This paper reviews the evolution of live load design models for bridges and associated design specification provisions before, during and after the Interstate era, taken as the last 80 years. The types of vehicles on the roads are evaluated and comparisons are made to force effects generated by standard AASHTO design loadings. The introduction of the Federal Bridge Formula is reviewed and a comparison is made to the standard AASHTO HS20 design vehicle used throughout most of the Interstate period. The change in legal loads as well as extra legal loads are reviewed and the implication of the exclusion to the legal load limit made in various states are reviewed and compared to the HS20 loading. The basis for periodic changes to the live load design models, load distribution, and impact is also reviewed. Finally, a brief summary of the development of the post-interstate era live load model, the HL93 loading in the AASHTO LRFD Specifications, is also presented.

INTRODUCTION

Various editions of the AASHTO (or AASHO) Standard Specification for Highway Bridge Design (1) will be referred to herein as the “Standard Specifications.” Likewise, various editions of the AASHTO LRFD Bridge Design Specifications (2) will be referred to as “AASHTO LRFD.” LRFD stands for Load and Resistance Factor Design.

SPECIFIED DESIGN LIVE LOADS PRIOR TO THE INTERSTATE HIGHWAY SYSTEM

The history of highway bridge design codification in the United States, including provisions applicable to the live load, had their origin in a joint effort by designers of highway bridges and railroad bridges, for which specifications already existed, working together on the Special Committee on Specifications for Bridge Design and Construction. Their Final Report on Specifications for Design and Construction of Steel Highway Bridge Superstructure was presented at the spring meeting of ASCE on April 9, 1924, and is published in the 1924 transactions of the American Society of Civil Engineers (3). The table of contents listed the following eight sections to the proposed specification:

1. Loads and stresses,
2. Unit stresses,
3. Details of design,
4. Workmanship
5. Full-size eyebar tests,
6. Weighing and shipping,
7. Structural steel for bridges, and
8. Structural nickel steel.

The 1931 1st edition of AASHO’s Standard Specification for Highway Design, which was based in part on the 1924 committee report, contained a representation of a truck and/or a group of trucks for use in design. The basic design truck was a single unit weighing up to 40 kips, which was known as the H20 truck. Lighter variations of this vehicle were also considered and were designated as HXX, e.g., H15. Groups of H15 trucks, with an occasional H20 truck, were also utilized as a truck-train.

The first edition also instituted the lane load to be used in specific circumstances. For the HS20 loading, this consisted of a uniform load of 0.64 kip/ft and a moving concentrated load or loads. A concentrated load of 26 kip was used for shear and for reaction, two 18-kip concentrated loads were used for negative moment at a support and were positioned in two adjacent spans, and a single 18-kip load was used for all other moment calculations. Proportional lane loads were specified for other HXX loadings.

In the early 1940s the truck was extended into a tractor–semi-trailer combination, known in the 1944 Standard Specifications as the H20-S16-44 and commonly referred to as simply the HS20 truck. This vehicle weighed a total of 72 kips and was comprised of a single steering axle weighing 8 kips and two axles that supported the semi-trailer, each weighing 32 kips. The axle spacing on the semi-trailer could vary from 14 to 30 ft, and it was assumed that there was 14 ft between the steering axle and the adjacent axle that formed part of the tractor. These loads are shown in Figure 1. The HS20 truck was an idealization and did not represent one particular truck, although it was clearly indicative of the group of vehicles commonly known as 3-S2s, e.g., the common 18 wheeler.

![HS20 truck loading](image)
Of course, designing an element of a structure requires more than just the definition of the live-load configuration. It also has to include, at a minimum, a factor to account for dynamic application of live loads and vibration, commonly called the impact factor; a way of deciding how many design lanes fit on the roadway; a means of considering the probability of simultaneous loadings in different lanes of the structure positioned to produce the maximum response in the element under design, commonly called multiple presence factors; as well as a means of associating the portion of each of those lanes to be carried by an individual element, often referred to as the girder distribution factor (GDF). The 1st edition of the AASHO Standard Specifications dealt with each of these issues as follows:

- **Impact**: Remarkably, impact specified as \( \frac{50}{(L+125)} \). Despite all of the testing that has been done on structures in the last 60 years, this factor remains in use today in the 17th edition of the Standard Specifications, even though it does not correlate particularly well to actual measured dynamic amplification.

- **Traffic lanes**: Truck lanes or equivalent loads would be placed in lanes 9-ft wide, which at that time was the width specified for the standard truck. Within the roadway between curbs, the traffic lanes were assumed to occupy any position which would produce the maximum stress, but would not result in overlapping of adjacent lanes. The center of the lane was to be at least 4.5 ft from the face of the curb which, considering that the truck was 9-ft wide, placed the wheels adjacent to the curb.

- **Multiple presence**: The basic provision was that, with the number of traffic lanes specified as the curb-to-curb width divided by 9 ft, the associated multiple presence factor included reducing the load by 1% for each foot of roadway over 18 ft with a maximum reduction of 25% when the width reaches 43 ft. There were other caveats and exceptions which will not be dealt with herein.

- **GDF**: For the most common types of bridges, the distribution factor was the familiar \( \frac{S}{D} \) expression where \( S \) is the girder spacing. For the case of one traffic lane for stringers supporting a concrete deck the GDF was \( \frac{S}{6.0} \) with a limited stringer spacing of 6 ft for application, and for two or more lanes, \( \frac{S}{5.5} \), with a limiting stringer spacing of 10 ft. If the spacing of stringers exceeded the maximum value specified, the load per stringer was determined assuming a hinge over each girder and using a simple span distribution between stringers, a method referred to in AASHTO LRFD as “the lever rule.”

**START OF THE INTERSTATE ERA**

At the start of the Interstate era the specifications had advanced to the 6th edition issued in 1953. This specification defined the default design loads as the HS20 truck and lane loads defined above. The other features of load were as follows:

- **Impact remained to be computed as specified above.**

- **Multiple presence** was the now-familiar one or two lanes at 100%, three lanes at 90%, and four or more at 75%.

- **Traffic lanes** were assumed to be 12 ft wide, with a truck or lane occupying 10 ft of the 12 ft, positioned therein for maximum effect.
• The number of traffic lanes was determined from the provisions reproduced below from the 6th edition (1953) of the Standard Specifications.

3.2.6—Traffic Lanes
Where the spacing of main supporting members exceeded 6.5 feet for timber floors or 10.5 feet for concrete or steel grid floors, the lane loading or standard trucks shall be assumed to occupy a width of 10 feet. These loads shall be placed in design traffic lanes having a width of

\[ W = \frac{W_c}{N} \]  

(1)

where

- \( W_c \) = roadway width between curbs exclusive of median strip
- \( N \) = number of design traffic lanes as shown in the following table
- \( W \) = width of design traffic lane

\[
\begin{array}{lcc}
W_c \text{ (in feet)} & N \\
20 \text{ to } 30 \text{ inclusive} & 2 \\
\text{over } 30 \text{ to } 42 \text{ inclusive} & 3 \\
\text{over } 42 \text{ to } 54 \text{ inclusive} & 4 \\
\text{over } 54 \text{ to } 66 \text{ inclusive} & 5 \\
\text{over } 66 \text{ to } 78 \text{ inclusive} & 6 \\
\text{over } 78 \text{ to } 90 \text{ inclusive} & 7 \\
\text{over } 90 \text{ to } 102 \text{ inclusive} & 8 \\
\text{over } 102 \text{ to } 114 \text{ inclusive} & 9 \\
\text{over } 114 \text{ to } 126 \text{ inclusive} & 10 \\
\end{array}
\]

The lane loadings or standard trucks shall be assumed to occupy any position within their individual design traffic lane (\( W \)) which will produce the maximum stress.

• Starting with the 7th edition (1957) of the Standard Specifications (or perhaps in a 1954, 1955, or 1956 Interim), the distribution factor for stringer elements was specified as \( S/5.5 \) for wheel loads despite the growing amount of research that showed that this could be quite approximate. Research clearly showed that to get a better, less empirical, distribution factor required some sort of recognition of the ratio of the longitudinal to the transverse rigidity in the bridge deck.

But design and operations were, and still are, different issues. As summarized in *Federal Size Regulations for Commercial Motor Vehicles* (4), in 1956 Congress legislated maximum axle weight, gross vehicle weight, and width limits for trucks operating on Interstate highways based on limits recommended in 1946 by AASHO, now AASHTO: 18,000 lbs on a single axle, 32,000
lbs on a tandem axle, and 73,280 lbs gross weight. The federal law also authorized states to allow operation of heavier trucks on Interstate highways, but only if such operation was legal in the state prior to July 1, 1956. This became known as a “grandfather right.” The grandfather clause was, therefore, enacted to avoid a rollback of legal vehicle weights in those states, while the AASHO standard set an upper limit on weights otherwise allowable. There are three different grandfather clauses in Section 127, Title 23, U.S.C. The first, enacted in 1956, deals principally with axle weights, gross weights, and permit practices; the second, adopted in 1975, applies to bridge formula and axle spacing tables; and the third, enacted in 1991, ratified State practices with respect to long combination vehicles (LCVs). These vehicles will be discussed in more detail below.

**CHANGES TO REGULATORY CLIMATE AND DESIGN LIVE LOAD DURING THE INTERSTATE DESIGN YEARS**

Throughout the Interstate era, the typical live load remained the HS20 loading, as specified above, although variations to this are discussed below. The multiple presence factor and the girder distribution factors remained as specified in the beginning of the era, but again, the continuing weight of research, as well as changes in design configurations, began to render many of these approximations less and less accurate.

The number of design lanes continued to be determined as the curb-to-curb width divided by 12, but in 1974, some subtleties about the placing of the lanes and the trucks within the lanes were introduced as follows:

**Article 1.2.6—Traffic Lanes**

The lane loading or standard truck shall be assumed to occupy a width of 10 feet.

These loads shall be placed in 12-foot wide design traffic lanes, spaced across the entire bridge roadway width, in numbers and positions required to produce the maximum stress in the member under consideration. Roadway width shall be the distance between curbs. Fractional parts of design lanes shall not be used. Roadway widths from 20 to 24 feet shall have two design lanes each equal to one-half the roadway width.

The lane loadings or standard trucks having a 10-foot width shall be assumed to occupy any position within their individual design traffic lane, which will produce the maximum stress.”

These changes had made very little difference in the way the load was distributed to stringer elements as most designs continued to use the simple $S/5.5$ distribution factor, but, since the lane positions and the vehicle position within the lanes now varied, this made for some interesting complications for the positioning of loads on transverse elements such as floor beams, especially in areas where structures had gores to accommodate ramps.
The operation issues were further complicated by evolving truck configurations. As stated in Farris’ *Should the Federal Government Allow the States to Increase Truck-size Limits?* (5), “It is under the grandfather provisions that LCVs are able to operate today in some states. Those states with weight limits above 80,000 pounds in 1956 are allowed to choose whether LCVs can operate on their highways, while states that had lower limits 35 years ago do not have this choice.”

The term LCV commonly refers to one of three types of vehicles (5):

- A truck tractor pulling three 28- or 28.5-ft trailers (triples),
- Tractor–trailer combinations involving two 48- or 45-ft trailers (turnpike doubles), or
- Double 48s, or tractor–trailer combinations involving one 48- or 45-ft trailer and one 28- or 29-ft trailer (Rocky Mountain doubles).

Individual states are free to allow longer vehicles on Interstates and the national network (NN), but they must permit vehicles of at least this length. Widespread use of LCVs is currently limited, not by statutory limits on length, but by a federal limit on overall vehicle weight. A summary of states permitting LCV’s on at least some highways by 1991 is shown in Table 1.

The actual configurations of trucks are almost limitless, and it has long been recognized that the regulatory control of vehicles is a significantly different matter than the choice of a design model upon which to base calculations. The links between these two needs are state legal loads and the federal bridge formula which will be discussed further below. These two regulatory devices have the objective of recognizing that the commercial needs of the Country can only be satisfied by a plethora of vehicle configurations and that it is sometimes necessary that these configurations create significantly larger force effects in structures than those calculated using the HS20 design model. The AASHTO *Manual for Maintenance Inspection of Bridges* (7) has long recognized this by prescribing two stress levels commonly referred to as the inventory level, which approximates design stresses or design force effects, and the operating level, which

<table>
<thead>
<tr>
<th>State</th>
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<tbody>
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<td>Arizona</td>
<td>New Mexico</td>
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</tr>
<tr>
<td>Montana</td>
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</tbody>
</table>
results in approximately 1/3 more total force effect on the basis that it will occur, although relatively infrequently. The range between these two stress levels is often used as a basis of establishing fleets of vehicles which produce more force effect than legal loads, but for which permits will be routinely granted without the need for individual structural calculations.

In response to energy use concerns, the Federal Aid Highway Amendment of 1974 increased the allowable single axle, tandem axle, and gross weight limits on the Interstate to 20,000, 34,000, and 80,000 lbs, respectively, although not all the states adopted these limits (4). The bridge formula and a corresponding grandfather clause were added at the same time. This second grandfather clause allows states to retain any bridge formulas or axle spacing tables in effect on January 4, 1975, which allowed greater vehicle weights at the same axle spacing than the new federal bridge formula. However, states were not required to adopt these higher limits. Some did not (4).

A new design live load configuration was added in the 1976 Interims to the AASHTO Standard Specifications. This configuration consisted of two 24-kip axles spaced 4 ft apart, i.e., a design tandem. It appeared in a new Article 1.2.5(G) entitled Interstate Highway Bridge Loading and was known as the Alternate Military Loading.

In 1982 Congress required that all states allow on their Interstate highways loads of 20,000 lbs on single axles, 34,000 lbs on tandem axles, 80,000 lbs total for a vehicle, and enforce the federal bridge formula. The federal length and width provisions were extended beyond the Interstate system to the designated NN for large trucks and related access roads. States having grandfather rights were authorized to determine what vehicles and operating situations would be considered grandfatherable.

Throughout the late 1970s and early 1980s, some states responded to the observed increase in volume and weight of truck traffic by increasing the standard HSXX load to HS25. This consisted of a 90-kip truck with the same axle spacing and weight proportions of the HS20 truck, an increase of 25% in the lane load and possibly the same increase in the alternate military loading. It is not clear whether all of the states which opted to use HS25 increased all three loadings proportionally. Consideration of an increase to HS25 was often associated with a state’s adoption of the load factor design methodology. The increase in design load was seen as a means of maintaining a reserve in capacity similar to that provided by allowable stress design. The reserve strength was utilized in operating ratings and for the issuance of overload permits. As shown later herein, this was a step in the right direction, but did not really relate to the multitude of truck configurations using the highways.

THE FEDERAL BRIDGE FORMULA

The federal bridge formula (8) limits the gross weight on any group of axles to the lesser of the cap or a value determined by the number of axles and the distance between them. The heavier the weight the greater the spacing required. States with grandfathered bridge formulas in effect before 1975 did not have to enforce the federal formula. Limits on axle loads also applied and capped the weight allowed by the federal bridge formula, which is given below.

\[ W = 500 \left\{ \frac{LN}{(N - 1)} + 12N + 36 \right\} \]  

where
\[ W = \] the maximum weight in pounds that can be carried on a group of two or more axles to the nearest 500 lbs;

\[ L = \] the spacing in feet between the outer axles of any two or more axles; and

\[ N = \] the number of axles being considered.

The bridge formula is intended to limit the weights of shorter trucks to levels which will limit the overstress in well maintained bridges designed with the HS20 loading (including the lane load) to about 3\%, and in well maintained HS15 bridges to about 30\%.

For simplicity, the federal bridge formula was reduced to a table that showed the permissible gross loads for vehicles with various numbers of axles for different truck lengths \((8)\).

It may be of some interest to check the HS20 design truck with the federal bridge formula. For the sake of this comparison, we will replace the two 32-kip axles with 16-kip tandem axles spaced 4 ft apart. This makes the axle distances (Figure 1) 12 ft, 4 ft, the variable spacing 10 to 26 ft, and 4 ft. If we consider the five-axle truck, the total length from the steering axle to the rear most axle of the tandem group varies from 30 to 46 ft. In order to be acceptable as a 72-kip truck by the federal bridge formula, the overall length would have to be at least 39 ft. Anything shorter than this, i.e., 30 to 38 ft, would not pass the formula. But, the formula also has to be applied to subconfigurations of the axles. If we consider the out-to-out of the two tandem axle groups the distance would vary from 18 to 34 ft. To be acceptable as a 64-kip axle string, the federal bridge formula would require a length of at least 33 ft, i.e., anything between 18 and 32 ft would be unacceptable.

SOME ALTERNATIVE DESIGN AND REGULATORY LOAD MODELS PROPOSED IN THE LATTER HALF OF THE INTERSTATE ERA

As a result of the evolution summarized above, it became increasingly apparent that the HS loading did not bear a uniform relationship to many of the vehicles allowed on the roads. It was becoming out-of-date. In developing a new design specification, i.e., the AASHTO LRFD, it became apparent quite early in the development process that if the objective of developing a new specification was a more uniform and consistent safety of bridges, a new live load model would be necessary in order to produce that consistency.

As outlined above, the current regulatory situation embodied in state legal loads, unanalyzed permit loads, grandfather provisions and the federal bridge formula, could provide one basis for identifying live-load force effects which could be extended into a new design load. In 1990, the Transportation Research Board published Special Report 225: Truck Weight Limits: Issues and Options \((9)\) summarizing a study into the state legal loads, grandfather provisions, the current bridge formula, and various attempts to extend the bridge formula, or to develop other regulatory models. This study contained extensive information on the estimated benefits from more efficient movement of goods compared to the cost and accelerated damage to roadways and bridges. A group of vehicles were identified in Special Report 225 and were made available to the LRFD development group as part of personal correspondence. In some cases the final published configurations were slightly different but not enough so to matter. In other cases Special Report 225 was a starting point. Vehicle configurations considered are shown in Figure 2.
• The vehicles shown in Figure 2a as AASHTO rating vehicles were thought to be typical legal load types used by many states for rating, instead of the HS20 load configuration.
• Vehicle configurations representing the grandfather provision exclusions to legal loads available in various states are shown in Figure 2b. They represent various types of special hauling vehicles and LCVs common in the United States. This family of vehicles is referred to herein as exclusion loads. The LCVs are clearly evident in the longer configurations.
• A proposal by the National Truck Weight Action Committee (NTWAC) was reduced to three special hauling vehicles (SHVs) shown in Figure 2c. These vehicles can be represented as a three-axle single unit weighing 80 kips, a four-axle single unit with a tridem axle unit weighing a total of 82 kips, and a 3-S-2 weighing 110 kips.
• Vehicles representative of a project to produce a modified bridge formula conducted by the Texas Transportation Institute (TTI), can be embodied in the four configurations shown in Figure 2d, which are similar in shape, but not weight, to the NTWAC vehicles.
• The Canadian interprovincial loads, which resulted from a 1988 agreement by the Canadian Council of Ministries of Transportation and Highway Safety, produced a common set of weight limits (RTAC 1988) for tractor-semi-trailers and double trailer combinations, as shown in Figure 2e. Some of these vehicles are similar to the TTI vehicles. An additional axle is added to the 3-S2-2, and the spacings and weights of the axles are somewhat different, and a fifth configuration, called a 3-S2-4, is also added.
• The extended bridge formula vehicles, shown in Figure 2f, are LCVs intended to extend the bridge formula past 80 kips.
• Turner trucks, two-vehicle combinations known as the Turner A and Turner B Trucks, shown in Figure 2g, which were developed on the principle that pavement would be less damaged by vehicles with increased gross vehicle weight if the weight per axle was reduced by adding additional axles.

![Figure 2a: AASHTO rating vehicles.](image)
FIGURE 2 (continued)  (b) Exclusion vehicles.
FIGURE 2 (continued)  (c) NTWAC special hauling vehicles, and (d) modified TTI formula vehicles.
FIGURE 2 (continued) (e) Canadian interprovincial load vehicles, and (f) extended bridge formula vehicles.
COMPARISON OF EFFECTS OF LOAD PROPOSALS TO THE STANDARD INTERSTATE DESIGN LIVE LOAD

The force effect of one lane loaded, i.e., without distribution, on bridge structures from these various families of vehicles were studied by calculating the envelope of force effects from each of the representative vehicles in a family for:

- Centerline moment of a simply-supported beam, i.e., not the absolute maximum moment;
- Positive and negative moment at the 0.4L point of a two-span continuous girder, with equal spans;
- Positive and negative end shear ($+V_{ab}$, $-V_{ab}$) and shear at an interior support ($-V_{ba}$) of a two-span continuous girder with equal spans; and
- Negative moment at the interior pier of a two-span continuous girder with equal spans.

Figures 3 through 9 compare the force effects created by the HS20 truck, the NTWAC trucks, the Turner trucks, the TTI trucks, the extended bridge formula trucks (EXTBRFOR) and the exclusion (EXCL) trucks for various span lengths and expressed in either kips or kip/ft, as appropriate. The AASHTO rating vehicles were not found to govern these conditions and have not been plotted on the figures.

In the case of negative moment of an interior support, the results shown in these figures correspond to one vehicle on the span. Additional studies evaluated the effects of two vehicles on the span, and this possibility is accounted for in Figure 10 as seen in the large ratios for negative moment at an interior support. Note that in the case of negative moment support, the HS loading is often actually controlled by the lane load and a pair of concentrated loads, and hence, it is more representative of that force effect than the HS loading appears to be for moment at the centerline of a simple span, as shown in Figure 6.
FIGURE 3  Centerline moments in kip-ft: simple span.

FIGURE 4  Negative moments in kip-ft at 0.4L.
FIGURE 5  Negative moments in kip-ft at support.

FIGURE 6  Positive moments in kip-ft at 0.4L.
FIGURE 7  Positive shear in kips at $+V_{ab}$.

FIGURE 8  Negative shear in kips at $-V_{ab}$. 
For most of Figures 3 through 9 the effect of the exclusion loads is clearly evident. These loads were selected as the basis of developing a new notional national design load (10).

Figure 10 shows a comparison of the various moment-type force effects, identified above, for spans from 5 to 150 ft generated by the exclusion vehicles compared to the HS20 truck. This comparison is developed by plotting the ratio of the force effect from the envelope of exclusion vehicles divided by the corresponding force effect from the HS20 vehicle on a vertical axis, against span length on the horizontal axis. Thus, a complete match of force effects, indicating that the HS20 vehicle was an accurate and representative model of the exclusion loads, would be indicated by a horizontal line passing through the vertical axis at a value of 1.0. Corresponding information for the shear force effects identified above are shown in Figure 11, in which $V_{ab}$ is the shear at the simply-supported end and $V_{ba}$ is the shear adjacent to the interior support. Figures 10 and 11, taken together, show that the HS20 vehicle was not representative of current loads on the highways and documents the need to develop a new live load model.

**PREPARING FOR THE POST-INTERSTATE DESIGN ERA**

Three decades after the start of the Interstate era, state bridge engineers, through the AASHTO Highway Subcommittee on Bridges and Structures, authorized development of an updated bridge design specification which became known as the AASHTO LRFD. Recognizing the change in vehicle configurations and weights as summarized above, it became clear that a new design live load model was needed.

Five candidate notional loads were identified early in the search for a new design model.
A single vehicle, called the HTL57, weighing a total of 114 kips and having a fixed wheel base and fixed axle spacing and weights shown in Figure 12. This vehicle is similar to the design vehicle contained in the 1983 edition of the *Ontario Highway Bridge Design Code* (11).
A family of three loads shown in Figure 13, consisting of a tandem, a four-axle single unit, with a tridem rear combination, and a 3-S-2 axle configuration taken together with a uniform load, preceding and following that axle grouping.
A design family called HL93 consisting of subsets or combinations of a design tandem similar to that shown in Figure 13, the HS20 truck shown in Figure 1, and a uniform load of 0.64 kips per running foot of lane, as shown in Figure 14. Also shown in Figure 14 is an extension of this loading to include 90% of two HS20 trucks with 14-ft wheel bases and 90% of the uniform load to also be investigated for negative moment near supports and interior reactions of continuous spans greater than 50 feet.

- An equivalent uniform load in kip/ft of lane required to produce the same force effect as the envelope of the exclusion vehicles for various span lengths as shown in Figure 15.
- Not shown is a slight variation of the combination of the HS vehicle and the uniform load, which involves an HS25 load, followed and preceded by a uniform load of 0.48 kips per running foot of lane, with the uniformly distributed load interrupted for the HS vehicle.

The candidate live loads were processed using influence line analysis. Generally speaking, it was found that the load model involving a combination of either a pair of 25 kip tandem axles and the uniform load, or the HS20 and the uniform load, seem to produce the best fit to the exclusion vehicles. This is summarized in tabular form in Table 2, in which the mean and standard deviations for each of the force effects indicated for each of the models under consideration. This summary also indicates that the model consisting of either the tandem plus the uniform load, or the HS20 plus the uniform load, produce the best results.

A summary of the force effect ratios for the exclusion vehicles divided by either the tandem plus the uniform load, or the HS20 truck plus the uniform load, is shown in Figures 16 and 17. It can be seen that the results for force effects under consideration are tightly clustered, very parallel, and form bands of data which are essentially horizontal. The tight clustering of data for the various force effects indicates that one notional model can be developed for all of the force effects under consideration. The fact that the data is essentially horizontal indicates that both the model and the load factor applied to live load can be independent of span length. The tight clustering of all the data for all force effects further indicates that one live load factor will suffice.

![FIGURE 15 Equivalent uniform load.](image-url)
TABLE 2  Live Load Models Versus Exclusion Load Mean and Standard Deviation

<table>
<thead>
<tr>
<th></th>
<th>HS20</th>
<th>HS20+0.64 (Prop.)</th>
<th>HS25+0.48</th>
<th>HTL</th>
<th>FAMILY-3</th>
<th>HTL MOD LF</th>
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<tbody>
<tr>
<td>–M 0.4L Mean</td>
<td>1.600</td>
<td>1.060</td>
<td>0.982</td>
<td>1.189</td>
<td>1.048</td>
<td>1.091</td>
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<td>0.1290</td>
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<tr>
<td>–M @SUPT Mean</td>
<td>1.111</td>
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<td>0.835</td>
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<td>0.871</td>
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<tr>
<td>Std Dev</td>
<td>0.1253</td>
<td>0.0929</td>
<td>0.0755</td>
<td>0.1290</td>
<td>0.0379</td>
<td>0.0389</td>
</tr>
<tr>
<td>–V_{ba} Mean</td>
<td>1.461</td>
<td>1.011</td>
<td>0.932</td>
<td>1.111</td>
<td>1.006</td>
<td>1.022</td>
</tr>
<tr>
<td>Std Dev</td>
<td>0.1448</td>
<td>0.0355</td>
<td>0.0475</td>
<td>0.1093</td>
<td>0.0522</td>
<td>0.0540</td>
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<tr>
<td>+V_{ab} Mean</td>
<td>1.415</td>
<td>1.024</td>
<td>0.919</td>
<td>1.132</td>
<td>1.017</td>
<td>1.232</td>
</tr>
<tr>
<td>Std Dev</td>
<td>0.1447</td>
<td>0.0391</td>
<td>0.0391</td>
<td>0.1027</td>
<td>0.0558</td>
<td>0.2792</td>
</tr>
</tbody>
</table>

FIGURE 16  EXCL/HS20+0.64 kips/ft or dual 25 kip moment ratio.

Thus, the combination of the tandem with the uniform load and the HS20 with the uniform load, were shown to be an adequate basis for a notional design load in the LRFD Specifications. The process of developing the notional design load described above relates to the representation in the specifications. In a calibrated, reliability-based design specification such as AASHTO LRFD the notional design load must still be shown to be a reasonable fit to a statistically projected live load. In the case of AASHTO LRFD the process of developing the statistically projected load and the determination of load and resistance based in part on both the notional design load and the statistically projected live load is described in (12).
But, even if a more realistic live load model is possible, GDF, impact, and multiple presences must still be considered.

Early in the Interstate era, when beam and girder spans were relatively short and the elements were relatively close together, the simple expressions for GDF yielded reasonably realistic results. However, as longer girders replaced truss spans out to over 500 ft, and the girder spacing changed from 6 or 7 ft to 12, 13, or 14 ft, the simple $S/D$ expression became more and more unrealistic. Fortunately, the results obtained with this simple approximation were usually quite conservative. This has been verified through dozens of field stress measurements, as well as analytic investigations going back 40 years or more. The literature is full of countless research efforts oriented towards developing a better approximation.

In the design environment, the introduction of matrix structural analysis and, eventually finite element analysis, made it practical for designers to make grid or continuum models of many types of bridges, including the ubiquitous stringer bridge. Early design oriented grid analyses were being done 15 years into the Interstate era. The growing need for curved structures, the competition between the steel concrete industries, the requirement for alternative designs in steel and concrete, and the rise of contractor alternatives and value engineering all drove the bridge industry to improve on $S/D$ distribution factors.

Some specifications, such as the OHBDC, have developed charts and tables in order to implement an orthotropic plate analogy requiring the longitudinal stiffness of the bridge, the transverse stiffness of the bridge, and the cross-term used in plate theory. Other approaches have been to continue to evolve, newer and presumable better equations.

After consideration of both the orthotropic plate analogy and research efforts, the decision was made to base load distribution in the LRFD specifications on a two-level approach. The first level is to provide a relatively simple set of equations; the second level is to collect and validate the use of two- or three-dimensional methods.

The simplified equations were based on the work of Zokaie et al (13), done under the auspices of the NCHRP and AASHTO Technical Committee T-5 for Loads and Load
Distribution. Generally speaking, the equations developed were somewhat more complex than $S/D$ because they attempted to take into account the relative longitudinal and transverse rigidities of the bridge, as well as the influence of span length. As an example, one such equation applicable to bending moments in interior beams in stringer-type bridges is given below.

$$g = 0.075 + \left( \frac{S}{9.5} \right)^{0.6} \left( \frac{S}{L} \right)^{0.2} \left( \frac{K_g}{12 L t_s^2} \right)^{0.1}$$  \hspace{1cm} (3)$$

where, subject to some numerical limits:

- $S$ = girder spacing (ft);
- $L$ = span length (ft);
- $K_g = n (I + A e_g^2)$ (in.$^4$);
- $A$ = area of the beam only, i.e., the noncomposite section (in.$^2$);
- $N$ = modular ratio of girder material to slab material;
- $I$ = moment of inertia of the beam only, i.e., the noncomposite section (in.$^4$);
- $e_g$ = eccentricity of the girder (i.e., distance from centroid of girder to midpoint of slab) (in.); and
- $t_s$ = slab thickness (in.).

Figure 18 shows a comparison of the distribution factor $S/D$ and the results of Equation 3 neglecting the last term for various girder spacings, and, in the case of Equation 3, various span lengths. Neglecting the last term is generally found to be conservative. As shown in Figure 18, the distribution factor produced by $S/D$ is generally conservative, sometimes as much as 40% conservative for the girder spacings and spans shown. Interestingly the figure shows that there is as a range of close girder spacings and short spans for which $S/D$ where $D$ is 5.5 is generally not conservative.

Equation 3 is thought by some to be more complicated than many designers would prefer, however, and an effort was completed under NCHRP Project 12-62 to simplify the load distribution provisions. As of this writing (early 2006) a final report on NCHRP 12-62 is still in progress; a summary has been published in (14) showing that the proposed provisions are simpler and even more accurate than the current provisions in AASHTO LRFD. Figures 19 and 20 are from that research. Figure 19 shows the results of $S/5.5$ for several hundred bridges and a trend line for that data, compared to the results of grid analysis which have a slope of 1. This figure clearly shows how inaccurate, but generally conservative the $S/D$-type factors were. Figure 20 shows a re-evaluation of the equations in the LRFD Specifications. The improvement is clearly evident.

In considering both the live load model and the distribution factor, it seems apparent that one of the reasons why the Standard Specifications have been as successful as they have been is that there are compensating tendencies for the live load model and the distribution factors therein. Generally, the live load model appears to understate the types of vehicles currently allowed on the highway system. However, the distribution factor appears to produce a conservative result, thus offsetting that difference in the live load proportion delivered to the member under design. One of the premises in developing the LRFD specifications was to try to make the individual components of the design process more realistic and more accurate so that the effect of changes with future knowledge will be more evident and more discernible.

FIGURE 19 Comparison of S/D to grid analysis distribution.

FIGURE 20 Comparison of LRFD factor to grid analysis.
While the original 1931 impact factor is still specified in the Standard Specification, newer specifications, such as AASHTO LRFD and the CSA Canadian Highway Bridge Design Code (15) now specify a constant, i.e., span independent, percentage of the weight of the design axle string to be applied as an impact load. Modern specifications may also vary impact for different limit states and for special wheel intensive or geometry challenged components such as expansion dams.

Finally, multiple presence factors can now be more fully justified by probability-based calculations rather than judgment.

**SUMMARY**

From a design and operational point of view, the Interstate era started with a legacy of more than two decades of experience with the AASHTO Standard Specifications. The years following the adoption of the first edition of the specifications in 1931 through the beginning of the Interstate era in 1956 saw a steady stream of increased knowledge through research projects and field evaluations.

As the IHS was being designed, research efforts expanded and knowledge of the behavior of bridges grew rapidly. At the same time, the need to recognize operational and economic issues related to the states that already had vehicles using the highway system that exceeded the Interstate limits of 73,280 lb (later increased to 80,000 lb) lead to the grandfather provisions that permitted the long combination vehicles and other heavier configurations in some states. Evidence accumulated that some of the provisions used to apply the loads to structures, including the load distribution and impact provisions, were simply not properly describing the response of the structure. Changes in the configurations of bridges, including wider girder spacings and longer spans made of girders rather than trusses, accentuated the problem. Some states reacted to this by increasing the design vehicle to HS25, but this vehicle still did not model the loading produced by trucks particularly well.

All of these factors eventually lead to the decision to upgrade the specifications including changes to the load model, the load distribution provisions, the impact provisions, and the multiple presence factors. All of this became embodied in the post Interstate design era AASHTO LRFD. With these new design provisions, design is based on a more realistic evaluation of structural response and behavior than was available at the beginning of the Interstate era.

**REFERENCES**

5. Farris, R. E. *Should the Federal Government Allow the States to Increase Truck-Size Limits?* Cato