the characteristic load effects must account for the three event types – single-truck events in either lane and side-by-side events. A given load effect value can be exceeded by each of the three events, and each event has a different probability. For single-truck events, these probabilities are calculated from the measured data, while for side-by-side events probabilities are calculated from the simulated data. The true probability of exceedance is a combination of all three probabilities and is calculated using a Composite Distribution Statistics (CDS) approach (Caprani & OBrien 2010; Caprani et al. 2008). The approach is derived from the theorem of total probability and is described in Equation (1).

$$F_C(y) = \prod_{j=1}^{N} F_j(y) \tag{1}$$

where $F_C(y)$ is the composite distribution for maximum daily load effect, for a mixture of N event types, and $F_j(y)$ is the cumulative distribution for event type j.

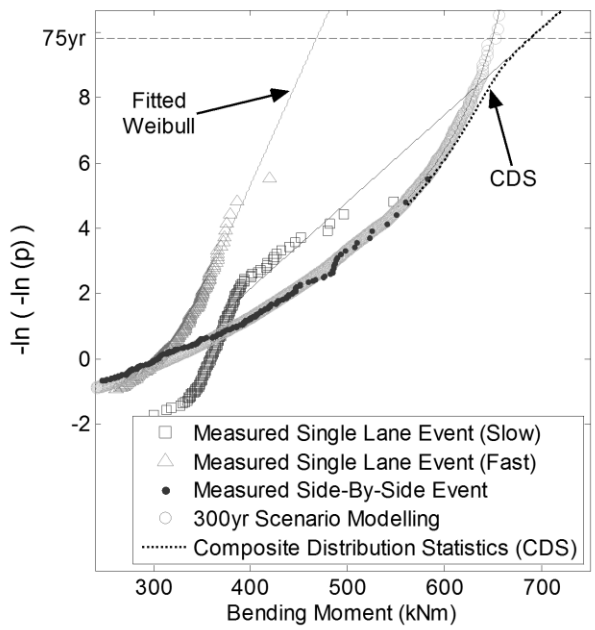
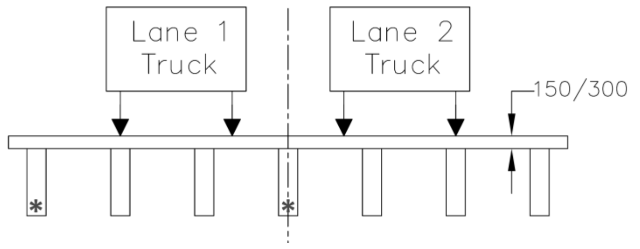


Figure 3. Mid-span bending moment in the girder where lanes meet in a 30 m girder bridge at the Indiana site.

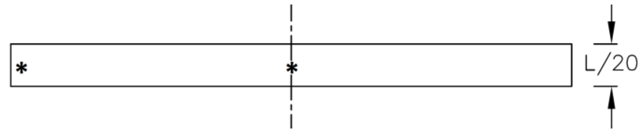
2.3.2 Finite Element Model

For loading on a two-lane same-direction bridge, the HL-93 model assumes that the critical event involves two trucks of the same weight, side-by-side on the bridge. This is based on the assumption that every 15th truck is side-by-side on the bridge. Of these side-by-side events, it is further assumed that the two truck weights are partially correlated for every 10th event and fully correlated for every 30th event (Kulicki et al. 2007). Based on these assumptions it was calculated that the 75-year event comprises of two two-month trucks side-by-side on the bridge (Nowak 1999).

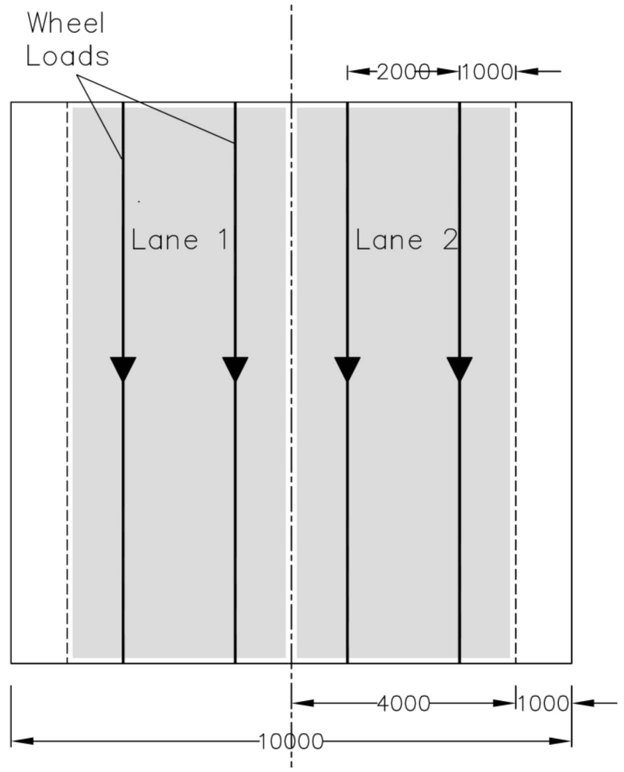
The HL-93 model cannot be accurately assessed without accounting for the transverse distribution of load on the bridge. A finite element model is used to generate influence lines for the two-lane case. Mid-span bending and shear at the exit, for both solid slab bridges and girder bridges are examined. A range of bridge lengths is considered and the load effects are calculated at the center (between lanes) and at the slow lane edge – see Figure 4. Both locations are examined as the edge is mainly influenced by loading in the adjacent slow lane whereas side-by-side loading events are more important when examining the effects along the center line of the bridge. The bridge properties are given in Table 3 and the geometries are shown in Figure 4. The two bridge types are examined as they are typically used for different ranges of spans. For the slab bridge, three different lengths are examined. A span to depth ratio of 20:1 is used for all slabs. The girder bridge is considered for 20 and 30 m spans. In each case slab depths of 0.15 and 0.30 m are examined to account for different levels of relative transverse stiffness. A lane width of 4 m is used to fit with the 0.5 m element size used. All wheel loads are assumed to be 2 m apart, transversely, with the truck situated in the center of the lane. Uniformly distributed loads are applied across the full 4 m lane width leaving a 1 m unloaded verge between the lane and the edge of the structure. The finite element model assumes the concrete to be uncracked and the same Young's modulus is used in both directions.



(a) Section through girder bridge



(b) Section through solid slab



(c) Plan view (dimensions in mm)

Figure 4. Bridges used in the two-lane same-direction analysis (* = locations where load effects are examined).

3. Results

3.1 Single-Lane Case

The characteristic 75-year load effects, as calculated for each site, are normalized by dividing them by the corresponding HL-93 load effects (including the single-lane multiple presence adjustment factor of 1.2) and these normalized values are plotted in Figure 5. The sites are ordered according to the 'severity' of loading, i.e., by how much, on average, the load effects exceed the corresponding HL-93 value. A good load model should give similar levels of conservatism across all load effects and bridge lengths. This will manifest itself in Figure 5 as similar values for the ratios of characteristic load effect to the corresponding HL-93 value. The wide spread in the points in the figure is evidence of inconsistent levels of conservatism in the traffic component for bridges designed to HL-93.

Table 3. Bridges examined for two-lane same-direction loading.

Bridge Type	Bridge Length (m)	Total Bridge Depth (m)	Second Moment of Area[1] (m^4)
Slab	10	0.5	0.104
	15	0.75	0.352
	20	1.0	0.833
Girder	20	1.05	0.565
	20	1.20	0.566
	30	1.55	1.999
	30	1.70	2.000

[1] Second moment of area for the entire bridge cross section

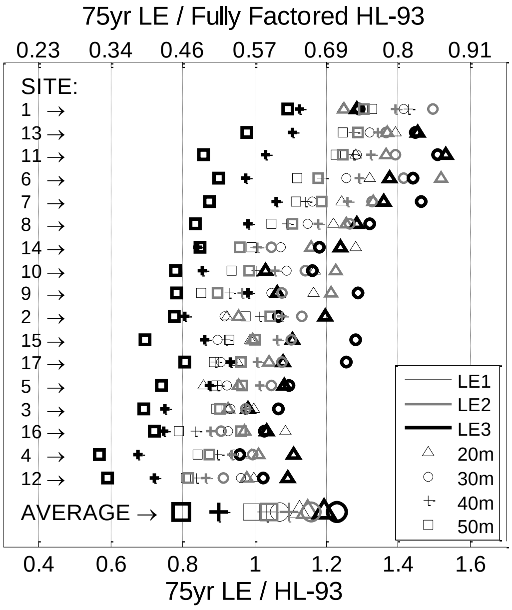


Figure 5. Single-lane characteristic load effects, normalized with respect to HL-93 (mean of site standard deviations is 0.139).

On average the normalized load effects are greater than one, i.e., the true load effects are greater than the corresponding HL-93 values. However, the difference is small with the average for all load effects at all sites just 6.2% greater than the HL-93. A more significant issue is the spread of the values at each site. For example, 50 m two-span continuous bridges are being over-designed for LE3 at most sites while most of the corresponding 30 m bridges are being under-designed – see Figure 5. For the over-designed load effects, this results in excess capacity and a waste of material, whereas for the under-designed load effects, the bridges are below the target safety level. In general, the HL-93 model is better at representing

LE1 and LE2 than LE3.The standard deviations of the ratios of 75-year load effect to HL-93, are calculated for each site. The mean of these standard deviations is 0.139 which, it should be noted, is within the ±20% range expected during the AASHTO LRFD calibration. For a typical site, mean ± 2 standard deviations (95% interval) gives a range of ratios from 0.78 to 1.34, i.e., from 22% over-design to 34% under-design.

There is a general trend in Figure 5 of HL-93 being more conservative for longer spans than for shorter ones (the squares tend to be on the left and the circles and triangles tend to be on the right). As spans increase, dead load becomes a greater portion of the total total, with traffic becoming proportionally less. Hence, while the traffic load model may be more conservative, its effect on total load is proportionally less which diminishes the effect.
In bridge design the HL-93 loading is multiplied by a load factor to determine the strength limit state (AASHTO 2010). This load factor aims to achieve the target reliability of 3.5 (Kulicki et al. 2007; Moses 2001). It accounts for a live load coefficient of variation of 20%, which includes load distribution uncertainties, dynamic allowance uncertainties, site to site variability and uncertainties in estimating the 75-year load effect (Sivakumar et al. 2008). A live load factor of 1.75 is used for the Strength I loading. A secondary axis at the top of Figure 5 shows the ratios for the fully factored HL-93 and all load effects fall below the strength limit state.

3.1.1 Proposed Load Model

Fixed and variable uniformly distributed loads (UDL), knife edge loads and tandem loads are used in past and current bridge design codes (BS5400, 2006; EC1, 2003; Highways Agency, 2001). Some of these alternatives were assessed to find their appropriateness as a load model here. A knife edge load is found to perform well for short bridges but not for bridges with longer influence lines. A fixed UDL performs well for longer bridges but gives poor results on shorter bridges where the load effects are governed by a large load concentrated over a short distance. A variable UDL gives the best compromise as it can apply a large concentration of load on the shorter bridges and less on longer bridges, where the critical loads are generally spread over a longer truck length. The disadvantage of a UDL type model is that individual large wheel loads, which may cause critical local effects, are not modeled and additional local checks may be required.

For each bridge length there is an optimum UDL which best fits the data from all sites. These calculated values are then plotted against bridge length, and a quadratic equation is used to fit a variable UDL to these values – see Figure 6. The normalized load effects for this model are shown in Figure 7. There is a significant improvement in the variation of load effects across all sites and this improvement is particularly evident in the average site values. For the HL-93 model the average values range from about 0.8 to 1.2; in the proposed model these values are clustered much closer to the ideal value of unity. There is also a good improvement in the consistency at each site, with the mean of the site standard deviations reducing from 0.139 to 0.064. In particular, there is a large improvement in the variation of LE3 with bridge length.
It is observed in Figure 7 that some sites are more heavily loaded than others. This can be allowed for using adjustment factors which are applied to the load model for specific states or

sites. The Eurocode (EC1 2003) uses 'alpha' factors which are specified at a national level to reflect the difference in loading between countries. Such factors are especially important for bridge assessment where over-conservatism can be very costly.

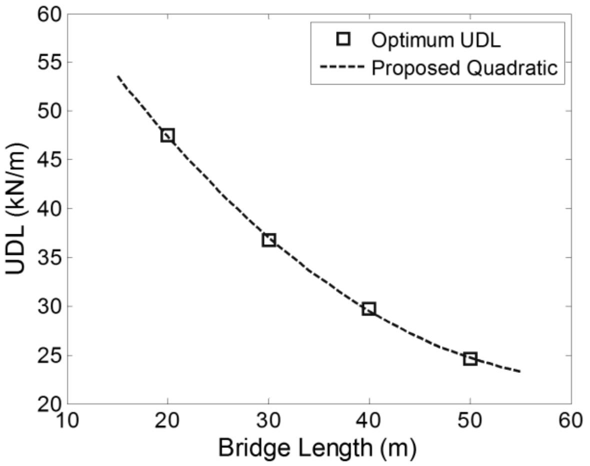


Figure 6. Proposed variable UDL single-lane load model (UDL= $0.014L^2 - 1.73L + 76.4$ kN/m, L = bridge length).

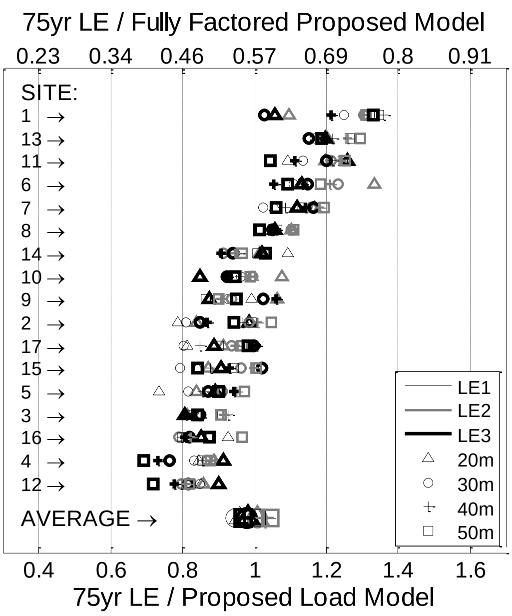


Figure 7. Single-lane characteristic load effects, normalized with respect to proposed variable UDL load model (mean of site standard deviations is 0.064).

3.2 Two-Lane Same-Direction Case

The results for the assessment of HL-93 for the two-lane same-direction case are shown in Figure 8. It is acknowledged that this is based on data from just four WIM sites and that the findings should therefore be treated with caution. The results for the two bridge types are separated with the edge load effects in gray. It can be seen in the results that the HL-93 has a higher level of conservatism for central load effects than for edge effects, with a mean value of 0.90 for center effects and 1.02 for edge effects. This seems reasonable as the model represents a loading event with the same load in both lanes, whereas the long run simulation results show that the critical side-by-side events typically involve a heavy slow lane truck being overtaken by a lighter fast lane truck. The different levels of conservatism for edge and center effects, and the simulation results, suggest that a load model should have a larger load in the slow lane than the fast lane. It is also seen that the model is less conservative for the shorter slab bridges. This suggests that a greater intensity of loading is required for the shorter bridges, as is provided by a variable UDL such as that proposed for the single-lane case.

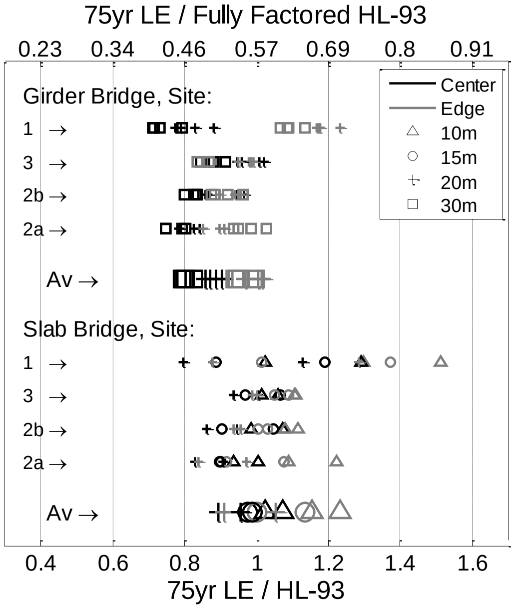


Figure 8. Two-lane same-direction characteristic load effects, normalized with respect to HL-93.

3.2.1 Proposed Load Model

Bridge design load models have different approaches for the distribution of loading for the two-lane case. With the current HL-93 model, the UDL is reduced to 0.83 times that used for the single-lane case and is applied equally in both lanes (AASHTO 2010). The Eurocode (EC1 2003) acknowledges the more dominant loading in the slow lane by applying the same UDL in this lane as is used for a single-lane bridge and applying a fast lane UDL which is 0.28 times that of the slow lane. Considering the results in Figure 8, suggesting that the modeling of load effects at the center of the bridge tends to be more conservative than the edge, a smaller UDL is applied to the fast lane here.

As the UDL proposed for the single-lane case is calibrated across many more sites than are available for the two-lane case, a version of it is kept for the slow lane. 80% of the single-lane bridge UDL is put in the slow lane. Different values were considered and 80% was found to give the best results for the load effects examined. Using a similar methodology as is used with the single-lane bridge, an optimum fast-lane UDL can then be calculated across the selected sites for each bridge length – see Figure 9. A quadratic curve is then fitted to these points. This gives good results for the bridge lengths examined and a fixed UDL of 10kN/m would seem appropriate for all bridge lengths above 30 m.

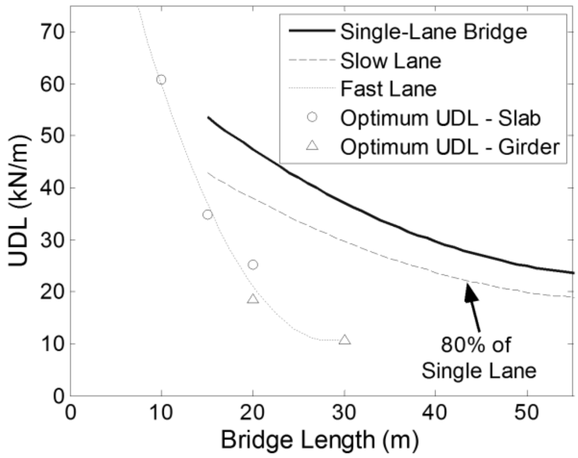


Figure 9. Proposed two-lane load model alongside the proposed single-lane model (Fast Lane UDL= $0.143L^2 - 8.17L + 127.4$ kN/m, L = bridge length).

The normalized load effects for the proposed model are shown in Figure 10. The proposed model is a compromise between finding a model which is appropriate for all bridge lengths, and one which correctly represents load effects at both the center and edge of the bridge. The difference in the level of conservatism between edge and center effects is much improved with the proposed model. A mean value of 1.03 for center effects and 1.00 for edge effects is achieved, compared with 0.90 and 1.02 for HL-93. The improvement is particularly noticeable at site 2a on the girder bridge where the distinct difference in conservatism which is identified with the HL-93 is now removed. The increase in conservatism with bridge length which was present with the HL-93 is much less pronounced in the proposed model.

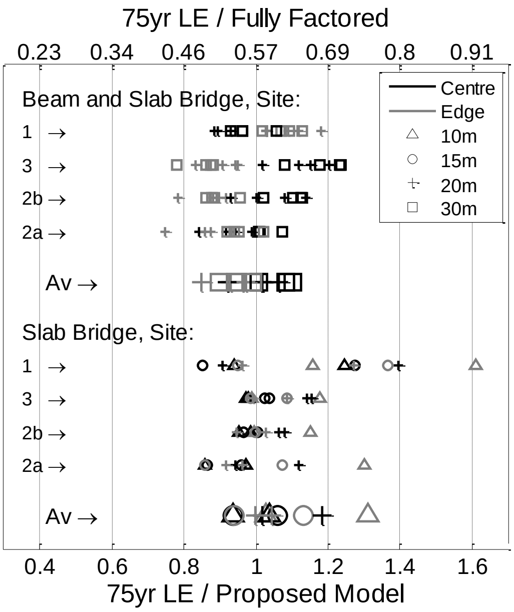


Figure 10. Two-lane same-direction characteristic load effects, normalized with respect to proposed variable UDL load model.

4. Conclusion

Analysis of a substantial US WIM database is used to assess the current US bridge design load model for both single-lane and two-lane bridges. For a given site the current model is found to give large variations in conservatism depending on the load effect and bridge length examined. This may be resulting in inefficient design and assessment of bridges.

For single-lane bridges, data from 17 WIM sites across the US is used. It is found that for a given site there is large variation in the level of conservatism from traffic induced load effect to load effect with the results for hogging moment over the central support of a two-span continuous bridge showing the most variation. To reduce the variability a UDL model which varies with bridge length is proposed. The model reduces the mean of the standard deviations for each site’s load effects from 0.139 to 0.064. This results in a reduction in the over-design for some load effects and the under-design of others and gives a more consistent level of safety for the traffic components of all load effects at a given site.

Less data is available for two-lane same-direction traffic, with 4 site directions examined. It is found that the HL-93 model, which assumes that the critical event occurs when two trucks of identical weight are side-by-side on the bridge, is overly conservative for load effects along the center line of the bridge. This is due to the fact that in practice, side-by-side events typically involve a lighter truck in the fast lane overtaking a heavier truck in the slow lane. It is also found that with the two-lane case, the level of conservatism with the HL-93 increases with bridge length. To address these issues a model which puts a different UDL in each lane is proposed. The slow lane has a UDL which is 80% of that proposed for the single-lane case and a second UDL, which again varies with bridge length, is used for the fast lane. This fast-lane UDL reduces more quickly with bridge length than the slow lane and provides the differential loading required to provide a more consistent load model. It must be emphasized that this 2-lane model is based on data from just four WIM sites.

The assessment of the HL-93 presented here shows that improvements on the current model can be achieved, especially in terms of the consistency from load effect to load effect. The model proposed for the single-lane case is based on the analysis of a large database and the improvements in consistency are clear. While the analysis of the two-lane case does identify some weaknesses with HL-93, the improvements achieved by the proposed model are less conclusive than the single-lane case and similar analysis with a larger database may be needed to provide more confidence in the results.

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6. References

AASHTO, 2010. *AASHTO LRFD Bridge Design Specifications* 5th ed., Washington D.C.: American Association of State Highway and Transportation Officials.

BS5400, 2006. *Part 2: Steel, concrete and composite bridges. Specification for loads*, London: British Standards Institution.

Caprani, C.C. & OBrien, E.J., 2010. "The use of predictive likelihood to estimate the distribution of extreme bridge traffic load effect." *Structural Safety*, 32(2), 138–144.

Caprani, C.C., OBrien, E.J. & McLachlan, G., 2008. "Characteristic traffic load effects from a mixture of loading events on short to medium span bridges." *Structural Safety*, 30(5), 394–404.

Castillo, E., 1988. *Extreme value theory in engineering*, London: Academic Press.

COST323, 2002. *Weigh-in-Motion of Road Vehicles - Final Report* B. Jacob, E. OBrien, & S. Jehaes, eds., PARIS: LCPC.

Crespo-Minguillón, C. & Casas, J.R., 1997. "A comprehensive traffic load model for bridge safety checking." *Structural Safety*, 19(4), 339–359.

EC1, 2003. *Eurocode 1: Actions on structures, Part 2: Traffic loads on bridges. European Standard EN 1991-2:2003,* Brussels: European Committee for Standardization.

Enright, B. & OBrien, E.J., 2011. *Cleaning Weigh-in-Motion Data: Techniques and Recommendations*, University College Dublin & Dublin Institute of Technology, Retrieved from http://iswim.free.fr/doc/wim_data_cleaning_ie.pdf

Enright, B. & OBrien, E.J., 2013. "Monte Carlo simulation of extreme traffic loading on short and medium span bridges." *Structure and Infrastructure Engineering*, 9(12), 1267–1282.

FHWA, 1995. *Traffic Monitoring Guide* 3rd ed., U.S. Department of Transportation Federal Highway Administration.

Fu, G. & You, J., 2011. "Truck load modeling and bridge code calibration." In Faber, Kohler, & Nishijima, eds. *Applications of Statistics and Probability in Civil Engineering*. London: Taylor & Francis Group, 406–413.

Getachew, A. & OBrien, E.J., 2007. "Simplified site-specific traffic load models for bridge assessment." *Structure and Infrastructure Engineering*, 3(4), 303–311.

HighwaysAgency, 2011. DMRB, Vol. 3, Section 4, Part 19, BD 86/11. In *The Assessment of highway bridges and structures for the effects of special types general order (STGO) and special order (SO) vehicles*. London.

HighwaysAgency, 2001. DMRB, Vol. 3, Section 4, Part 3, BD 21/01. In *The Assessment of Highway Bridges and Structures*. London.

Kulicki, J.M. et al., 2007. *Updating The Calibration Report For AASHTO LRFD Code - NCHRP 20-7/186*, Washington D.C.: Transport Research Board.

Leahy, C., 2014. *Predicting Extreme Traffic Loading on Bridges Using Weigh-In-Motion Data*. University College Dublin.

Miao, T.J. & Chan, T.H.T., 2002. "Bridge live load models from WIM data." *Engineering Structures*, 24(8), 1071–1084.

Moses, F., 2001. *Calibration of Load Factors for LRFR Bridge Evaluation - NCHRP Report 454*, Washington D.C.: Transport Research Board.

Nowak, A., 1999. *Calibration of LRFD Bridge Design Code - NCHRP Report 368*, Washington D.C.: Transport Research Board.

Nowak, A., 1993. "Live load model for highway bridges." *Structural Safety*, 13(1-2), 53–66.

Nowak, A. & Rakoczy, P., 2012. "WIM Based Simulation Model Of Site Specific Live Load Effect On The Bridges." In B. Jacob et al., eds. *6th International Conference on Weigh-In-Motion*. Dallas, 352–358.

Nowak, A. & Szerszen, M.M., 1998. "Bridge load and resistance models." *Engineering structures*, 20(11), 985–990.

O'Connor, A. et al., 2001. "Report of Current Studies Performed on Normal Load Model of EC1: Part 2. Traffic Loads on Bridges. » *Revue française de génie civil*, 5(4), 411–433.

O'Connor, A. & Enevoldsen, I., 2009. "Probability-based assessment of highway bridges according to the new Danish guideline." *Structure and Infrastructure Engineering*, 5(2), 157–168.

OBrien, E.J., Leahy, C., et al., 2013. "Validation of Scenario Modelling for Bridge Loading." *Submitted for publication*.

OBrien, E.J. & Enright, B., 2011. "Modeling same-direction two-lane traffic for bridge loading." *Structural safety*, 33(4), 296–304.

OBrien, E.J., Enright, B. & Getachew, A., 2010. "Importance of the tail in truck weight modelling for bridge assessment." *ASCE Journal of Bridge Engineering*, 15(2), 210–213.

OBrien, E.J., Enright, B. & Leahy, C., 2013. "The Effect of Truck Permitting Policy on US Bridge Loading." In G. Geodatis, B. R. Ellingwood, & D. M. Frangopol, eds. *11th International Conference on Structural Safety & Reliability*. New York.

Schulman, J.F., 2003. *Heavy Truck Weight and Dimension Limits in Canada*, Railway Association of Canada.

Sivakumar, B., Ghosn, M. & Moses, F., 2008. *Protocols for Collecting and Using Traffic Data in Bridge Design - NCHRP Report 683*, Washington D.C.: Transport Research Board.

USDOT, 2000. *Comprehensive Truck Size and Weight Study*, U.S. Department of Transportation. FHWA-PL-00-029.

Walker, D. & Cebon, D., 2012. "The Metamorphosis of LTPP Traffic Data." In B. Jacob et al., eds. *6th International Conference on Weigh-In-Motion*. Dallas, 242 – 249.

Walker, D., Selezneva, O. & Wolf, D.J., 2012. "Findings From LTPP SPS WIM Systems Validation Study." In B. Jacob et al., eds. *6th International Conference on Weigh-In-Motion*. Dallas, 214 – 221.

Zhao, J. & Tabatabai, H., 2012. "Evaluation of a Permit Vehicle Model Using Weigh-in-Motion Truck Records." *Journal of Bridge Engineering*, 17, 389–392.