ABSTRACT: This paper reports on some latest developments in efforts to balance truck loads and the capacity of highway bridges that carry the loads. One of them is the completion of the development of a method for estimating effects of truck weight limit change on bridge network costs, funded by the US National Cooperative Highway Research Program (NCHRP). Four categories of cost impact are addressed in this new method: steel fatigue, reinforced concrete (RC) deck fatigue, additional inadequate exiting bridges, and higher design load for new bridges. This development has taken into account the constraints on data availability at the State infrastructure system level. Another recent development is the completion of a research effort examining the adequacy of bridge design load for the State of Michigan in the US, with respect to real truck loads measured recently. It was found that there is a need to develop a more rational design load to cover the risk represented. These developments offer effective tools for response to the trend of increasing truck loads.

1. INTRODUCTION

Heavy trucks represent major loads to highway bridges. Accordingly, highway bridges should be designed and maintained such that they are able to sustain these loads all the time. Along with economic development, truck loads change their patterns over time, including their magnitudes. Bridge engineers have been striving to manage a rational balance between the truck loads and the bridge capacity. Note that there is an amount of uncertainty associated with both the load and the capacity, which is important to be acknowledged and to be covered in making related decisions. This paper addresses a number of relevant issues in these efforts.

One of the subjects almost always in the center of bridge engineering is the load capacity requirement for bridge design. It is important because this requirement has significant implications to the normal operation of the bridges over their life spans. The load capacity requirement is dictated by the design truck load and its associated load factors, depending on the failure modes considered. They largely determine the load carrying capacity of the bridges at the time the construction is completed. Furthermore, they also dominantly influence the bridge capacity in future years when the truck load becomes more severe and/or the bridge suffers from deterioration. Therefore, it is important to periodically review the bridge design load to assure that it, along with its load factors, is
indeed able to cover the changes in truck loads that have taken place and that are expected to take place. A study has been recently conducted to investigate this very issue for the State of Michigan, jointly by researchers at Michigan Technological University and Wayne State University (Van de Lindt et al 2003) for the Michigan Department of Transportation (MDOT). Its process and major results are summarized below.

Trucks deliver a significant portion of the product for many nations in the world. In the US in 1974, for example, this includes 60 percent of all inter-city shipments of manufactured products, 80 percent of all fruits and vegetables, and 100 percent of all livestock (RJHansen 1979). While we benefit from truck transportation, highway agencies have spent a significant amount of resources to establish and maintain the highway system. Quantifying causes of these expenditures has been a focus of several studies in a number of countries (Moses 1989, Fu et al 2002).

To that end, this paper also presents a method for estimating the costs of truck weight limit changes for a network of bridges. (Truck weight here collectively refers to the truck’s gross weight, axle weights, and spacings of axles.) This method is a major product of a research project funded by the US National Cooperative Highway Research Program (NCHRP). This subject is important because trucking at higher gross vehicle weight (GVW) is more productive but is envisioned to be more costly to the infrastructure. Thus, transportation agencies receive constant pressure to increase truck weight limits. The new method is to help transportation agencies deal with such pressure quantitatively and rationally at the network level.

2. BRIDGE DESIGN LOAD VS. TRUCK OPERATING LOAD

The HS20 truck load specified in current AASHTO design code (1996) has been used as the highway bridge design load for several decades in the US. On the other hand, a significant number of states have started to change this design load in their respective jurisdiction to a higher load. For example, in 1972 MDOT adopted HS25 as its standard design load for bridge on the interstate and arterial highways. Note that Michigan has the highest legal truck weights in the US. In several studies on the behavior of truck loads (Snyder et al. 1985, Moses 2001), it has been established that truck loads have been increasing in both magnitude and volume, as a result of economic development. With a concern about the load increase contrasted by relatively constant design load, MDOT funded a study to investigate the adequacy of its design truck load for highway bridges, with respect to their capability to cover real truck loads.

A sample of 20 bridges was randomly selected from the population of new bridges constructed in the past 10 years in Michigan. These bridges were used as specimens to understand the effect of design regarding the provided capacity. Most recently recorded truck weights and configurations were used as typical loads to these bridges. The data for the capacity and the load were then used in an assessment of the structural reliability for the primary bridge component (beams) and a secondary component (reinforced concrete deck). This assessment focused on the strength failure mode only. Other failure modes,
such as the fatigue failure mode, were out of the scope. A target reliability index $\beta=3.5$ was used to determine whether the design load is adequate or not.

2.1 Bridge and Truck Load Samples

In a survey over the population of the new bridges built in the past 10 years in Michigan, it was found that the new bridges are mainly of beam-deck type. They consist of the following 4 types according to the cross section arrangement: 1) steel beams (40.0%), 2) prestressed concrete I beams (30.6%), 3) prestressed box beams adjacent to each other (14.6%), and 4) spread prestressed concrete box beams (5.6%). Accordingly, it was scoped for this study to investigate the structural reliability of only these bridge types. For each type, 5 bridges were randomly selected to form a 20-bridge sample as the specimens. Besides the load carrying capacity, these bridges also provided general information for typical highway bridge construction in Michigan, such as span type (simple vs. continuous spans), spacings between beams, typical beam cross sections, etc. These parameters defined a manageable and realistic scope for analysis.

In the early 1990s, Nowak (1994) and his students at University of Michigan collected truck axle weight and configuration data from several bridges in the metropolitan Detroit area, which is the most industry-intensive region in Michigan. They applied a weigh-in-motion (WIM) technique in gathering this data set, using the bridge as a scale for weighing trucks. We found that this data set was the latest available of the kind. Note that there are also other WIM data available, which used a different kind of WIM technique that is considered to be less accurate. A total of about 39,000 trucks were included in this data set, forming 8 bridge sites. These bridges were located on 4 different types of highways referred to as Functional Classes 01, 11, 12, and 14. They are respectively: Principal Arterial – Interstate Rural (01), Principal Arterial – Interstate Urban (11), Principal Arterial – Urban (12), and Other Principal Arterial - Urban (14).

Practically, these sample bridges could be possibly constructed on any of the 4 different functional classes of highway. It is because the current design approach does not differentiate truck load patterns in design for the strength failure mode. Accordingly, the following approach was taken in this research project. For each bridge span of the 20 sample bridges, an influence line was developed for each possibly critical load effect (either moment or shear). Then a measured vehicle from the WIM data set was “driven” through the influence line to find the maximum load effect for that span. All the recorded vehicles in the data set for a functional class, after driven through the influence line, produced statistics about the particular load effect at the particular location in the bridge. Considering the time interval in which all the used vehicles were recorded, this result was then projected to 75 years (as the intended life span of the new bridges in Michigan) for a probability distribution of the load effect. This established a probabilistic description of the load effect at a particular location of the bridge for its intended life span of 75 years, which is defined as a random variable. This description is then used below for a reliability analysis for the bridge component of interest. Note that this analysis was repeated for all the possibly critical load effects in the bridge spans. In addition, this covered all the bridges in the sample selected.
2.2 Bridge Structural Reliability and Safety Requirement

For a load effect at a cross section of a bridge component, the failure of the cross section is defined as the following safety margin \( Z \) becoming negative:

\[
Z = R - S
\]  

(1)

where \( R \) is the resistance of the component for that load effect and \( S \) is the load effect. Both \( R \) and \( S \) refer to the same cross section of the component, and are modeled as random variables. The probabilistic description of the total load effect for a time period of 75 years has been discussed above. It is then modeled as follows for the load effect on a single component:

\[
S = S_t D I + DL
\]

(2)

where \( S_t \) is the total load effect on the bridge section discussed above, \( D \) is a factor to distribute the total load effect to a single component, and \( I \) is the impact factor to cover the dynamic effect of the moving load. \( DL \) is the dead load effect on the component. These four random variables are modeled as lognormal variables. The statistical parameters (the mean and the standard deviation) for \( D, I, \) and \( DL \) are taken from (Nowak 1999). Those for \( S_t \) are from the previously discussed procedure using the latest WIM data available for truck weights and configurations in the Detroit area.

The probabilistic description for \( R \) was also established but using more generally accepted statistical parameters in (Nowak 1999). These parameters are based on a concept that the nominal values of the resistance used in the design (e.g., the steel cross section’s strength) are correlated with their mean values. Their statistical parameters were used in calibrating the AASHTO LRFD Bridge Design Specifications. Using these probabilistic descriptions of \( R \) and \( S \) in Eq.1, the result of the reliability analysis is the reliability index \( \beta \), which is defined as

\[
\beta = \Phi^{-1}[1 - \text{Probability}(Z < 0)]
\]

(2)

Note that \( \Phi^{-1} \) is inversely monotonic with the failure probability or \( \text{Probability}(Z \leq 0) \). Namely, a smaller failure probability leads to a larger \( \beta \).

In the calibration of the AASHTO LRFD Bridge Design Specifications (1998), a target \( \beta \) value of 3.5 was used (Nowak 1999). This target represents an average reliability index level implied in the previous generation of the AASHTO bridge design code (1996) in the US. It also represents a generally accepted safety level for bridge components. Therefore, this same target level was used in this study to judge whether or not the current design load for Michigan (HS25) is adequate.

2.3 Results, Discussions, and Conclusions
Table 1 summarizes the reliability index values for all the possibly critical cross sections of the selected bridge spans. The table consists of two parts: the first one using the as-designed strength that usually is higher (sometimes much higher) than the minimum strength required by the design specifications (AASHTO 1996), and the second one using the minimum strength required by the design code. For each part, all 4 types of bridges are included: S for steel beam bridges, PI for prestressed concrete I beam bridges, PCS for prestressed concrete spread box beam bridges, and PCA for prestressed concrete adjacent box beam bridges. For each bridge type, two load effects were considered: shear and moment. For each load effect, four kinds of highway functional classes (01, 11, 12, and 14) were included here, each representing a class of highway, as discussed earlier. The available WIM data were collected from sites of these four functional classes only. Furthermore, for each functional class, an individual bridge site may have a volume of truck traffic that is different from other bridges also belonging to the same functional class. Thus, two representative truck volumes are used here to provide a general understanding for the influence of the truck volume: 1) the average truck volume, i.e., the 50th percentile value, and 2) the 90th percentile value of the truck volumes for the specific functional class. The 50th percentile value means that 50 percent of the bridge sites in the same functional class have a truck volume lower than this value. Consistently, the 90th percentile means that 90 percent of the bridge sites in the same functional class have a truck volume lower than this value.

Table 1 Average Reliability Index $\beta$ for Beam Capacity of New Bridges in Michigan

<table>
<thead>
<tr>
<th></th>
<th>Functional Class</th>
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<tbody>
<tr>
<td></td>
<td></td>
<td>01</td>
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<td>12</td>
<td>14</td>
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<td></td>
<td></td>
</tr>
<tr>
<td>S</td>
<td>Shear</td>
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<td>5.1</td>
<td>5.0</td>
</tr>
<tr>
<td></td>
<td>Moment</td>
<td>5.5</td>
<td>5.5</td>
<td>3.9</td>
<td>3.9</td>
</tr>
<tr>
<td>PI</td>
<td>Shear</td>
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<td>8.1</td>
<td>6.6</td>
<td>6.5</td>
</tr>
<tr>
<td></td>
<td>Moment</td>
<td>5.8</td>
<td>5.8</td>
<td>3.6</td>
<td>3.5</td>
</tr>
<tr>
<td>PCS</td>
<td>Shear</td>
<td>8.2</td>
<td>8.2</td>
<td>6.3</td>
<td>6.2</td>
</tr>
<tr>
<td></td>
<td>Moment</td>
<td>8.3</td>
<td>8.3</td>
<td>5.4</td>
<td>5.4</td>
</tr>
<tr>
<td>PCA</td>
<td>Shear</td>
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<td>4.9</td>
<td>2.9</td>
<td>2.9</td>
</tr>
<tr>
<td></td>
<td>Moment</td>
<td>6.1</td>
<td>6.1</td>
<td>3.8</td>
<td>3.8</td>
</tr>
<tr>
<td><strong>Design-minimum</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Bridge Type</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>SC</td>
<td>Shear</td>
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<td>4.3</td>
<td>2.3</td>
<td>2.3</td>
</tr>
<tr>
<td></td>
<td>Moment</td>
<td>4.1</td>
<td>4.1</td>
<td>2.4</td>
<td>2.4</td>
</tr>
<tr>
<td>PI</td>
<td>Shear</td>
<td>3.5</td>
<td>3.5</td>
<td>2.0</td>
<td>2.0</td>
</tr>
<tr>
<td></td>
<td>Moment</td>
<td>4.3</td>
<td>4.3</td>
<td>2.1</td>
<td>2.0</td>
</tr>
<tr>
<td>PCS</td>
<td>Shear</td>
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<td>3.8</td>
<td>2.0</td>
<td>1.9</td>
</tr>
<tr>
<td></td>
<td>Moment</td>
<td>5.0</td>
<td>5.0</td>
<td>2.2</td>
<td>2.1</td>
</tr>
<tr>
<td>PCA</td>
<td>Shear</td>
<td>3.1</td>
<td>3.1</td>
<td>1.2</td>
<td>1.2</td>
</tr>
<tr>
<td></td>
<td>Moment</td>
<td>5.6</td>
<td>5.6</td>
<td>3.2</td>
<td>3.1</td>
</tr>
</tbody>
</table>
In Table 1, those $\beta$ values lower than the target level of 3.5 are shaded. Note that for the same bridge, the strength of the bridge component’s cross section is the same for all different functional classes of highways, because the current design procedure is not different for different functional classes when the strength failure mode is concerned. Thus, different $\beta$ values for the same bridge in Table 1 but for different functional classes are due to the differences in truck loads – for example, with respect to the distribution of the types of vehicles in the truck population, the trucks’ weights and configurations, truck traffic volumes, etc. It is seen that Functional Classes 11 and 12 represent the most severe truck loads in those studied here, resulting in more cases of lower $\beta$ than 3.5. In addition, the as-designed strengths obviously provide higher structural reliability than those design-minimum strengths, sometime much higher.

It is important to note that the as-designed strengths also have led to $\beta$ values lower than 3.5 for a few cases. These cases are particularly of interest because they show that unsystematically practiced conservatism that adds strength does not necessarily always provide adequate safety margins. These lower $\beta$ values for the as-designed strength essentially were due to the very low $\beta$ values for the design-minimum strengths. For example, the PCA bridges (prestressed concrete adjacent box beam bridges) have as low as 1.2 for $\beta$ for the case of shear capacity at sites belonging to Functional Class 11. This has led to a conclusion that current highway bridge design load (for beams) at least for the metropolitan Detroit area needs to be updated to accommodate the loads now observed. On the other hand, the design load for RC decks was found to be adequate (Van de Lindt et al 2003).

3. A METHOD TO ESTIMATE EFFECT OF TRUCK WEIGHT LIMIT CHANGE ON BRIDGES

This new method was developed for transportation agencies to estimate the impact costs for a network of bridges. It starts with prediction for the truck load spectra (truck weight histograms). The resulting load spectra are then used for individual bridges for impact cost estimation, respectively for four different categories of cost impact: additional fatigue accumulation in steel members, additional fatigue accumulation in RC decks, additional deficient bridges according to the load rating, and additional cost for higher design load.

3.1 Predicting Truck Load Spectra

A direct impact of truck weight limit change is the change in truck load spectra to bridges. It includes changes in truck weight histograms (TWHs) and wheel weight histograms (WWHs). The former represents the load to the entire bridge, affecting the bridge’s relative strength demand. It also influences steel bridge fatigue accumulation. The latter is the load to bridge decks that transfer wheel loads to the supporting structural frame. A new method is presented below for predicting the TWHs and WWHs under a change in truck weight limits.
Predicting TWHs

Changes in TWHs due to truck weight limit changes may be classified into the following three types of freight shifting. 1) Load shifts without changing truck types (i.e., truck configurations), referred to as truck load shift hereafter. 2) Load shifts with change of truck configuration, referred to as truck type shift below. 3) Exogenous shifts, such as economy growth and mode shift (e.g., from and to rail) due to competition. These shifts are individually dealt with below. Base Case used below refers to the condition before the considered change in truck weight limits. Alternative Scenario represents the condition after the change.

It is assumed that TWHs for the Base Case are available for each type of vehicles, except automobiles and 4-tire light trucks. These two types of vehicles are considered irrelevant to issues related to bridge strength and fatigue. These TWHs for the Base Case may be obtained using weigh-in-motion (WIM) data available with State highway agencies. The vehicle-mile-of-travel (VMT) data issued by the US Federal Highway Administration (FHWA) for Year 2000 may also be used as the default data set (Fu et al 2002). This default data set is available for 12 functional classes and for all states.

Truck load shift

In truckload shifting as a result of truck weight limit increase, trucks of a given type can be loaded heavier. This is because the Alternative Scenario’s practical maximum GVW (PMGVW) is higher than the Base Case PMGVW. This type of change in TWHs is expected to occur when the Alternative Scenario does not require trucks to change their configuration for carrying the new allowable loads.

In other words, only the TWH for that type of vehicle will be subject to change (shifting). This shifting should be performed only for weight-limit dependent truck traffic. For example, trucks heavier than Base Case PMGVW operating under special permits will not need to change in response to the weight limit change. The amount of traffic responding to the change is identified using a window shown in Fig.1 over the Base Case TWH (for the impacted type of trucks). Namely, the traffic that is within this window will be subject to shifting and otherwise not. The window is defined by five parameters: \(a_1\), \(a_2\), \(b_1\), \(b_2\), and \(c\), all in fraction or percentage. These parameters are referred to as window parameters.

Parameters \(b_1\) and \(b_2\) define a neighborhood of weight-limit sensitive traffic, with reference to the Base Case’s PMGVW. When \(\text{GVW}_{\text{BC}} / \text{PMGVW}_{\text{BC}}\) is close to 1 between \(1-a_1\) and \(1+a_2\), the level of weight-limit dependence is described by \(c\). It indicates the percentage of the traffic to be changed under the Alternative Scenario. Beyond this small range to the left, the weight-limit dependence is assumed to vary linearly from \(c\) at \(1-a_1\) to zero at \(1-b_1\) being the lower boundary of the neighborhood. To the right from \(1+a_2\) a similar behavior is assumed of weight-limit dependence up to \(1+b_2\).
After the weight-limit dependent truck traffic $TT'_{GVWk,BC}$ is identified for weight interval $k$, the following equations will be thereto applied in modifying the Base Case TWH, as a response to the considered change in truck weight limits:

$$GVW_{AS} = GVW_{k,BC}(PMGVW_{AS}/PMGVW_{k,BC})$$  \hspace{1cm} (3)  

$$TT_{GVW,AS} = TT'_{GVWk,BC} \times (GVW_{k,BC} - TARE_{BC})/(GVW_{AS} - TARE_{AS})$$  \hspace{1cm} (4)  

where subscripts $BC$ and $AS$ refer to the Base Case and Alternative Scenario, respectively. TARE is the empty weight of truck. $TT_{GVW,AS}$ is the truck traffic at weight $GVW_{AS}$ under the Alternative Scenario.

Eq.3 indicates the change in operating weight or GVW. It occurs only when the operating weight is within the window in Fig.1. Eq.4 enforces the condition that the total payload travel (in kN-km or kip-mile) is conserved during load-shifting since the total amount of freight carried remains constant.

Note that when $PMGVW_{AS}$ is greater than $PMGVW_{BC}$ representing an increase in weight limit, the total amount of truck traffic will decrease since fewer trips will be required to transport the same amount of freight (payload). Note that possible payload changes are covered below addressing external factors, such as economy-growth-dependent payload increase and competition-induced payload shift from or to rail.

In applying Eqs.3 and 4, $GVW_{BC}$ is taken at the midpoint of a weight interval falling in the window defined in Fig.1. Consequently, the value of $GVW_{AS}$ generally will not match the midpoint of any weight interval. It is then appropriate to distribute $TT_{GVW,AS}$ between two neighboring weight intervals to achieve the desired value of $GVW_{AS}$, which are designated as the $i$th and the $i+1$th intervals, respectively. The distribution ratios $p_i$
and \( p_i \) for the \( i \)th and the \( i+1 \)th weight intervals are required to satisfy the following equations:

\[
p_i + p_{i+1} = 1 \quad (5)
\]

\[
p_i \text{GVW}_{i,AS} + p_{i+1} \text{GVW}_{i+1,AS} = \text{GVW}_{AS} \quad (6)
\]

Then the truck traffic equal to \( p_i \text{TT}_{\text{GVW,AS}} \) is to be moved to the \( i \)th GVW interval and \( p_{i+1} \text{TT}_{\text{GVW,AS}} \) to the \( i+1 \)th interval.

The following assumptions have been used for the proposed prediction method. A) For many commodities (e.g., potato chips), the cubic capacity of the truck is the limiting factor. B) Heavier trucks excessively above PMGVW\(_{BC}\) and operating under special permits may not react to weight limit changes if other factors do not change. C) The total payload traveled (in kN-km or kip-miles) remains the same before and after the weight limit change, i.e., Payload x Distance of Travel = Constant. This has been expressed in Eq.4, and the distribution of this traffic over the truck-weight intervals is altered due to shifting. Selecting parameters \( a_1, a_2, b_1, b_2, \) and \( c \) for the proposed method may require measured data and appropriate engineering judgment.

**Truck type shift**

The same equations 3 and 4 for truck load shift can also be used for truck type shifting. However, \( \text{TT'}_{\text{GVW},BC}, \text{PMGVW}_{BC} \) and \( \text{TARE}_{BC} \) now refer to the truck type from which traffic is shifted away, and \( \text{TT}_{\text{GVW},AS}, \text{PMGVW}_{AS}, \) and \( \text{TARE}_{AS} \) refer to the truck type to which traffic is shifted in. This reflects the fact that a new truck type of configuration will be needed in order to take advantage of the new weight limits.

**Exogenous shift**

Exogenous shifts here refer to those changes to TWHs due to external factors, other than those discussed above. The influencing factors may be, for example, economic growth, competitiveness with other transportation modes (e.g., rail), etc. (Cambridge Systematics et al. 1997) provides detailed discussions on transportation modal shifts for freight demand predictions. The guidelines presented there are of help in understanding relevant issues, and estimate the amount of truck traffic shift.

The first step of accounting for these effects is to identify the traffic in the TWHs that is subject to exogenous shift. For the case of overall economic growth as an example, all traffic should be subject to change, unless otherwise objected. This may be readily taken into account by using a growth factor to be applied to all traffic:

\[
\text{ADTT}_{AS} = g \text{ADTT}_{BC} \quad (7)
\]

where \( g \) is the growth factor, which could be estimated based on data at the network level. ADTT stands for annual daily truck traffic.
For the case of transportation modal change due to a truck weight limit change, it would be reasonable to use the same kind of window in Fig.1 for identifying the impacted traffic. In addition, a multiplier \( r \) can be applied to the affected traffic at weight \( GVW \):

\[
TT_{GVW,AS} = r_{GVWk}TT'_{GVWk,BC} x(GVW_{k,BC} - TARE_{BC})/(GVW_{AS} - TARE_{AS})
\]  

(8)

As indicated, \( r_{GVWk} \) can be a function of operating weight \( GVW \) at the \( k \)th interval. This multiplier is higher than 1.0 for traffic increase and less than 1.0 for decrease.

Two examples of testing the suggested prediction method have been included in the study (Fu et al 2002). Measured truck weight data are used for these examples. They include a statewide truck weight limit increase from about 73 kips to 80 kips in the 1980s for State of Arkansas and a permit weight limit change from 105 kips to 129 kips for two routes in State of Idaho. Results show that the prediction method can capture the resulting changes in TWHs, using the following window parameters: \( c=0.95 \), \( a_1=a_2=0.10 \) and \( b_1=b_2=0.20 \). These values may be used as the default values, if no more site specific data are available.

**Predicting WWHs**

For assessing reinforced concrete (RC) deck fatigue, truck wheel weight distributions are needed to estimate the cost effects of changes in truck weight limits. It is suggested here that predicting WWHs be based on GVW, assuming that there is a correlation between the wheel weights and the gross weight. This assumption is particularly valid for trucks loaded to the limits, which are dominant in RC deck fatigue. When a TWH is available, possibly obtained using the method recommended above or directly from truck weight data, the wheel weights can be estimated using the following empirical relations:

\[
\text{Wheel Weight} = E + F \times GVW + X
\]  

(9)

where \( E \) and \( F \) are model coefficients for each axle. They can be obtained using WIM data and a regression analysis. In (Fu et al 2002), examples of \( E \) and \( F \) were obtained using data from the State of California. It is also recommended that agencies use their own WIM data to obtain those coefficients for typical truck types within the jurisdiction. The first part of Eq.9 (i.e., \( E + F \times GVW \)) represents the average wheel weight obtainable using statistical regression. \( X \) is the residual portion beyond the average values, which is important for estimating RC deck fatigue accumulation. Based on WIM data, it is modeled as a truncated skewed double exponential random variable (Fu et al 2002).

**3.2 Prioritized Cost Impact Categories**

A number of categories of cost impact are attributable to increased truck loads as a result of truck weight limit changes. Examples are increased steel bridge-joint fatigue or wearing, and increase of concrete cracking in both severity and quantity. Based on available knowledge on damage mechanism, four cost impact categories are prioritized in this study: 1) steel bridge fatigue, 2) RC bridge deck fatigue, 3) additional inadequate load rating for existing bridges, and 4) higher load requirement for new bridges. This
prioritization also considers the data availability for a reasonably reliable cost estimate. The suggested method for estimation is planned not to require data currently unavailable with State transportation agencies.

**Steel bridge fatigue**

Fatigue of steel bridge components has been extensively investigated (Moses et al. 1987). The vast majority of state agencies have experience with fatigue damage. Under an increase in truck weight limits, fatigue accumulation is expected to increase due to load (and thus stress range) increase, although the truck traffic is expected to decrease if the total payload remains constant.

The following procedure is suggested to estimate the impact cost due to additional fatigue accumulation. 1) Identify possibly vulnerable bridges. 2) Decide to analyze all or a sample of possibly vulnerable bridges. 3) For the analysis of each bridge, generate the TWH under the Base Case and predict the TWH under the Alternative Scenario. 4) Estimate remaining safe life and remaining mean life for both the Base Case and Alternative Scenario. 5) Select responding action for treating possible fatigue failure. 6) Estimate the costs for the selected action. 7) Sum the costs for all bridges. 8) Perform a sensitivity analysis to understand possible controlling effects of the input data.

For each weld detail, the fatigue assessment should follow the AASHTO procedure (1994), to be consistent with current practice:

\[
Y = \frac{fK \times 10^6}{(T_a/T)TC(R_s S_r)^3}
\]

where \(Y\) is the total life in years. \(K\) is a constant tabulated for each type of fatigue sensitive detail in the AASHTO specifications (1994), and \(f\) equal to 1 for safe life and 2 for mean life. \(C\) is the number of cycles for a passage of the fatigue truck. \(R_s\) is a reliability factor. \(S_r\) is the stress range for a passage of the fatigue truck whose weight can be determined using WIM data. (Note that a considered weight limit change may require a change to the fatigue truck model, when a majority of trucks are to be affected in operating behavior.) For the Base Case and the Alternative Scenario, this stress range should be calculated using respective THWs. The Base Case TWH is based on site specific WIM data or the default VMT data (Fu et al 2002). The Alternative Scenario TWH is to be developed using the Base Cases TWH and the prediction method discussed above. \(T\) is the current annual daily truck volume for the outer lane. \(T_a\) is an estimated lifetime-average daily truck volume in the outer lane. The AASHTO specifications (1994) provide the values for these parameters or guidelines about determining the values. Note that the AASHTO procedure for \(T_a\) represents an approximation, which may lead to under- or over-estimates. The following formula is recommended to improve this assessment:

\[
\frac{T_s}{T} = \sum_{i=1,2,\ldots,Y} \frac{(1+u)^i}{Y(1+u)^Y}
\]
where \( u \) is the annual traffic growth rate. It may be estimated using information in the agency’s bridge inventory. \( A \) is the current age of the bridge. Using Eq.11, a non-iterative procedure to find \( Y \) is developed as follows.

\[
Y = \frac{\log[\frac{fK \times 10^6}{TC(R_sS_r)^3}u(1+u)^{4-1} + 1]}{\log(1+u)}
\]  

The expected impact costs are to be estimated as follows:

\[
\text{Expected Impact Cost} = \text{Cost} \left( P_{f,\text{AS}} - P_{f,\text{BC}} \right) > 0
\]  

where subscripts \( \text{AS} \) and \( \text{BC} \) indicate respectively the Base Case and the Alternative Scenario. In case the expected impact cost turns out to be negative, then it is set zero because no impact is expected due to the considered change in weight limits. The impact cost here depends on the action selected in response to possible fatigue failure. This action can be repair, replacement, monitoring, and/or their combinations. The default is recommended to be repair. \( P_{f,\text{AS}} \) and \( P_{f,\text{BC}} \) are probabilities of fatigue failure within a predetermined planning period \( PP \). They are calculated using a truncated lognormal distribution for fatigue life (Fu et al 2002).

**Reinforced concrete deck fatigue**

Based on previous studies on RC deck fatigue under wheel load (Matsui 1991, Perdikaris et al 1993, Fu et al 1992), the following procedure is recommended for assessing fatigue accumulation using a similar format to that in Eq.10

\[
Y_d = \frac{K_d K_p}{(T_a/T) T C_d \left( R_d I P_s P/P_u \right)^{17.95}}
\]  

where \( Y_d \) is the service life of the deck. \( Y_d \) will be the mean service life for the reliability factor \( R_d \) set equal to 1 and the evaluation life for \( R_d \) equal to 1.35. \( T_a \) and \( T \) have been defined in Eq.11. \( C_d \) is the average number of axles per truck. \( P/P_u \) is the equivalent stress ratio caused by wheel load \( P \)

\[
P/P_u = \left[ \sum f_i(P/P_0) (P_i/P_u)^{17.95} \right]^{1/17.95}
\]  

where \( P_u \) is the ultimate shear capacity of the deck. Eq.14 uses the same linear damage accumulation assumption (the Miner’s Law) as for steel fatigue. \( K_d \) is a coefficient that covers the model uncertainty (with respect to the assumed Miner’s Law). \( K_p \) addresses the difference between the state of deck failure recognized in the laboratory and the state of real decks when treatment was applied (Fu et al 2002). \( P_u \) is the nominal ultimate shear strength of the deck and suggested to be estimated as follows, according to the ACI code (1995):
\[ P_u = \left( 2 + 4/\alpha \right) (f_c')^{1/2} b_0 d \gamma < 4 (f_c')^{1/2} b_0 d \gamma \]  

where \( f_c' \) is the concrete compressive strength in psi. \( \alpha \) is the ratio of the tire print’s long side to short side, set equal to 2.5 for a nominal tire print of 0.508 m by 0.203 m (20 in. by 8 in.) for dual tires. \( d \) is the deck’s effective thickness equal to the total thickness minus the bottom cover thickness. It is recommended to also subtract a 0.00635 m (0.25 in.) thick layer from the nominal thickness to account for wearing observed in bridge decks. \( b_0 \) is the perimeter of the critical section, which is defined by the straight lines parallel to and at a distance \( d/2 \) from the edges of the tire print used. \( \gamma \) is a model correction parameter, which is set at 1.55 based on the test data in Perdikaris et al (1993). It should be noted that the above parameters are nominal values of respective variables with uncertainty, as in many other cases for strength or fatigue assessment.

The expected impact cost can be estimated in the same way as Eq.13 except that \( Y_{d,\text{Mean}}^{\text{Remaining}} \) and \( Y_{d,\text{Evaluation Remaining}}^{\text{Remaining}} \) should be used. The cost can be for patching, overlay, or replacement, depending on the responding action selected. Portland cement concrete overlay is selected here as the default action responding to deck deterioration. It should be noted that steel rebar corrosion has been widely accepted to be a major cause of RC deck deterioration, especially in areas where a large amount of de-icing salt is used. In these areas, RC deck fatigue becomes secondary or negligible for deck deterioration or consumption.

**Additional inadequate existing bridges**

Currently in the US highway system, there are a number of bridges that are inadequate in load carrying capacity. This is indicated by their load rating factor lower than 1, according to the AASHTO requirement (1994, 1998). When higher truck loads are legalized or permitted, more bridges will become inadequate. Costs to correct the additional inadequacy are covered in this cost impact category. The new rating factor is recommended to be calculated as follows:

\[ RF_{AS} = RF_{BC}/AF_{\text{rating}} \left( M_{BC,\text{rating vehicle}} / M_{AS,\text{rating vehicle}} \right) \]  

where \( RF_{AS} \) is the rating factor for the Alternative Scenario. \( RF_{BC} \) is the rating factor for the Base Case (likely the existing rating factor). \( M_{BC,\text{rating vehicle}} / M_{AS,\text{rating vehicle}} \) is the ratio between the maximum load effects due to the rating vehicle under the Base Case and due to the new rating vehicle under the Alternative Scenario. The new rating vehicle is a model representing the practical maximum load permissible under the changed weight limits. It could be a set of vehicles. \( AF_{\text{rating}} \) is the ratio between the live load factors for the Base Case and the Alternative Scenario:

\[ AF_{\text{rating}} = \left[ 2W_{AS}^* + 1.41 t(\text{ADTT}_{AS}) \sigma_{AS}^* \right] / \left[ 2W_{BC}^* + 1.41 t(\text{ADTT}_{BC}) \sigma_{BC}^* \right] \]  

where \( W^* \) and \( \sigma^* \) are the mean and standard deviation of the top 20 percent of the TWH, and \( t \) is a function of ADTT (Fu et al 2002). Subscripts _BS_ and _AS_ respectively refer to the
Base Case and the Alternative Scenario. This approach is consistent with the concept of load and resistance factor rating under development (AGLichtenstein 1999) for AASHTO.

In Eq.17, \( \frac{M_{BC,\text{rating vehicle}}}{M_{AS,\text{rating vehicle}}} \) reflects the adjustment to rating due to the new truck model. The adjustment factor \( AF_{\text{rating}} \) is the ratio of the live load factors to cover uncertainty changes in truck weight spectra.

For cost estimation, those bridges that are inadequate with \( RF_{BC}<1 \) under the Base Case should be excluded, because they do not contribute to the cost impact (additional costs). When a bridge is found to be inadequate or overstressed under the Alternative Scenario but adequate under the Base Case (i.e., \( RF_{BC} \geq 1 \) and \( RF_{AS}<1 \)), an action needs to be selected as the basis for cost estimation. It can be, for example, posting, strengthening, replacing, or a combination thereof. Note that, in reality, the decision making process requires information on a number of other factors, not only the load rating.

**Higher load requirement for new bridges**

The bridge design load is supposed to envelope current and expected future loads for the bridge life span. When higher loads are legalized or permitted, the design load needs to be adjusted to assure relatively uniform safety of the bridges. The costs caused by this adjustment are covered in this cost impact category. The analysis will require the following steps. 1) Identify the new bridges to be constructed in the future within PP. 2) Estimate the required design load for each of these bridges under the Alternative Scenario. 3) Estimate the additional costs for each of these bridges under the new design load.

Step 1) may be approximated using the bridges constructed in recent years and averaged to an annual population of new bridges. It can be done using the agency’s bridge inventory. Step 2) is to be accomplished using the following formula for the amount of design load change:

\[
\text{DLCF} = \left( \frac{M_{AS,\text{design vehicle}}}{M_{BC,\text{design vehicle}}} \right) AF_{\text{design}}
\]

\[
M_{AS,\text{design vehicle}} / M_{BC,\text{design vehicle}} \geq 1
\]

\[
AF_{\text{design}} = \frac{2W_{AS}* + 6.9 \sigma_{AS}^*}{2W_{BC}* + 6.9 \sigma_{BC}^*}
\]

where DLCF stands for design load change factor indicating the ratio between the design load effects under the Base Case and the Alternative Scenario. \( M_{AS,\text{design vehicle}} / M_{BC,\text{design vehicle}} \) is the ratio of the maximum load effects due to the design vehicle under the Base Case and the same under the Alternative Scenario. Practically, it should not be lower than 1. Namely, when \( M_{AS,\text{design vehicle}} \) is smaller than \( M_{BC,\text{design vehicle}} \), the design vehicle under the Base Case would be the governing load and the ratio should be taken as 1 in Eq.20. This will assure that the new design load effect will not be lower than the current design load effect. \( AF_{\text{design}} \) is the ratio between the live load factors under the Base Case and the Alternative Scenario. It is an adjustment factor for the design load used to cover
the change in uncertainty associated with the considered Alternative Scenario. It plays a similar role as $AF_{\text{rating}}$ in Eq.18 for additional deficiency in existing bridges.

DLCF in Eq.19 indicates the relative increase in the design load effect. The incremental cost can be accordingly calculated as the impact cost. A set of default costs data have been prepared for this purpose, if no more specific data are available (Fu et al 2002).

3.3. Application Examples

This study included two application examples for the proposed method of estimating costs of a bridge network resulting from truck weight limit changes. The first example investigated such effects due to an increase in permit truck weight from 105 kips to 129 kips for two routes in the State of Idaho. This weight limit increase was required to be in accordance with the federal bridge formula. The second example was for the State of Michigan for a scenario of legalizing 6-axle semi-trailers at a GVW of 97 kips. This truck type is legal in Canada and permissible in several States close to Michigan. The State of Michigan is under pressure to permit or legalize it. Example results indicate the major contributors to the total cost impact being additional inadequate bridges and higher requirement for new bridges.

4. SUMMARY AND CONCLUSIONS

For dealing with increasing truck loads, this paper offers several new methods for response. One of them is a method to determine whether current bridge design load is adequate with respect to current truck operating weights. This method can also be extended for developing rational design loads to respond to observed truck load increases. Another method is offered here to predict the cost impact for bridges resulting from a change in the truck weight limits. This method consists of prediction methods for truck weight histograms and wheel weight histograms as a result of such change. These methods may be used by transportation agencies for rational responses to truck load changes over time.

The cases analyzed using these proposed methods also lead to the following conclusions.

1) Truck loads are very much site dependent. This may cause the safety of bridges vary significantly, although these bridges may have been designed according to the same specifications. 2) The locality of truck loads may lead to very low structural safety of the bridges, which deserves adequate attention. 3) It appears to be more likely that additional deficient bridges and higher design loads will cause dominant cost impact for increase in truck weight limits.

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