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50 Years of Interstate Structures

Past, Present, and Future

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TRANSPORTATION RESEARCH CIRCULAR E-C104

Fifty Years of Interstate Structures
Past, Present, and Future

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Foreword

The 50th Anniversary of the Interstate Highway System marks a milestone for one of the largest public works projects ever undertaken in the United States—and perhaps in the world. This Circular documents some of the key legislation, specifications development, and advances in research, design, and construction for bridges and other highway structures since the signing of the Federal Highway Act of 1956. The eight papers in this Circular were sponsored by the Structures Section (AFF00) committees: General Structures (AFF10); Steel Bridges (AFF20); Concrete Bridges (AFF30); Dynamics and Field Testing of Bridges (AFF40); Seismic Design of Bridges (AFF50); Tunnels and Underground Structures (AFF60); Culverts and Hydraulic Structures (AFF70); and Structural Fiber-Reinforced Polymers (AFF80).

Advancements have been made possible through research, empirical knowledge gained during construction, and hindsight gleaned from failures. The papers also look forward to the future, commenting on the challenges ahead, including possible solutions to accommodate the nation's ever increasing vehicle loads and numbers. There are numerous possible techniques, methods, and materials on the horizon that may help to manage these challenges.

The eight papers were presented at the 85th Annual Meeting of the Transportation Research Board (TRB) during January 2006 in Washington. D.C. They were presented in two successive podium sessions titled "50 Years of Interstate Structures: Successes and Future Developments (Parts 1 and 2)." Immediately following the podium sessions, a brief poster session was held; audience members and other interested persons were able to interact with the authors.

The chairs of the TRB Structures Section coordinated the preparation, conducted peer reviews, and some even coauthored the papers presented in this Circular. Great appreciation and thanks are due for their fine leadership throughout this effort. All of the authors made major contributions of time and effort, and without their significant knowledge and dedication, the podium sessions, poster session, and Circular could not have been possible. We thank the chairs and authors for their invaluable service to the industry.

Finally, heartfelt thanks are extended to TRB Engineer of Design Stephen Maher for his dedication and continued support and assistance that made this special project possible.

—Mary Lou Ralls Newman
Chair
TRB Structures Section

Contents

GENERAL STRUCTURES COMMITTEE

Evolution of Vehicular Live Load Models During the Interstate Design Era and Beyond

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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.

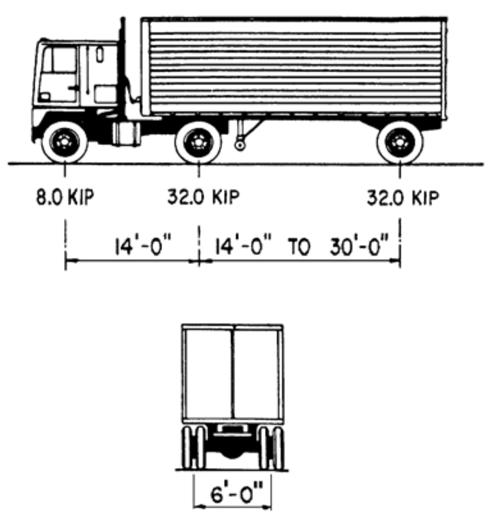


FIGURE 1 HS20 truck loading.

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 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 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 $S/6.0$ with a limited stringer spacing of 6 ft for application, and for two or more lanes, $S/5.5$, 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
>
> $$W = \frac{W_c}{N} \tag{1}$$
>
> 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

W_c (in feet)	N
20 to 30 inclusive	2
over 30 to 42 inclusive	3
over 42 to 54 inclusive	4
over 54 to 66 inclusive	5
over 66 to 78 inclusive	6
over 78 to 90 inclusive	7
over 90 to 102 inclusive	8
over 102 to 114 inclusive	9
over 114 to 126 inclusive	10

> 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 1 Operations of Longer Combination: Vehicles, 1991 (*6*)

State	
Alaska	Nebraska
Arizona	New Mexico
Colorado	Nevada
Florida	New York
Iowa	North Dakota
Idaho	Ohio
Indiana	Oklahoma
Kansas	Oregon
Massachusetts	South Dakoa
Michigan	Utah
Missouri	Washington
Mississippi	Wyoming
Montana	

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.

$$W = 500\ \{[LN/(N-1)] + 12N + 36\} \qquad (2)$$

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 2*a* 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 2*b*. 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 2*c*. 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 2*d*, 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 2*e*. 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 2*f*, 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 2*g*, 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.

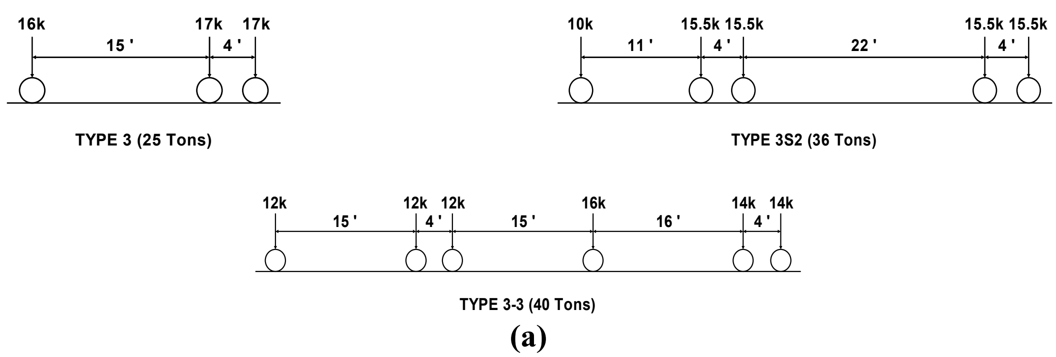
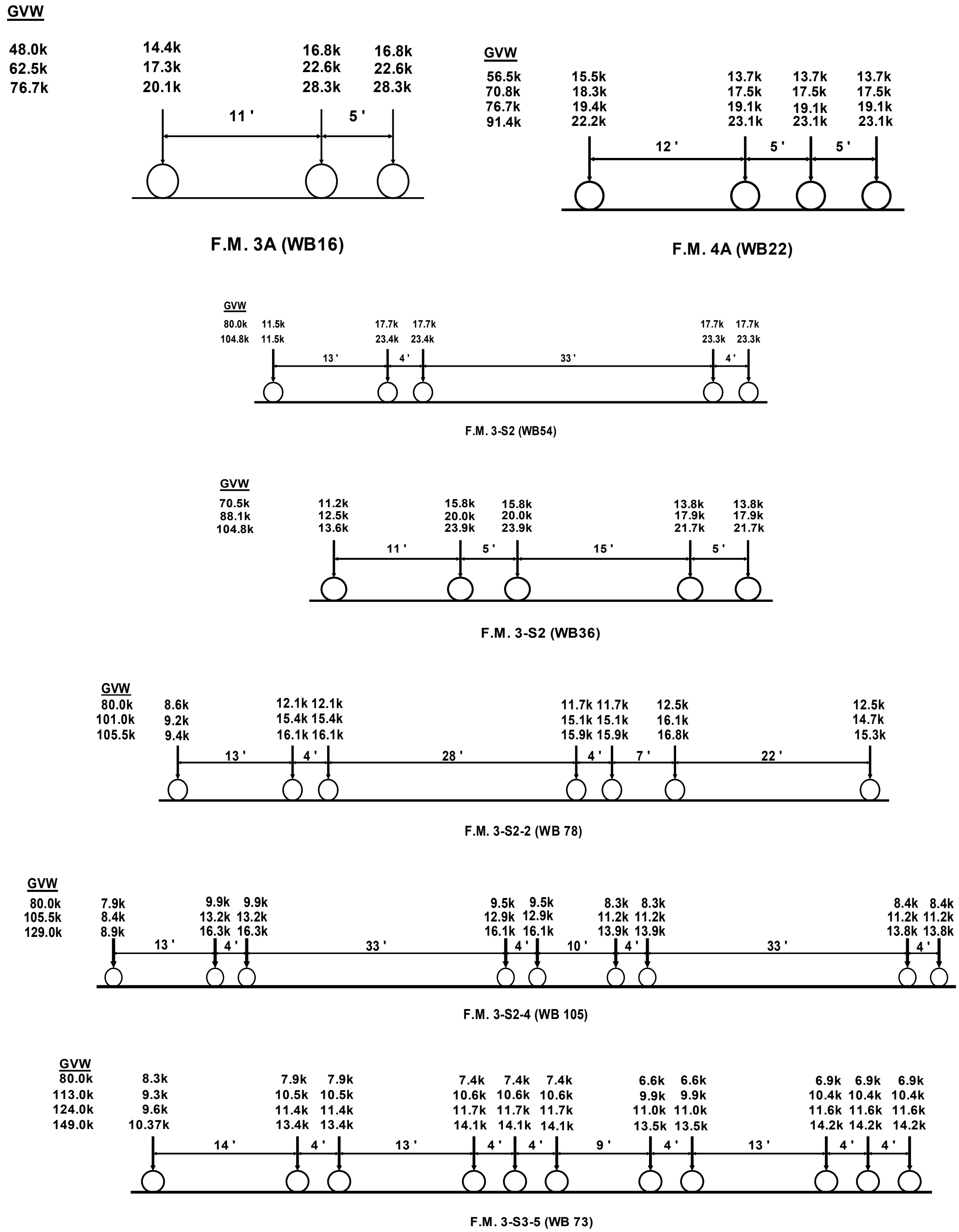


FIGURE 2 (*a*) AASHTO rating vehicles.

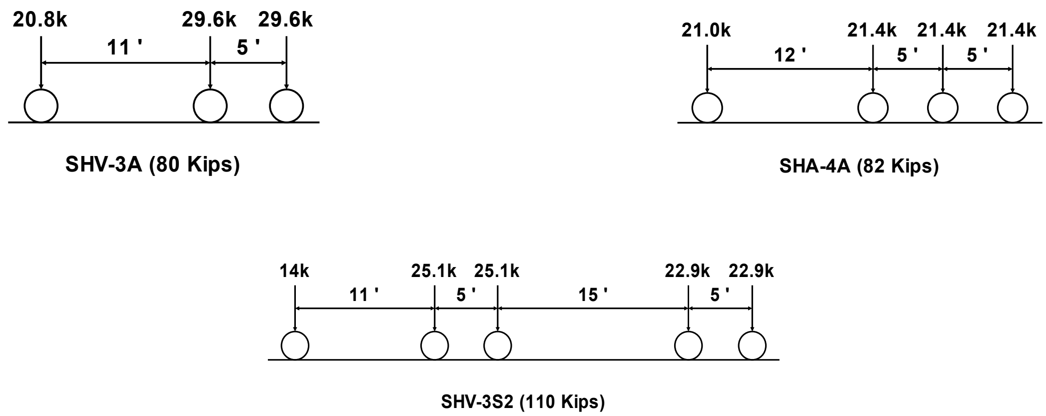
(continued)



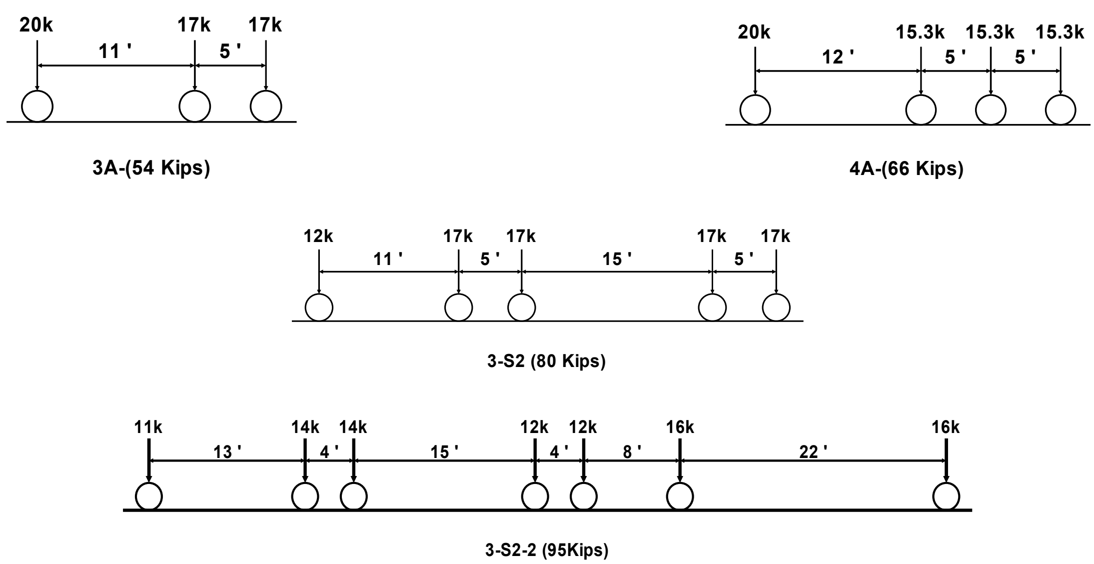
(b)

FIGURE 2 (*continued*) **(*b*) Exclusion vehicles.**

(continued)



(c)



(d)

FIGURE 2 (*continued*) **(*c*) NTWAC special hauling vehicles, and (*d*) modified TTI formula vehicles.**

(continued)

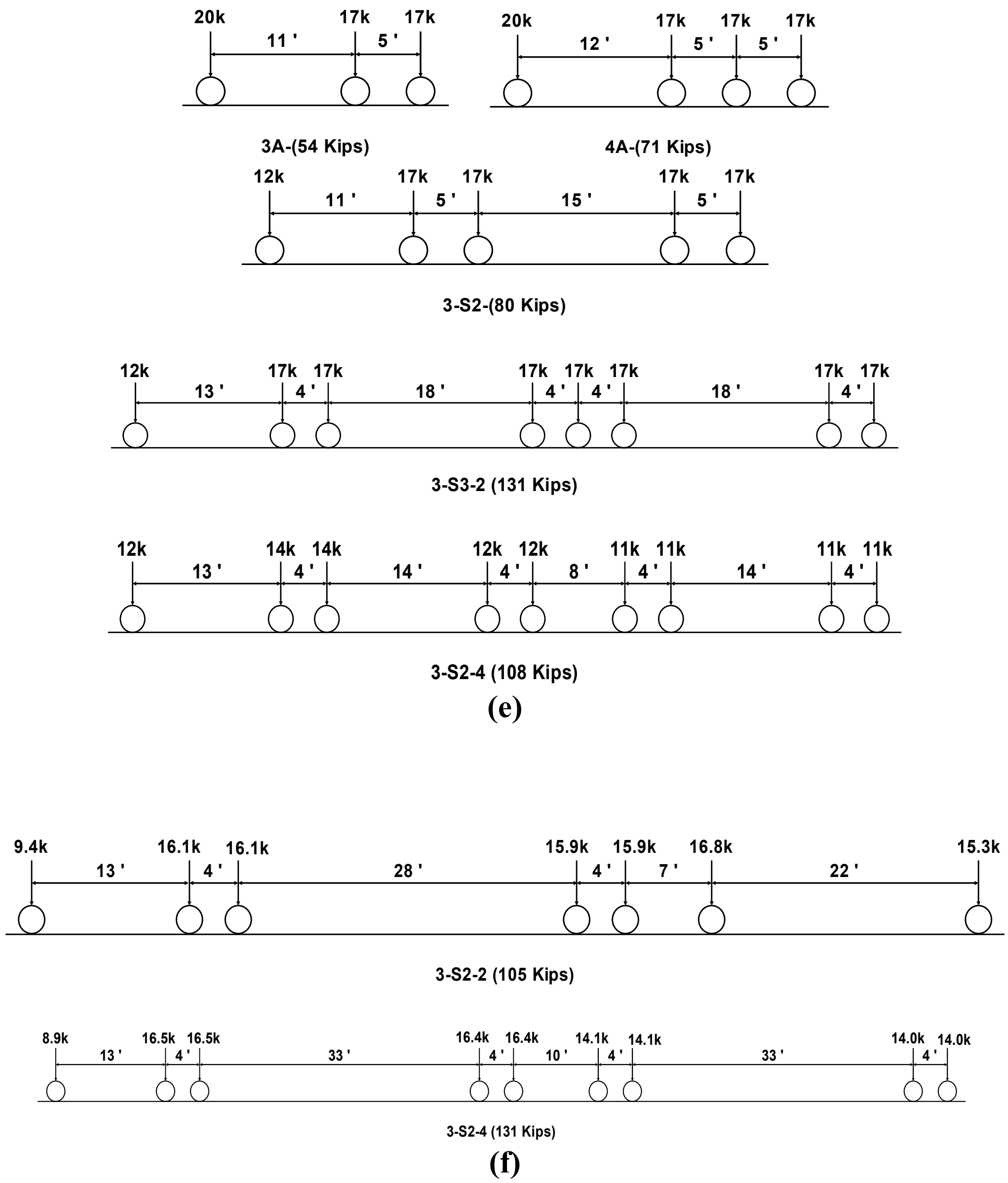
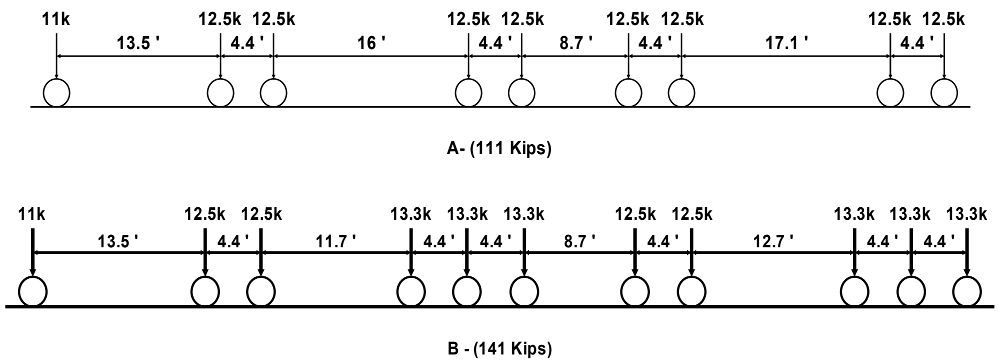


FIGURE 2 (*continued*) **(*e*) Canadian interprovincial load vehicles, and (*f*) extended bridge formula vehicles.**

(continued)



(g)

FIGURE 2 (*continued*) **(*g*) Turner trucks.**

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.

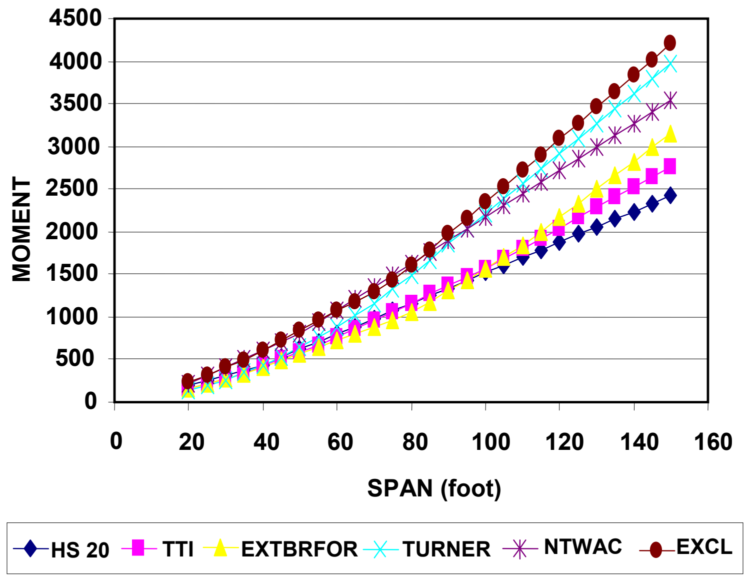


FIGURE 3 Centerline moments in kip-ft: simple span.

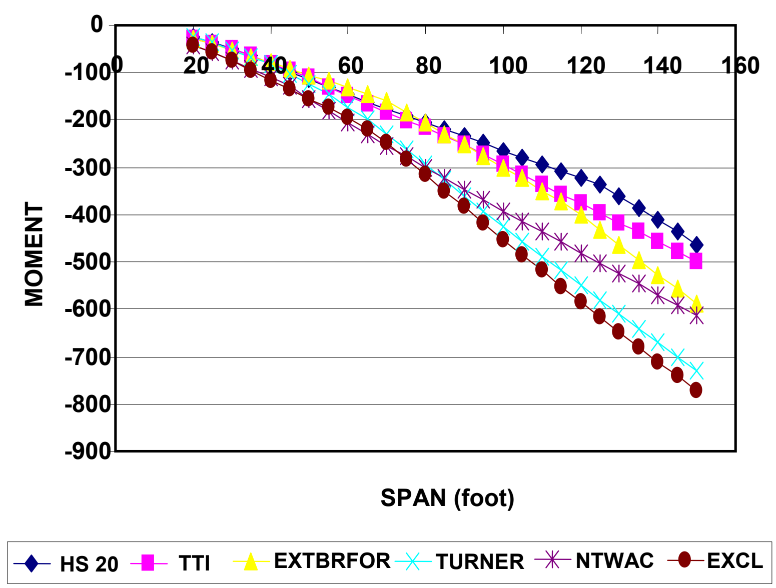


FIGURE 4 Negative moments in kip-ft at 0.4L.

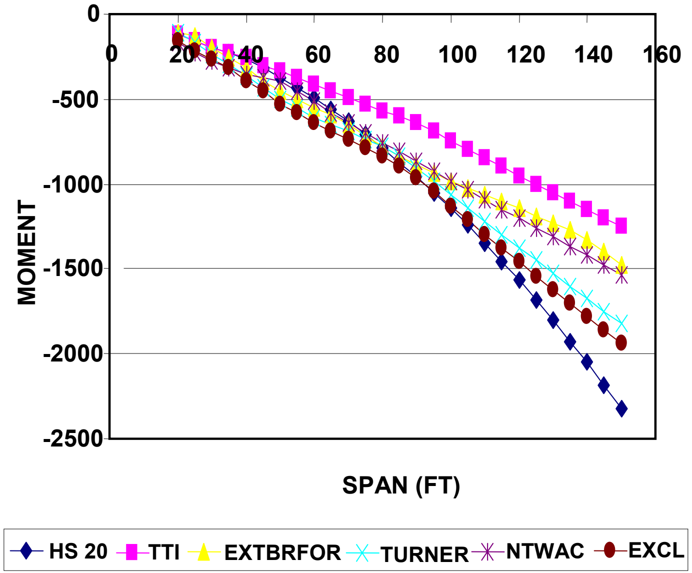


FIGURE 5 Negative moments in kip-ft at support.

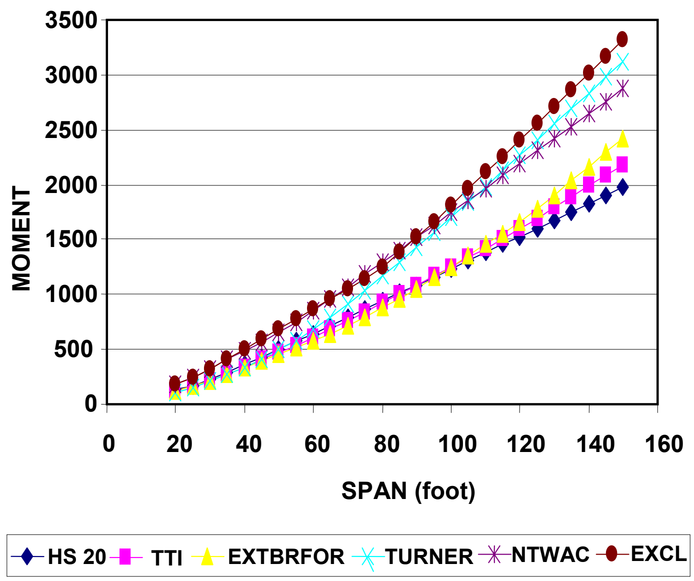


FIGURE 6 Positive moments in kip-ft at 0.4L.

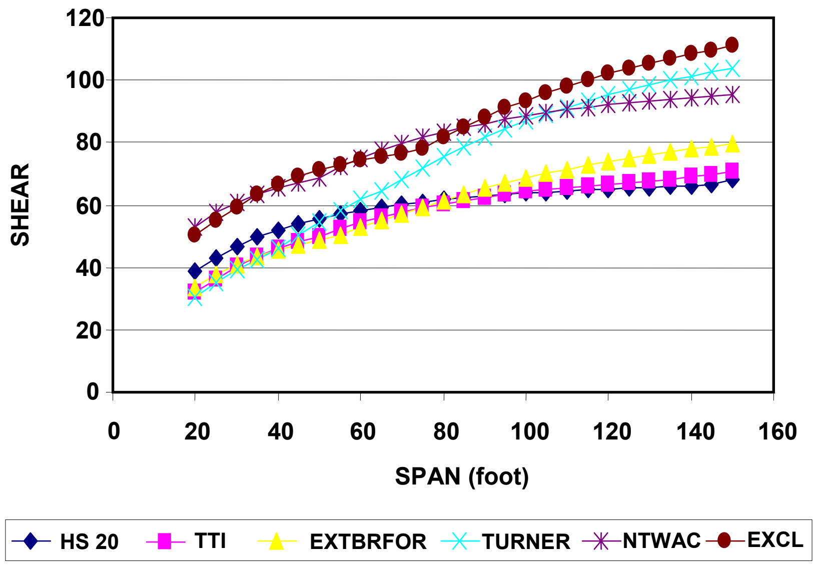


FIGURE 7 Positive shear in kips at $+V_{ab}$.

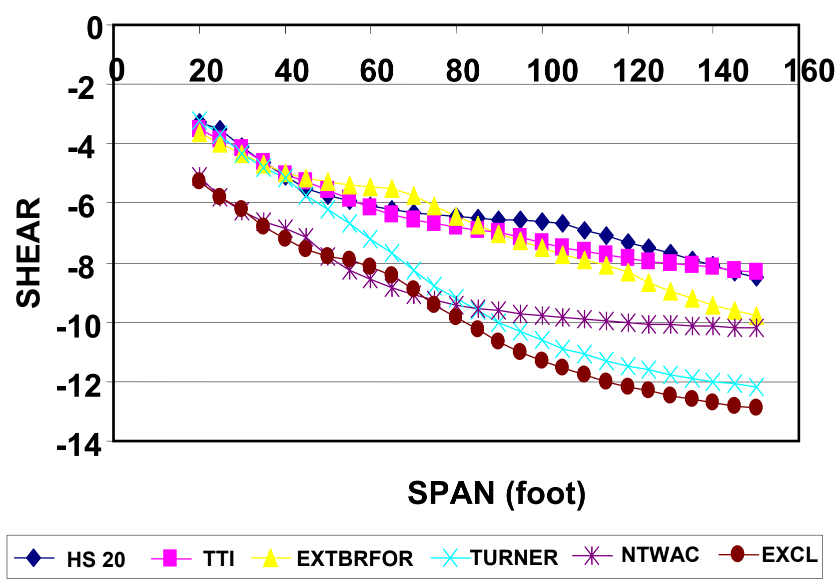


FIGURE 8 Negative shear in kips at $-V_{ab}$.

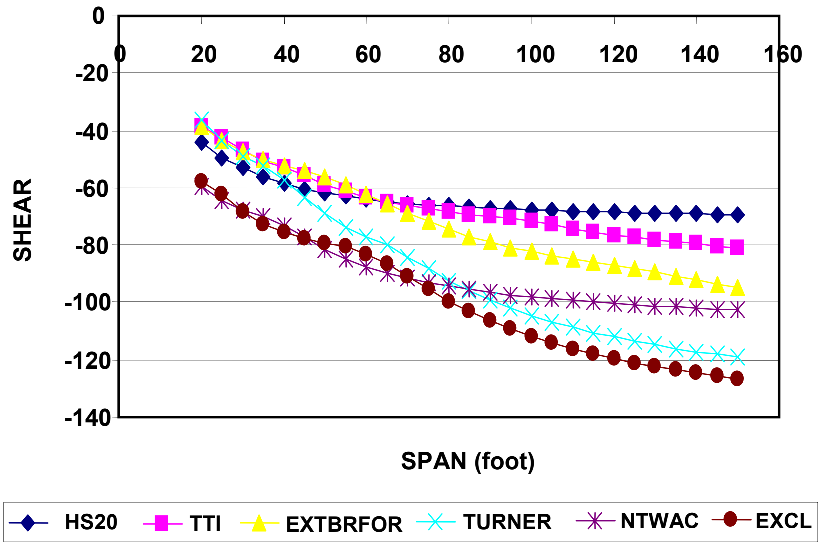


FIGURE 9 Negative shear in kips at $-V_{ba}$.

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.

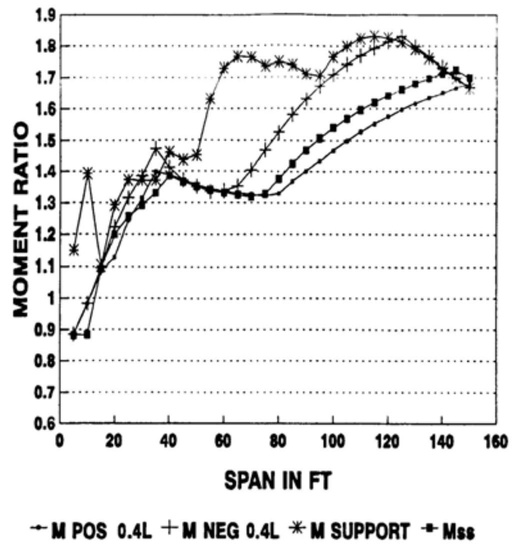


FIGURE 10 EXCL/HS20 truck or lane or two 24-kip axles at 4.0 ft.

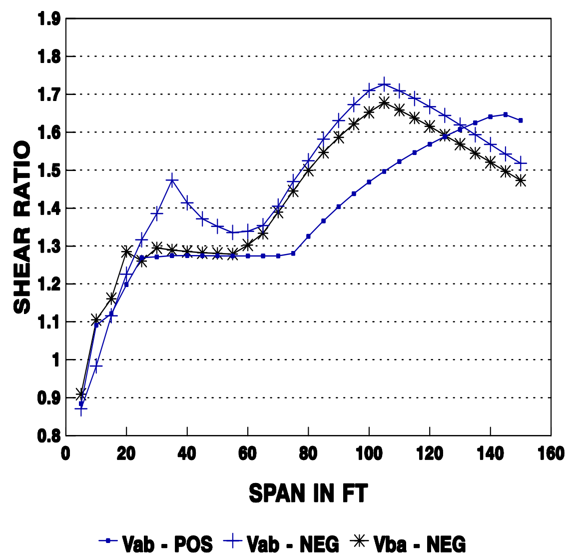


FIGURE 11 EXCL/HS20 truck or lane or two 24-kip axles at 4.0 ft.

- 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*).

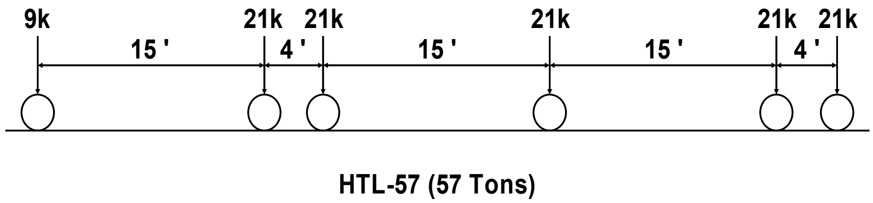


FIGURE 12 HTL-57 (114 kips).

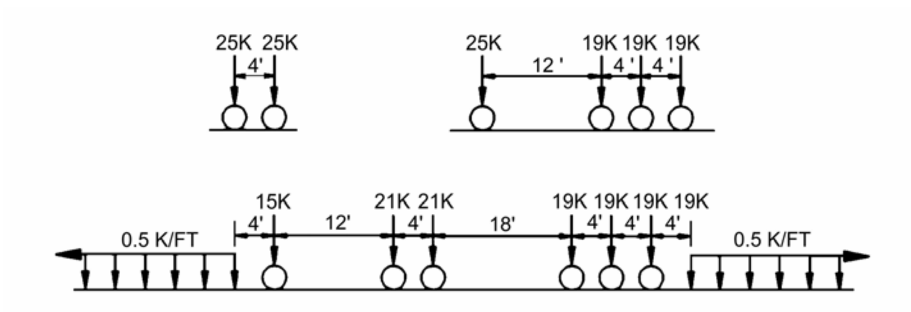


FIGURE 13 Family of three loads.

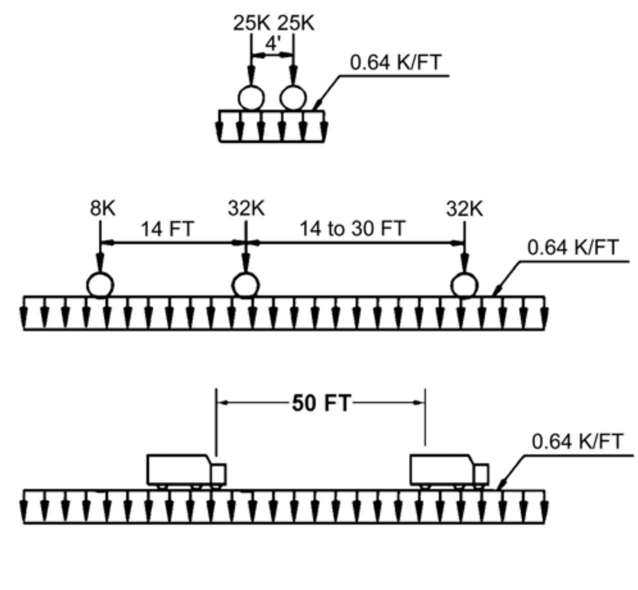


FIGURE 14 HL93 design load.

- 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.

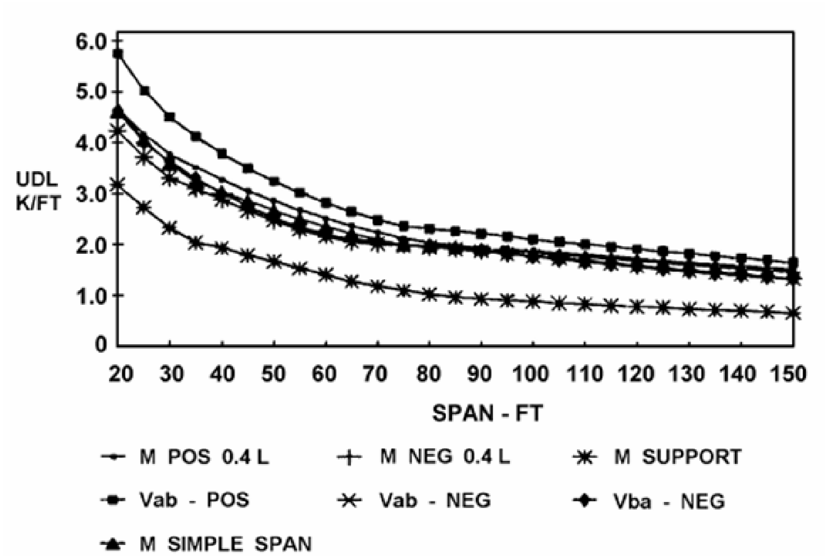


FIGURE 15 Equivalent uniform load.

TABLE 2 Live Load Models Versus Exclusion Load Mean and Standard Deviation

		HS20	HS20+0.64 (Prop.)	HS25+0.48	HTL	FAMILY-3	HTL MOD LF
–M 0.4L	Mean	1.600	1.060	0.982	1.189	1.048	1.091
	Std Dev	0.1679	0.0630	0.0755	0.1290	0.0379	0.0389
–M @SUPT	Mean	1.111	0.847	0.835	0.952	0.866	0.871
	Std Dev	0.2068	0.1201	0.0712	0.1427	0.0917	0.0575
+M 0.4L	Mean	1.459	1.018	0.924	1.200	1.041	1.103
	Std Dev	0.1186	0.0618	0.0409	0.1133	0.0475	0.0464
SIM SUP	Mean	1.506	1.001	0.941	1.198	1.050	1.100
	Std Dev	0.1387	0.0275	0.0586	0.1325	0.0363	0.0305
$-V_{ab}$	Mean	1.544	1.060	0.982	1.189	1.048	1.092
	Std Dev	0.1253	0.0929	0.0755	0.1290	0.0379	0.0389
$-V_{ba}$	Mean	1.461	1.011	0.932	1.111	1.006	1.022
	Std Dev	0.1448	0.0355	0.0475	0.1093	0.0522	0.0540
$+V_{ab}$	Mean	1.415	1.024	0.919	1.132	1.017	1.232
	Std Dev	0.1447	0.0391	0.0391	0.1027	0.0558	0.2792

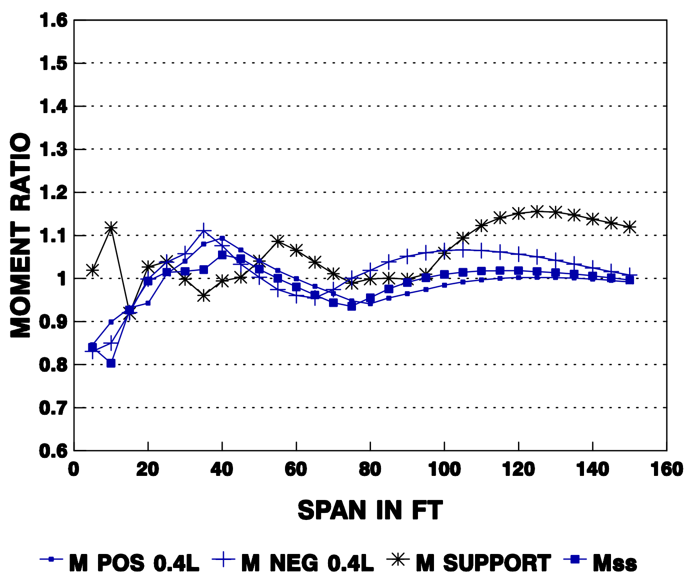


FIGURE 16 EXCL/HS20+0.64 kips/ft or dual 25 kip moment ratio.

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 AASAHTO 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*).

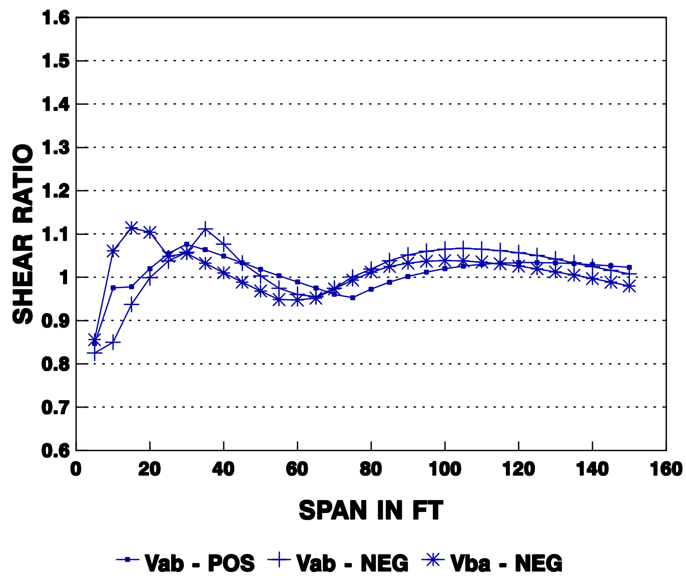


FIGURE 17 EXCL/HS20+0.64 kips/ft or dual 25 kip shear ratio.

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.

$$g = 0.075 + \left(\frac{S}{9.5}\right)^{0.6} \left(\frac{S}{L}\right)^{0.2} \left(\frac{K_g}{12\,Lt_s^3}\right)^{0.1} \tag{3}$$

where, subject to some numerical limits:

S = girder spacing (ft);
L = span length (ft);
K_g = $n\,(I+Ae_g^2)$ (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.

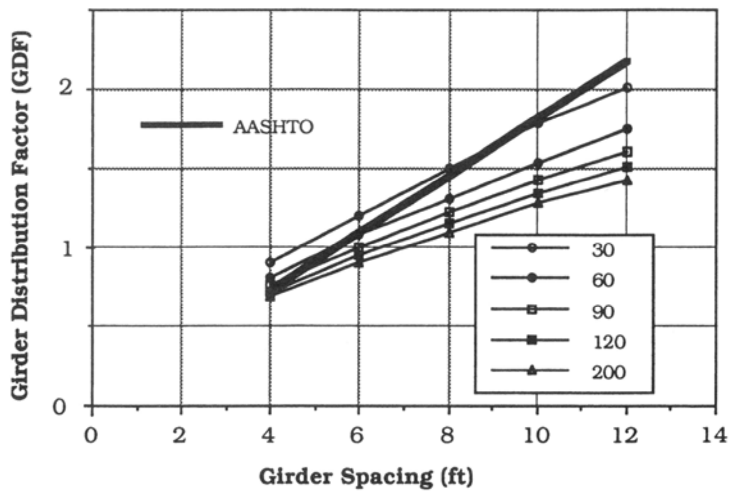


FIGURE 18 Initial (1991) comparison of *S*/*D* and LRFD distribution factor (*12*).

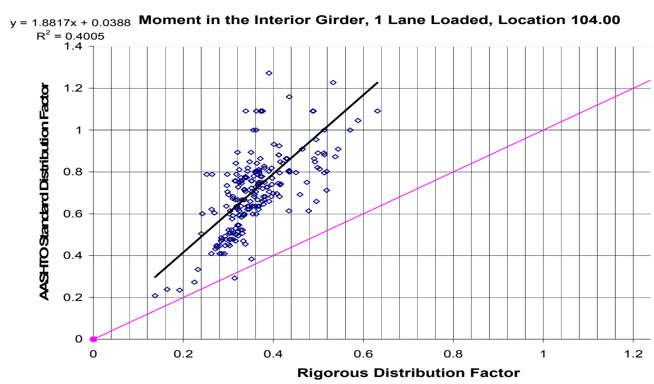


FIGURE 19 Comparison of *S*/*D* to grid analysis distribution.

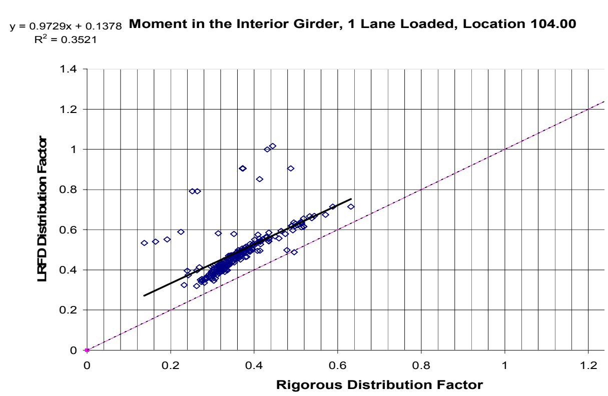


FIGURE 20 Comparison of LRFD factor to grid analysis.

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.

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STEEL BRIDGE COMMITTEE

Steel Bridges in the United States
Past, Present, and Future

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The 20th century represented a dramatic evolution in steel bridge technology. The first half of that century featured some of the most spectacular bridges of our time, as many of the major physical obstacles to transporting people and goods were spanned.

The second half of that century featured another dramatic evolution of our highway system as the population expanded and the automobile became a way of life. In 1956, Congress passed legislation that gave birth to our country's Interstate highway system (IHS). Thus began the building of an expansive highway network and its many bridges, linking primary urban centers.

Through this period, evolution in design codes, bridge types, materials and construction methods was amazing. This paper chronicles some of the highlights in this evolution in the history of steel bridges and concludes with a brief look forward.

INTRODUCTION

It is impossible to separate the history of steel bridges from history in general. The evolution of the steel span has been shaped and guided by wars, depressions, and catastrophes, as well as by periods of prosperity, industrial innovation, and federal transportation programs. Along the way, engineers responded to economic and political circumstances with creativity and ingenuity as they developed and improved upon steel bridge design and construction. Their brilliance ensured that steel bridges would hold an important position in our nation's history, both technically and aesthetically.

THE LATE 1800s AND EARLY 1900s

The so-called Gilded Age enjoyed by upper class Americans began in 1878, and had ended by 1890, when gold reserves dropped dramatically and numerous railroads failed. These events triggered the panic of 1893, followed by the second worst depression in our country's history. By 1896, economic recovery was underway and the nation entered a period of prosperity that continued into World War I and produced a surge in private and public construction.

Evolution of Materials: Iron Gives Way to Steel

Throughout most of the 19th century, cast iron and wrought iron were used for bridge construction. But near the end of that century, steel bridge design and construction became a necessity to meet the growing demand for railway bridges. In 1898, for example, the country's 375 railway lines added 3,000 mi of new track, bringing the total to 200,000 mi. Many large bridges were built by the railway companies, and people and goods were moving around the country as never before.

During the first two decades of the 20th century, A7 steel, higher-strength silicon steel (ASTM A94), and nickel steel (ASTM A8) were developed. It was the A7 steel, however, that remained the workhorse well into the 1950s. The centuries-old technology of hot-driven steel rivets was used to join these steels. The American Society for Testing Materials, formed in 1901, issued the first specification for the properties of carbon steel that later was designated A7 steel. That organization changed its name in 1961 to the American Society for Testing and Materials (ASTM).

Evolution of Bridge Design: Trouble with Trusses Leads to Arches

Earlier in the 1800s, truss bridges were constructed of wood or iron. The truss framework was then used for bridges made of steel later in the century, but often the new steel tension members failed in brittle fracture. Through the 1870s, the advent of open-hearth steel production superseded the Bessemer method, and the ductility and toughness of steel improved.

Those early years of steel bridge building contained an element of trial and error—leading, of course, to design improvements. Some highlights of this period:

- The Eads Bridge, crossing the Mississippi River at St. Louis, Missouri, opened for public use on July 4, 1874. The bridge, built by engineer James B. Eads, is notable because it replicated the arches that had been featured for years in masonry bridges, but for the first time, those arches were created from steel. When complete, the bridge had three arches of 502, 520, and 502 ft in length. At the time, many engineers thought that steel arches this long were impossible. Also, many material tests and testing equipment were initially developed specifically for the steel used on this bridge.
- The Brooklyn Bridge, started in 1869 and completed in 1883, represents another of the first steel bridges in the United States. It was designed by John Roebling, and completed (following his death) by his son, Washington Roebling. When opened, it was the longest suspension bridge in the world, and the first with cables formed from parallel steel wires spun in place.

- The country's largest construction project at the turn of the century was the 28-mi Chicago Sanitary and Ship Canal, which reversed the flow of the Chicago River from Lake Michigan to the Des Plaines and Mississippi Rivers. Contracts were awarded in 1897 for construction of 13 moveable bridges over the canal—six for railroads and seven for highways—plus additional bridges over the Des Plaines River.

  That same year, 1897, the nation's first all-steel suspension bridges were completed. One, designed by E. K. Morse, spanned the Ohio River at Rochester and Monaca, Pennsylvania, with an 800-ft center span. The second, designed by Hermann Laub 20 mi downstream at East Liverpool, Ohio, the Lincoln Highway Bridge was constructed with a 705-ft main span.
- Work also began on the world's longest suspension bridge, the Williamsburg Bridge over the East River, between Manhattan and Brooklyn, New York. It opened in 1903 with a 1,600-ft main span. The designer, Leffert L. Buck, specialized in steel arch bridges. That same year, his Clifton Bridge over the Niagara River was completed. Though John Augustus Roebling built a railroad suspension bridge over the Niagara River that opened in 1855, Buck's Clifton Bridge, with its 550-ft main span, was the first arch to span the Niagara.
- In 1907 during the erection phase of the Quebec bridge, the suspended central span, which was erected off-site and floated in, slipped off the end supports while being lifted into place, sinking in 200 ft of water. The cantilevered trusses did not sustain any damage. A second suspended span was fabricated and lifted successfully.
- The Hell Gate Railroad Arch (Figure 1), opened in 1917 and designed by engineer Gustav Lindenthal, is noted as an example of the early use of the arch form in steel. It spanned 977 ft and supported a load of 75,000 lbs/ft, the heaviest load intensity in the United States at the time. The bridge spans the East River and connects the Bronx to Queens in New York.

THE NEXT 20 YEARS: 1920 TO 1940

The post-World War I recession faded into the "Roaring 20s" by the year 1922. People began to spend money. Nationwide, coal and railroad strikes slowed the economy a bit, but a drop in the cost of materials and labor and a thriving stock market stimulated construction. In 1922 the volume of completed construction was $4.9 billion—a 41% increase from the previous year.

Evolution of the Suspension Bridge

During the era of the 1920s and 1930s, suspension bridges with much longer spans became the norm. As engineers became more familiar and comfortable with steel, they applied lessons learned during the previous 40 years, and steel bridge spans soared.

In 1924, a suspension bridge opened with a 1,632-ft main span crossing the Hudson River near Peekskill, New York. Called the Bear Mountain Bridge, it had two 355-ft steel towers, and was designed by Howard C. Baird. Plans were also announced for another bridge across the Hudson, at Poughkeepsie. That suspension bridge, designed by Ralph Modjeski and Daniel E. Moran, had a 1,500-ft main span. It was opened in 1932.

FIGURE 1 The Hell Gate Bridge spanning New York's East River opened in 1916. (Photo: HAER, en.structurae.de.)

Construction began on Ralph Modjeski's first suspension bridge project in 19220151the Benjamin Franklin Bridge over the Delaware River, linking Philadelphia and Camden, New Jersey. The bridge was completed in 1926, and its 1,750-ft main span made it the world's longest suspension bridge until 1929 with the opening of the 1,850-ft main span Ambassador Bridge over the Detroit River at Detroit designed by McClintic.

In the 1930s, the long span record was broken two more times with the 3,500-ft George Washington Bridge crossing the Hudson River at New York (Figure 2), designed by O. H. Ammann, and the 4,200-ft Golden Gate Bridge designed by Charles Ellis and Joseph P. Strauss .

Evolution of Arch and Truss Bridges

During the early 20th century, bridges usually featured the excessive ornamentation associated with the previous century. By the 1930s spans had increased and the Victorian era receded further. Bridges began to shed their ornamentation and began to reflect their true function and power. Some highlights of this period:

- In 1927, the Carquinez Strait Bridge crossing an arm of the San Francisco Bay opened to traffic on the very day that Lindberg flew to Paris. Engineer David B. Steinman proved that long span steel trusses could again be successfully constructed. It is of note that

FIGURE 3 The George Washington Bridge, crossing the Hudson River in New York, opened to traffic in 1931. The lower level opened in 1962. In 1981, the bridge was designated a National Historic Civil Engineering Landmark by the ASCE.

another bridge was built parallel to the original one, completed in 1958. The two bridges are jointly referred to as the Carquinez Bridge (Figure 3).

- Now there is another Carquinez Bridge. The new Carquinez is the first suspension bridge to be completed in the United States since 1964 (see below). It features a true orthotropic battle deck superstructure, the first of its kind in the United States.
- The first Carquinez Bridge was followed in 1930 by what was then the longest steel truss span in the United States at 1,200 ft. One of the most beautiful trusses in the country, it

FIGURE 3 The two bridges linking the San Francisco Bay Area and the Sacramento Valley are jointly referred to as the Carquinez Bridge. The three-lane southbound bridge opened in 1927. The northbound bridge, now with four lanes, opened in 1958 and is 200 ft east of the original bridge. In 2003, a new bridge was built just west of the 1927 bridge, replacing the aging span that did not meet standards for seismic design or traffic safety. (Photo courtesy of the California Department of Transportation.)

spans the Columbia River at Longview, Washington. The engineer was Charles Ellis, one of the first designers to emphasize functionality.

- In 1931, the Bayonne Bridge opened, connecting New Jersey to New York by way of Staten Island. It was considered an achievement in its expression of steel arch functionality, and it featured the longest arch span in the world, at the time—1,652 ft. Engineer O. H. Amman used high-strength manganese alloy steel to create the arch rib.
- Engineer Conde B. McCullough designed two steel bridges that opened in 1936 in Oregon: the Yaquina Bay Bridge, a steel arch span in Newport, and Coos Bay Bridge, a steel truss span in North Bend. The chords of the 600-ft truss-arch ribs of the Yaquina Bay structure are thin at the center and thicken slightly toward the supporting piers. This slight deepening of the ribs presents an intuitive feeling of the gradually increasing thrust force in the ribs near the supports and makes this bridge an outstanding example of an aesthetic bridge by an engineer in complete control of materials, stresses, and strains. With the 739-ft cantilever through-truss main span of the Coos Bay, McCullough effectively employed an unusual curved top and bottom chords, matching the concrete arch approach spans.

Evolution of the AASHTO Specification

Highway bridge design grew from the railway industry. Railway bridge engineers understood that it was necessary to limit deflections in order to ensure the safe performance of bridges under railway loads. Hence they limited the working stress, span-to-depth ratios, and computed deflection to levels that were deemed to be functional and safe. Compared with serviceability, strength was considered a secondary concern.

The development of steels with improved weldability in the 1960s and the increased competition of other materials led to the impetus for strength as well as serviceability to control designs.

The load factor design philosophy was developed to this end. The load factor specification assigned different load factors to different load types. .

Ultimately, the load and resistance factor design (LRFD) approach evolved based on probability and reliability. LRFD combines the probability of a given load and the variability in the strength of materials to provide a more uniform level of strength and serviceability (reliability) for all spans and bridge types.

Fundamentals of Live Load

The most commonly employed bridge live loading during the 19th and early 20th centuries in America was that presented by John Alexander Low Waddell.

Live loads were applied as a uniform load over the entire bridge deck area. The category of load used depended on the location of the bridge and the expected frequency of heavy vehicular traffic. Figure 4 depicts Waddell's live load intensity curves. "A" applied to urban areas. "B" applied to areas around cities and to industrial areas. "C" applied to rural areas where lighter loads were expected. Allowable stresses also varied by category. Design stresses for "A" were lowest. "B" and "C" were grouped with respect to stress and designed to a higher allowable stress.

AASHO 1925

The earliest known AASHO bridge design provisions were issued in 1925 with no edition number. Division V Section 4 of this typewritten edition gave the loads for which highway bridges were to be designed. There were four classes of highway loading: H20, H15, H12.5, and H10. The number designated the truck weight in tons. There also was an electric rail loading. There were five classes of bridges: AA, A, B, C, and D. Each class of bridge was loaded with successively lighter live load except Class D, which was for bridges carrying electric trains.

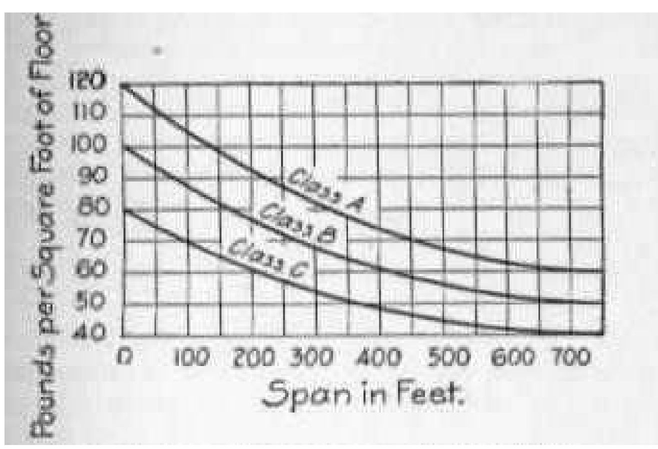

FIGURE 4 John Alexander Low Waddell's live load intensity.

First Official Edition—AASHO 1931

The first edition of the AASHO Bridge Specifications is dated 1931. The first edition was not available to the writer. In the second edition AASHO Specification, dated 1935, Article 5.2.8 specified three different live loads, H20, H15, and H10, for three classes of highways: AA, A, and B. The "H" indicated highway loading. The trucks were unchanged from the 1925 edition. Live load was specified as either a truck train or a 600-lb/ft lane load with one 28 kips concentrated load placed at the critical location.

The truck train consisted of the design vehicle centered between additional vehicles heading in the same direction over the remainder of the span, each weighing 75% of the design vehicle, heading in the same direction. The uniform load was increased to 640 lbs/ft with a single concentrated load reduced to 18 kips for moment; the concentrated load for shear was 26 kips. Spans less than 60 ft were required to be loaded by the truck train. Spans above 60 ft could be loaded with either the lane load or the truck train.

THE 1940s AND 1950s

Evolution of Fabrication and Detailing

The production of Liberty ships during World War II resulted in rapid advances in steel welding and the introduction of the automatic submerged arc welding process—knowledge that had an important impact on the evolution of steel bridges. The new welding method was ten times faster than hand arc welding and had fewer defects. Researchers found that hydrogen introduced into the weld passes during the welding process caused embrittlement of the weld. This discovery led to the development of low-hydrogen electrodes and the low-hydrogen welding process.

Some of the earlier Liberty ships broke apart, without apparent reason. Subsequently it was understood that the large hatches in the even larger decks of the Liberty ships acted as reentrant corners and stress risers that led to the sudden and catastrophic brittle failures of many of these vessels. Ultimately, the cause was found to be low toughness in the steels used, as measured by the Charpy Impact Method (ASTM E23), and sharp reentrant corners on a much smaller scale, not the arc welding. These events focused attention on using tougher steels and improved detailing.

Evolution of the AASHTO Specification Continues

Third Edition—AASHO 1941

The H-S type loading was introduced in the third edition to recognize the new semi-trailer trucks. The heaviest semi-trailer loading was specified as H20-S16. The first number gave the weight of the tractor in tons. The second number gave the weight of the semi-trailer in tons. The spacing of both axles was fixed at 14 ft. The semi-trailer loading was applied by itself. The uniform load for H20-S16 of 640 pounds per foot, was maintained, but the concentrated load was increased to 32 kips for moment and to 40 kips for shear. Hence, this lane load represented an H20-S16 truck surrounded by H15 trucks.

This edition also introduced an overload provision. The overload applied to the entire structure except the deck. The purpose of the overload was to account for "infrequent heavy loads." Thus, it applied to strength and not service considerations.

The design of continuous spans is first mentioned in this edition. The uniform load portion of the lane load was placed discontinuously to cause maximum stress. However, only one concentrated load was applied.

Fourth Edition—AASHO 1944

In this edition, the designation of the H-S loads was changed to the following style: HXX-SZZ-44 (the "44" designated the year of adoption). The distance between the rear axles of the H-S trucks was permitted to vary from 14 to 30 ft to represent the new longer semi-trailers. The truck-axle length remained six feet. Design moments were specified to be the larger of a single truck or the lane load.

Post–World War II Long-Span Developments

After the war ended, bridge construction was in high demand to meet the needs of the military, the surge in travel, industrial production, real- time delivery, and the corresponding demand for long distance domestic shipping that is now the basis of the trucking industry world-wide. This heightened activity resulted in the building of many notable bridges.

In 1948, bridge construction increased 50% over the previous year. New projects that year included the Mystic River Bridge, a 1,524-ft cantilever bridge with an 800-ft center span; the Delaware Memorial Bridge near Wilmington, with a 2,150-ft suspension span; the East St. Louis Bridge over the Mississippi, 7,620-ft long with a 963-ft cantilever span. A replacement for the Tacoma Narrows Bridge in Washington opened in 1950 (Figure 5). The original 2,800-ft suspension span at Tacoma Narrows failed in a wind storm in 1940.

FIGURE 5 Tacoma Narrows Bridge.

After the war, the fastener of choice remained the labor-intense hot driven rivet, since A7 steel could not be reliably welded. Bridge engineers were eager to use the welding methods developed during the war, and in 1946 the steel industry responded with weldable low-alloy steel, ASTM A242. This new steel had a minimum specified yield point as high as 50,000 psi, depending on thickness which raised the working stress-level from 18,000 psi for A7 to between 22,000 psi and 27,000 psi depending on thickness.

High strength bolts emerged around the year 1948, easily replacing the existing mild steel ASTM A307 bolt in most applications. This new type of bolt clamped steel plates together and prevented joint slippage with friction. The friction feature caused by the high tension force in the bolt also prevented the nut from loosening. The high strength bolts received the ASTM designation, A325. Later, a higher strength bolt ASTM A490, was developed. The turn-of-the-nut method and direct-tension-indicator washers provided a means for reliably controlling the tension in the bolt.

In the late 1950s, A7 steel was replaced by ASTM A36 and ASTM A441 which had a higher yield point and was very weldable. Shortly thereafter several proprietary steels were developed that led to minimum specified yield stresses between 42 and 60 ksi. Quenched and tempered steels with minimum specified yield stresses as high as 110 ksi received the ASTM designation A514. A modified copper-bearing version of ASTM A441 steel had weathering enhanced properties. It was the precursor of ASTM A588.

AASHO Road Test

Planning for the AASHO Road Test, which was initially a large scale pavement study, got underway in 1951. This important project is frequently referred to as the Ottawa Test, because the testing took place in Ottawa, Illinois, on the future alignment of Interstate 80. This test provided fundamental data for the development and calibration of AASHO design provisions for strength, dynamic response, and fatigue characteristics using realistic bridges and vehicles.

The study included 16 test bridges—simple designs representative of highway bridges under construction at the time. Eight bridges were steel beams, four were pre-stressed concrete beams, and four were reinforced concrete T-beam construction. Testing took place from August 1958 to December 1960.

Steel beams varied from bridge to bridge in the size of the cross section, the presence or absence of partial length cover plates on the bottom flange and the presence or absence of composite interaction with the slab. The structural steel was required to have physical properties as close to the minimum specified values as possible. To preclude bonding with the slab, the top surfaces of the steel beams of the non-composite bridges were coated with a mixture of graphite and linseed oil. Channel shear connectors were welded to the top flanges of composite bridges. On the replacement bridges a partial length cover plate was welded to both the bottom and the top flanges of the steel beams.

Dynamic tests of the test bridges, conducted in cooperation with the University of Illinois, started in the fall of 1958 and were concluded in October 1960. After the completion of the regular test traffic, two other tests were initiated: accelerated fatigue tests, followed by tests to gauge failure with increasing loads. The purpose of the first was to increase the number of cycles of maximum stress beyond that accumulated during the period of regular test traffic. The tests with increasing loads were conducted to determine the largest loads that could cross a bridge and to determine the mode of failure. Testing was completed in June of 1961.

After the conclusion of the regular test traffic, seven of the 13 surviving test bridges were subjected to accelerated fatigue tests. The bridges were excited with a mechanical oscillator to such amplitude that the maximum stress and the range of the fluctuating stress at the critical section approximated the average condition observed during the test traffic. The vibration continued until failure or until the total number of stress cycles reached 1,500,000. This total included the cycles accumulated during the test traffic, counting each trip of a regular test vehicle as one stress cycle.

Dynamic amplification of vehicle loads on bridges, commonly called impact, had been a long-standing concern in bridge design. As a result of the accelerated fatigue tests, a simple formula relating the impact factor to the length of the bridge evolved. But the major accomplishment of the dynamic tests on the AASHO Road Test bridges was the successful calibration of the analytical model of realistic bridges and vehicles.

This kind of test and the resulting specification have never been duplicated elsewhere in the world at this scale or effort.

Birth of the Interstate Highway System

On June 29, 1956, President Dwight D. Eisenhower signed into law the Federal-Aid Highway Act, and a 41,000-mi national IHS was born. Construction on the IHS began in earnest and continued unabated for the next 16 years.

Trusses and Arches Dominate

Two similar long-span cantilevered truss bridges opened in 1955 and 1956—the Tappan Zee, across the Hudson River in New York, and the Richmond–San Rafael, across San Francisco Bay. Each was independently financed and had funding problems, resulting in zigzag alignments and awkward towers. The 15,300-ft Tappan Zee has a main span of 1,112 ft, and the 21,340-ft Richmond–San Rafael has a main span of 1,070 ft. Other important steel truss and arch bridges include the following.

- The Kingston–Rhinecliff Bridge, crossing the Hudson River in New York, opened in 1957. It was the first bridge to utilize a series of continuous trusses totaling 5,200 ft in length. The designers were Robinson and Steinman.
- In 1958, the Greater New Orleans Bridge spanning the Mississippi River was completed, featuring a 1,575-ft cantilevered truss main span—a record at the time. The firm Modjeski and Masters was the designer. These truss and arch bridges were nearly the last of the long spans to use hot-driven rivets as the lower cost and more versatile welding and high-strength bolting technologies forced their demise.
- In 1959 the Bureau of Reclamation constructed a true deck arch over the Colorado River in Glen Canyon, Arizona, near the well-known dam of the same name. The arch ribs are steel trusses, supported by concrete skewbacks embedded in the canyon walls. The arch spans 1,028 ft and the deck is 700 ft above the river.

1960 TO 1980

Entering the 1960s, bridge construction boomed as the full impact of the new federal highway system took hold. More weldable and higher strength steels continued developing.

During these decades, engineers often departed from traditional construction methods, and their new ideas were on the mark. Concrete was introduced in combination with steel for fully composite designs. The use of prefabricated or preassembled parallel wire strand for suspension bridges caught the attention of designers. Bridge construction reached a record level in 1972, when the biggest, longest, highest, and most expensive spans were underway.

Family of Steel Bridges Expands: Steel Tied Arches, Trusses Joined by Cable-Stayed and Girder Designs

New bridge designs came on the scene during the 1960s and 1970s. Steel tied arches surpassed trusses in popularity because they were more aesthetically appealing and contractors were developing economical erection methods for tied arches. During that same time period, steel box girder bridges were introduced as a viable alternative to conventional plate girders. Important research into the design and behavior of box girder bridges demonstrated the inherent economy and performance of a multibox torsional section for bridges on curved and tangent alignments.

Early box girder examples in the United States were designed by the Massachusetts and New York State Highway Departments. Maryland followed years later, with some outstanding examples in the Baltimore area. Several of these early boxes were true four-plate boxes, or partially closed over the piers. They were also curved, in recognition of the superior performance of a St. Venant's section under torsional loads. (It should be noted that this characteristic of a closed section is applicable to tangent girder alignments, and has direct benefits in load distribution, bridge performance and economy.)

The Coronado Bay Bridge in San Diego (Figure 6), which opened to traffic in 1969, is arguably the single most important example of a steel box girder bridge. The 2.12-mi bridge has a vertical clearance of 200 ft or so, and it features beautiful curves and impressive towers. The braces and stiffeners for the bridge are contained within a box girder, ensuring a smooth exterior.

Other notable developments in the design and construction of box girders occurred in Louisiana, Pennsylvania, Florida, and Colorado, where integral bents were utilized in several instances, leading to the development of important details.

By the 1980s the steel box girder had become a common sight on Interstates in many locations. Unfortunately, the failure of several major box girder structures in the United Kingdom, Germany, and New South Wales during this period had a significant cooling affect on the acceptance of steel box girders in the United States.

Many records were set during these 20 years, thanks to new and improved design, materials, and construction methods. Bridges of note built during these decades include:

- The Lewiston–Queenston deck arch near the mouth of the Niagara River, spanning between New York and Ontario, Canada, opened in 1962. At 1,000 ft, it was then the world's longest fixed or true arch. The arch ribs were large box sections, erected in segments, by travelers riding on the erected segments. The ribs were supported on temporary, inclined false

FIGURE 6 The Coronado Bay Bridge in San Diego opened to traffic in 1969, and is arguably the single most important example of a steel box girder bridge. (Photo courtesy of the California Department of Transportation.)

work, a break from the traditional method of using tieback cables to support the ribs. The engineer was Hardesty and Hanover.

- In 1962, a plate girder bridge was built across the Mississippi River between Wisconsin and Minnesota with a main span of 450 ft that tied the record at that time for the longest girder span.
- The California Division of Highways designed and constructed a true deck arch near Santa Barbara in Cold Springs Canyon in 1962. The arch ribs spanning 700 ft were welded rectangular box sections and the spandrel columns were square welded tubes supporting the concrete deck.
- In 1970, a pedestrian crossing in Wisconsin—the nation's first cable stayed bridge—was completed.
- In 1972, the second Governor William Preston Lane Jr. Bridge over Chesapeake Bay near Annapolis, Maryland, was constructed just 450 ft north of an almost identical bridge built in 1952. The centerpiece is the 2,914 ft suspension bridge with a 1,600-ft main span, which was built with preassembled, parallel wire cable strands.
- Two through tied arches, placed in tandem, were featured in a bridge designed by Hazelet and Erdal across the Mississippi River at Memphis, Tennessee, in 1972. Tandem arches are used occasionally to lengthen a structure for wide river crossings. Each through-tied arch was 900 ft long.

- That same year, the very first vehicular cable-stayed bridge in the United States was constructed in Sitka, Alaska, with a main span of 450 ft. It was designed by the Alaska Department of Highways.
- In 1973, the Fremont Bridge in Portland, Oregon, set a U.S. record for tied arch spans at 1,255 ft. The bridge is a through tied arch with the thin arch rib stiffened by a heavy tie girder. The tie girder supports a lower concrete deck and an upper orthotropic steel deck. The erection set a record with a 6,000 ton lift. This record span is likely to stand for all time in the United States since the more economical cable stayed bridges are now the standard. The engineer was Parsons Brinkerhoff.
- A new length record for a cantilevered truss span was established in 1974 with the opening of the Commodore Barry Bridge, crossing the Delaware River between Pennsylvania and New Jersey. The main span is 1,644 ft, with a total length of steel bridge of 13,912 ft. This span for a cantilevered truss is also likely to stand for all time in the United States because cable stayed bridges for this span length are far more economical.
- The span-length world record for arch bridges was broken in 1977 when the New River Gorge Bridge opened in West Virginia. The true deck arch spans 1,700 ft between skewbacks. The arch ribs were erected with an overhead cableway while the ribs were temporarily held by tieback cables. The bridge, designed by engineer Michael Baker, Jr., is the highest above water in this country at 876 ft.
- In 1978, the first major cable-stayed vehicular bridge in North America was completed. The Pasco–Kennewick Bridge across the Columbia River in Washington state boasted a 981-ft main span. Although it was a continuous concrete girder, it was supported by one of the first applications in the United States of prefabricated parallel wire strand, and other innovative features that were precursors of a new generation of cable-stayed bridge technology. In total, there have been only about 16 steel cable-stayed bridges constructed in the United States since 1972, with another 10 constructed in concrete.

AASHTO Evolves: Load Factor Design (1971 Interim Specifications)

Up to this point, bridges had been designed using the working stress design (WSD) method in which the dead and live load stresses are added together and the resulting sum is compared to an allowable stress determined by dividing a critical stress by the factor of safety. By this method, the entire structure is treated as a whole with the stress in its various components checked against a nominal stress. This is different from later "strength" methods.

Load Factor Design (LFD) is a strength method. It applies different factors to dead loads and live loads.

Bridge design continued to evolve. Curved and skewed bridges are now common. The original AASHTO Guide Specification for horizontally curved highway bridges was first officially published in 1980. This was the first curved girder bridge specification in the world. The primary contributors were Galambos and Culver, among others. The first major revision of the guide specification (LFD) appeared in 2003. The 2005 Interims to the AASHTO LRFD specifications included an integrated design approach to curved girders. In this specification, straight girders are a special case.

Evolution of Fatigue and Detailing Provisions

Prior to the Ninth Edition of the AASHTO Bridge Specification, welded bridges were checked for fatigue using the American Welding Society (AWS) specifications for bridges. Fatigue provisions had been addressed previously by limiting alternating stresses. These earlier provisions were generally adequate for riveted bridges, but were inadequate for welded bridges. The 9th edition, in 1965, introduced cycles of maximum stress combined with the modified Goodman diagram to limit the maximum fatigue stresses for nine specific details. These provisions were based on relatively small scale tests carried out at the University of Illinois and elsewhere in the United States and Europe.

Fatigue cracks were still being found in some beams with partial length cover plates with as little as 13 years of service. Fatigue cracks were also frequently found in webs at the ends of web stiffener welds which were stopped short of the beam flange. It appeared that the 1965 fatigue provisions were inadequate. New studies funded by NCHRP included variable-cycle loading, full-scale bridge attachments, inspection and repair of fatigue cracks, and large scale experiments. Additional testing was carried out worldwide between 1960 and 1980. The NCHRP studies were published in nine NCHRP reports, seven from Lehigh University and two from the U.S. Steel Corporation. The 1973 Interims featured new design requirements based on the recent research results and the use of stress range rather than maximum stress.

1980 TO 2000

Toughness Requirements Get Tougher

The brittle fracture collapse of the Silver Bridge in 1967 and several other brittle fracture collapses in the 1960s and 1970s including the Kings Bridge, Australia; the Fremont Bridge in Portland, Oregon; the Bryte Bend Bridge near Sacramento, California; and the Quinnipiac River Bridge in New Haven, Connecticut, led to more research of the brittle fracture phenomenon and ultimately to new AASHTO fracture-toughness requirements in the 12th edition in 1977.

In 1980, a 4-in. crack was found in the top flange of the tie girder on the Prairie du Chien tied arch bridge, built across the Mississippi River in Wisconsin in 1974. Analysis of the flange steel revealed that both flanges did not meet the toughness requirements specified for the bridge at the time of its design. This generated a lengthy discussion regarding toughness requirements for bridges and how many tests on steel plates were necessary to verify the required toughness.

As a result, the AASHTO Bridge Design Specifications were modified to require testing both ends of plates for toughness at specified temperatures.

Further, fatigue fractures were found to be caused by older weld details that developed high localized stresses or weld details that caused out-of-plane bending. This time period was a turning point in the evolution of steel bridges, as attention turned to these problems to determine how best to prevent their recurrence. AASHTO Bridge Design Specifications began to include categories of increasing susceptibility to fatigue resistance, as well as toughness values according to the temperature zone in which the bridge is located.

Advent of High Performance Steel: Materials Continue to Evolve

In the 1990s high performance steel (HPS) was introduced as a joint effort between the FHWA and the U.S. steel industry. This new steel offered many grades (strength levels) from low to high strength, extremely high toughness, very good ductility and enhanced corrosion resistance. After successful welding procedures were developed, the steel was ready for the bridge market. Currently, bridges constructed from HPS have economic advantages and have demonstrated ease of welding and fabrication. HPS appears to be the steel of choice for the 21st century, although there are currently various serviceability issues to be resolved. The economy and performance of girder bridges continues to evolve with longer spans, fewer joints and bearings, and enhanced durability with the advent of more advanced deck systems.

First Edition of AASHTO Load and Resistance Factor Design, 1993

This specification has adopted a limit states philosophy and addresses bridge performance based on the probability of the load effect and the resistance of a component. The probability concept is based on statistical uncertainty. It was first adopted by the American Institute of Steel Construction (AISC) for steel building design, and was given the moniker LRFD by Professor George Winter at Cornell University.

A bridge version of LRFD was developed under the NCHRP 12-33 Project and first adopted for the AASHTO LRFD 1993 Specification.

A new live load was required to fit the new probabilistic model and to include many of the heavier live loads accepted earlier by many states. Serendipitously, a live-load model consisting of the old HS20 truck superimposed on the H20 uniform load addressed many of the grandfathered trucks not covered by HS20 loading. This live-load model was designated HL93, with “93” designating 1993—the original year of introduction. So the old loads were essentially retained by combining them in a new way. To react to the continuous span moment dilemma addressed in the 1940s by the addition of a second concentrated load, the new live load was given an additional HS20 truck to be applied in another (critical) span.

Comparison of Working Stress, Load Factor, and LRFD

Figure 7 details simple-span moments for selected live loads and dead load versus span, and Figure 8 details pier lane moments for selected live loads and dead load vs. span. The relatively constant magnitude of live load moments in longer spans over the years would seem to indicate a belief by the specification writers that heavier trucks and denser truck traffic experienced in recent years do not result in moments larger than Waddell provided for in 1884.

Typically, the term “factor of safety” is not employed in the limit states philosophy. However, it provides a convenient means of comparing the capacity of the girders designed by various specifications and live loads. The “factor of safety” for WSD is simply 1/.55 = 1.82 and is constant for any ratio of LL to DL.

This “factor of safety” is derived for LFD as follows:

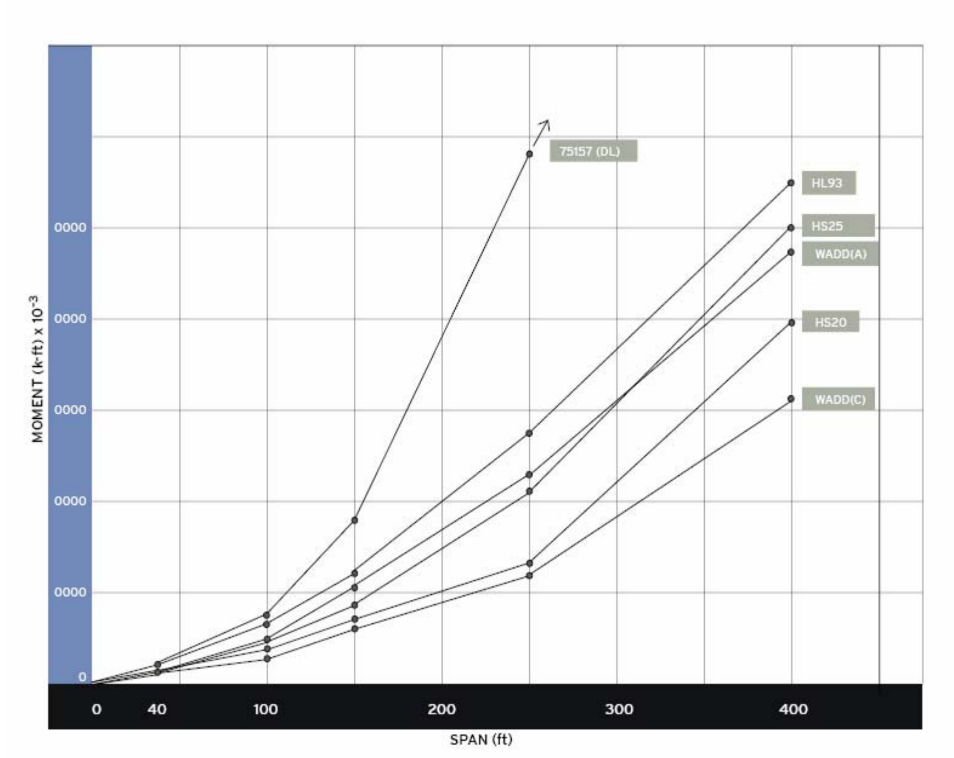

FIGURE 7 Simple-span moments for selected live loads and dead load versus span.

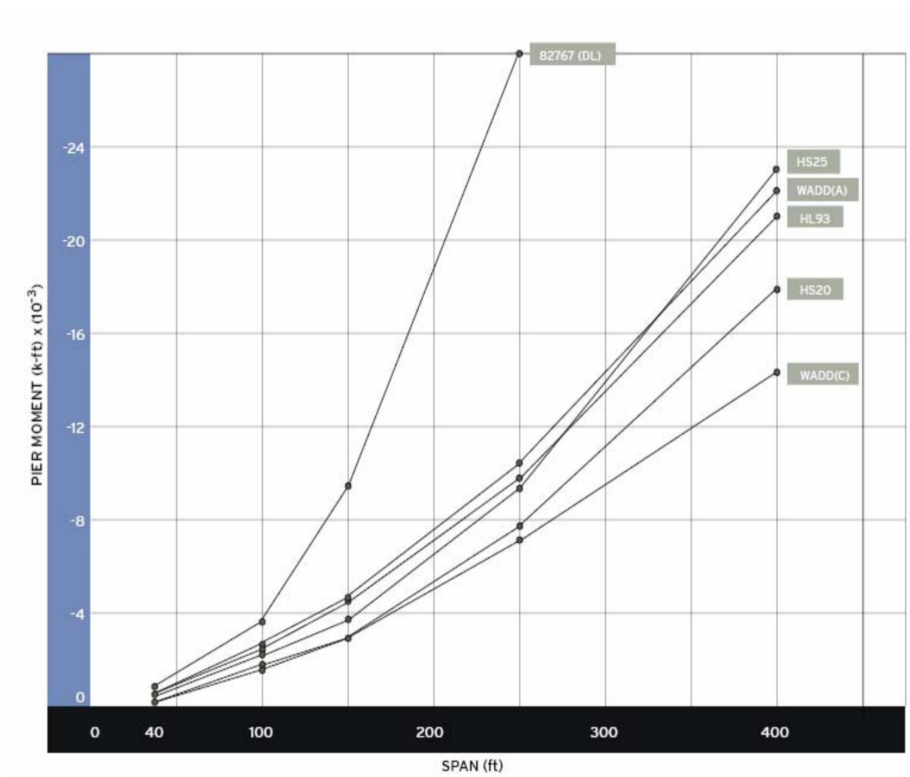

FIGURE 8 Pier lane moments for selected live loads and dead load versus span.

$$F.S. = \frac{Nominal\ strength}{(LL+I)+DL} = \frac{1.3DL + 2.17(LL+I)}{(LL+I)+DL}$$

$$Let\ K = \frac{(LL+I)}{DL}$$

$$F.S. = \frac{1.3DL + 2.17(K)}{(K+1)}$$

For LRFD a similar equation can be written:

$$F.S. = \frac{1.25DL + 1.75(K)}{(K+1)}$$

With LFD, as span length increases, the live-to-dead load ratio is reduced so the design is lighter than it would be with WSD. In terms of WSD, LFD provides a variable factor of safety or reserve capacity for the nominal design loads.

This factor of safety is a measure, based on the strength limit state, of the total capacity of the girder compared with the demand to resist the nominal dead and live loads. The figure quantifies the obvious fact that the LFD and LRFD provisions have a reduced factor of safety with diminished ratio of live-to-dead load. The ratio of the values for designs with different live loads and/or different provisions gives the ratio of excess capacity. Figure 9 demonstrates the factor of safety versus $(LL{+}I)/DL$ for WSD, LFD, and LRFD.

In longer spans where live load is a smaller portion of the total load, the "factor of safety," or reserve capacity of the bridge, approaches the dead load factor of 1.25. To deal with the case of very small live loads, an additional load combination, STRENGTH IV, was provided for dead load alone to be checked with a load factor of 1.5. This load combination is applied when the live load is less than 14% of dead load.

Cable-Stayed Bridges Become the Design of Choice

In the 1980s and 1990s, the cable-stayed bridge dominated, using concrete for shorter spans and composite steel and concrete for longer spans. Bridge highlights of these years:

- Leonhardt and Andra teamed up with a U.S. consultant once again in 1983—this time with engineer Modjeski and Masters—to design the Hale Boggs Bridge, in Luling, Louisiana, boasting a 1,222-ft-long main span.
- As an aside, in 1984, Modjeski and Masters designed and supervised construction of a cantilevered truss parallel to the one they had designed in 1958. The Second Greater New Orleans Bridge was wider and longer, at 1,595 ft, and the connections were shop welded and field bolted.
- The Weirton–Steubenville Bridge over the Ohio River was constructed in 1987 with a main span of 820 ft. It featured a concrete deck acting compositely with steel-edge girders. Designed by Michael Baker Jr., this bridge bears a remarkable similarity to the East Huntington Bridge just a few miles away designed in concrete by Arvid Grant, and opened in 1983.

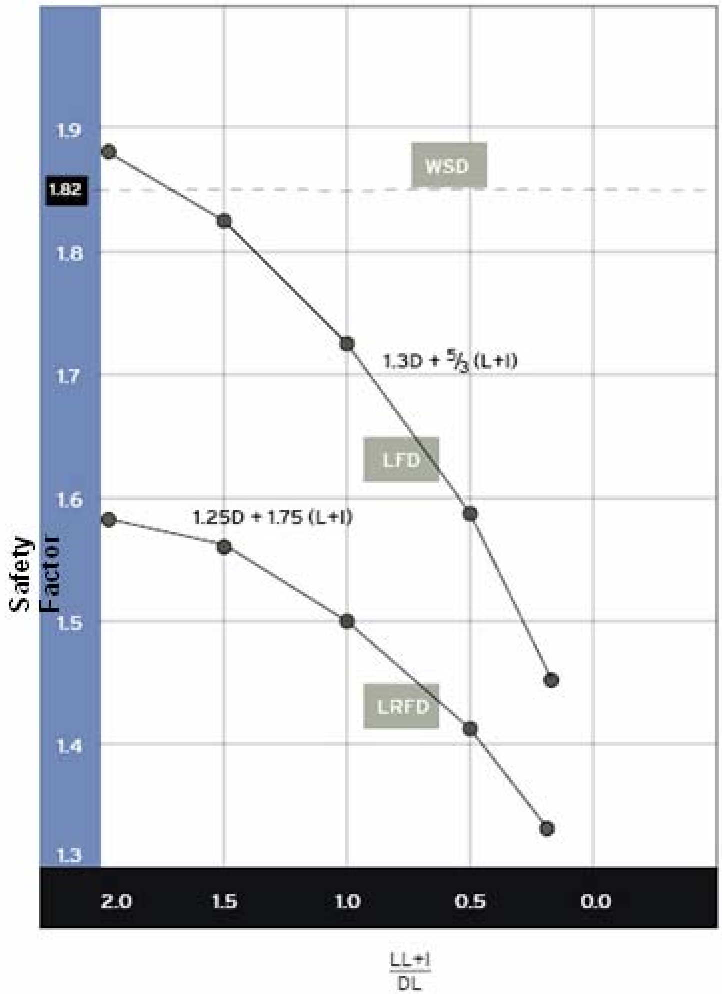

FIGURE 9 Factor of safety versus (*LL*+*I*)/*DL* for WSD, LFD, and LRFD.

- The I-526 Cooper River truss bridge at Charleston, South Carolina, was a departure from traditional truss design when it opened in 1992. The truss structure is a 1,600-ft continuous three-span, parallel-chord warren truss, with no vertical members and without sway bracing. The truss design provides clean lines and an open view to bridge users, totally unlike previous truss designs. The navigation span is 800 ft and the engineer was Howard Needles.

THE 21ST CENTURY

In 2003, the Third Carquinez Bridge opened, replacing the original bridge built in 1927 (Figure 3). The bridge features a closed cell orthotropic steel deck, air-spun cables, and concrete towers, and was designed by the Caltrans/De Leuw OPAC Steinman Joint Venture. This was the first suspension bridge constructed in the United States since the William Preston Lane Bridge opened across Chesapeake Bay in 1972.

WHAT DOES THE FUTURE HOLD?

Nearly 50 years ago, the federal Public Roads Administration approved the final routes to be included in the 41,000-mi National System of Interstate Highways. A new horizon for bridge engineering technology came into view. The next 50 years will no doubt stretch the view even further.

Long-Term Performance

The required life expectancy of bridges is increasing. The cost of maintenance and rehabilitation is increasing, and continues to force the issue of improved performance and lower operating costs.

High-Performance Materials

High performance materials address the issues of strength and durability. Designers will continue to develop and combine materials to create more efficient and longer lasting structures.

For example, Freyssinet combined high-strength steel strand and concrete into a "new material," providing improved stiffness, strength, and serviceability.

Strain compatibility, load redistribution and pre-stress losses have created issues .for composite concrete and steel bridges. As an example, many of these issues are solved in a single stroke by a soft web, consisting of corrugated or intermittent tubular web sections.

The evolution of coating systems will continue to enhance the long term performance of concrete and steel.

Advanced composite materials will become more common as the issues of life expectancy, performance and cost begin to converge.

Longer Bridges

It is thought by senior practitioners in the long-span business that span limitations for current materials have yet to be achieved. Suspension spans and cable-stayed spans can increase significantly before reaching their limitations.

Shorter Bridges

Suspension bridges and cable-stayed bridges are fundamentally efficient structural forms. Why are they limited to longer span ranges? We may see a trend toward use of suspension bridges at much smaller span ranges than previously considered economical as the issues of constructability, productivity and risk management in contracting are addressed.

Construction Technology

Constructability is the key. Designers need to continue to learn the craft of construction engineering.

Segmental construction is fundamentally a construction technique, not a bridge type. What could be more productive than a simple, mass-produced bridge component that can be readily fabricated in a controlled environment and rapidly assembled on-site or at difficult sites with limited access? Segmental steel construction may be a future prospect as construction technology evolves. It is already for orthotropic decks, both new and replacement projects.

Girder launching is coming into favor as the safety and simplicity of launching techniques is improved. It is now understood that the incremental launching of steel bridges has very little impact on the in-service design of the bridge and minimal overhead cost for extra equipment and mobilization.

Pre-cast composite systems and rapid deployment techniques will represent the next generation of technology for the production of bridges.

Contract Delivery and Cost

One thing that constantly amazes is the wide range of unit prices for which bridges are delivered in different parts of the world. An example at the low end is the Millau Bridge in France, which recently was constructed for an impossibly low price (by U.S. standards) of about US$500 per ft^2. This bridge shows incremental innovation on many fronts, including steel pylons assembled on-site and incremental launching of a cable-stayed structure on an unprecedented scale.

How does this happen? Risk in the bridge business should not be managed by spreading the damage around (as underwriters do) but by minimizing risk with the right design and construction solutions. This demands an integrated delivery program that includes design development by the owner and the engineer; final design and a comprehensive, independent peer review by qualified designers; and construction engineering by pre-qualified contractors.

Risk translates into higher construction costs when the contractor is asked to assume responsibility for risks over which he has no control. When a contractor can control his own risk, the result can be significant savings to the owner. This is one benefit of design–build.

CONCLUSION

Our infrastructure system has evolved into something larger than what was envisioned by many in 1956. It has now become a vital part of our urban environments in addition to being a national asset, vital to both local and national economies. We must think value in addition to cost. Value is found in longer life expectancy, enhanced constructability, aesthetics, reduced maintenance, not just least initial cost.

Public acceptance will continue to increase if infrastructure is viewed as an investment in redevelopment and property values.

In the future, our focus should be on the continued development of efficient structures and the increased understanding of constructability and contract delivery. It is not a matter of managing risk, but minimizing risk while enhancing productivity. The evolution of the design and construction of steel bridges will continue to be challenging and exciting.

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The Interstate Highway System and the Development of Prestressed Concrete Bridges

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When the Highway Act was passed in 1956, prestressed concrete was an emerging technology and the application of that technology to bridges was still in its infancy. In the decade prior to the passage of the act only 2% of the bridges built in the United States utilized prestressed concrete as their primary superstructure elements. By the beginning of the new millennium, the number had grown to just under 49%. Of the 47,000 Interstate system bridges built since the passage of the act, 29% have utilized prestressed concrete and 25% utilized reinforced concrete. It is clear that concrete has played a major role in the construction of interstate bridges. It is also clear that the construction of the Interstate system has played a major role in the utilization and development of prestressed concrete in bridges since 29% of all bridges built since the passage of the act use prestressed concrete. Because of its importance as a material component of the interstate system, an examination of the evolution of the technology in the United States is presented. This paper presents the general historical development of prestressed concrete bridges from the early 1940s to the present. This paper also presents some of the challenges that have to be faced by today's prestressed concrete bridge designers as well as some of the needs anticipated for the future.

INTRODUCTION

When the Highway Act of 1956 was passed, prestressed concrete was an emerging technology and the application of that technology to bridges in the United States was still in its infancy. In the decade prior to the passage of the act only 2% of the bridges built in the United States utilized prestressed concrete as their primary superstructure elements. By the beginning of the new millennium, 49% of the bridges built in the United States incorporate prestressed concrete (*1*). Of the 47,000 Interstate system bridges built since the passage of the 1956 Highway Act, 29% utilized prestressed concrete and 25% utilized reinforced concrete. It is clear that concrete has played a major role in the construction of Interstate bridges. It is also clear that the construction of the Interstate system has played a major role in the utilization and development of prestressed concrete in bridges. For this reason, a general presentation of the historical development of prestressed concrete bridges in the United States, as well as some of the present and future issues facing the material, will be presented (Figure 1).

FEDERAL-AID HIGHWAY ACT OF 1956

Recognition of the need for a unified highway system began in the 19th century. During the early history of the United States, virtually all road construction was performed by county and local governments or by private turnpike companies who made their money with tolls. This approach had resulted in a disjointed, inefficient system. In an effort to alleviate this growing problem, in 1893 the Office of Road Inquiry was created to gather and disseminate information on road building.

The Federal-Aid Highway Program began in 1916 during the Presidency of Woodrow Wilson (*2*). Federal aid funds were apportioned to the states to assist in road projects. One of the requirements for federal aid was that each state had to have an established state road agency.

It took the lessons of World War II before the federal government would take action on a National Highway System (NHS). During World War II, a tremendous increase in trucks and new roads were required. The war demonstrated how critical highways were to the defense effort. Thirteen percent of defense plants received all their supplies by truck, and almost all other plants shipped more than half of their products by vehicle. The war also revealed that local control of highways had led to a bewildering array of design standards. Even federal and state highways did not follow basic standards. Some states allowed trucks up to 36,000 lb, while

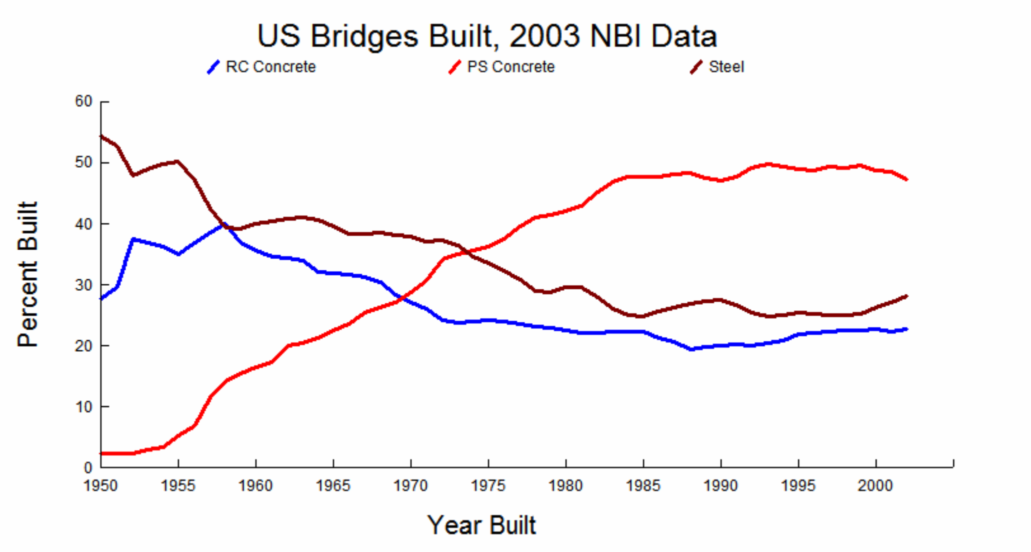

FIGURE 1 Breakdown of bridges by primary construction material.

others restricted anything over 7,000 lb. A government study recommended a national highway system of 33,920 mi, and Congress soon enacted the Federal-Aid Highway Act of 1944, which called for strict, centrally controlled design criteria.

Dwight Eisenhower was an avid supporter of the NHS. He first realized the value of good highways in 1919, when he participated in the U.S. Army's first transcontinental motor convoy from Washington, D.C., to San Francisco, California. On the way west, the convoy experienced an endless series of mechanical difficulties: vehicles stuck in mud or sand or choked by dust; trucks and other equipment crashing through wooden bridges; roads as slippery as ice or the consistency of "gumbo" (Figure 2) On September 5, 1919, after 62 days on the road, the convoy reached San Francisco, where it was greeted with medals, a parade, and speeches (*2*).

During World War II, General Eisenhower saw the advantages Germany enjoyed because of the Autobahn network and the resulting enhanced mobility of the Allies when they fought their way into Germany. This led him to later state, "The old convoy (1919) had started me thinking about good, two-lane highways, but Germany had made me see the wisdom of broader ribbons across the land" (*2*).

The Federal-Aid Highway Act of 1956, signed by President Dwight Eisenhower, expanded the program begun in the Federal-Aid Highway Act of 1944 to include funding for what is now officially called the Dwight D. Eisenhower System of Interstate and Defense Highways, more commonly called the Interstate Highway System. It was under this act that the Federal Highway Trust fund was established. This fund, collected from highway users, provided a stable source of funding for the federal-aid highway program. With a funding mechanism in place, construction of the Interstate system began in earnest and the demand for highway bridges increased accordingly.

During the period between the passage of the Federal-Aid Highway Act of 1944 and the Federal-Aid Highway Act of 1956, a new technology was introduced to the United States that would have a profound impact on the types of bridges that would be built as part of the Interstate system.

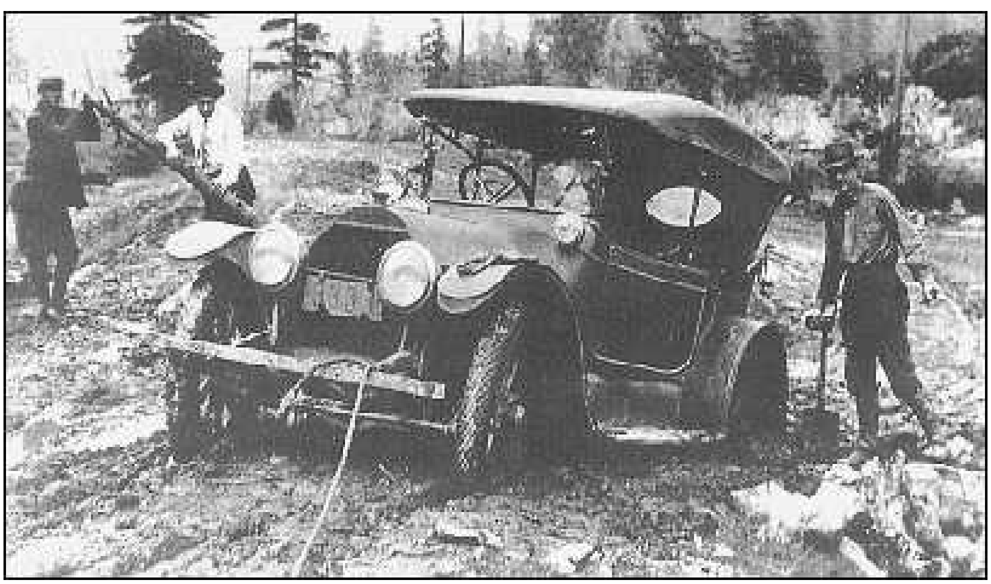

FIGURE 2 Stuck in the "gumbo."

PRESTRESSED BRIDGE TECHNOLOGY COMES TO AMERICA

Though the principals of prestressing had been applied to circular tanks in the United States as early as the 1930s and experimental prestressed piles were driven in New York Harbor in 1936, no single event impacted the development of prestressed concrete in the United States more than the 1946 visit of Professor Gustav Magnel of Belgium (Figure 3). From April to June 1946, Professor Magnel toured the United States as Belgian American Education Foundation Scientist. He was chosen to represent Belgium's entire construction industry as well as its educational and technical professions under the auspices of the Belgian–American Education Foundation (*3*).

Gustav Magnel was Professor of Engineering at the University of Ghent. A teacher revered by his students and known worldwide, he was admired for his methodical, efficient, and down-to-earth approach to his work. He had an easy outgoing manner and was always willing to listen. As a result, he was well received everywhere he went and every chance he got he would tell those he met about his work with prestressed concrete (*3*).

In addition to his lecturing about his work with prestressed concrete, two other events took place during his first visit to the United States that would have a direct bearing on the use of prestressed concrete for bridges in North America. The first event was his introduction to the Preload Corporation of New York. At the time of the introduction, this company was the leader in the design and construction of circular prestressed tanks. Because of their past experience they were well suited to promote Magnel's concept of linear prestressed concrete. (The Preload Corporation would eventually be a sub-contractor for the construction of the girders for the first prestressed concrete bridge in the United States.)

The second event was Magnel's gift to Charles C. Zollman, a former student, of three copies of a manuscript, written in French, of the first comprehensive design and analysis text for prestressed concrete members ever written. Zollman was so impressed with the clarity and simplicity of Magnel's explanation in the book that he requested and received permission from Magnel to translate the manuscript into English for publication. Unfortunately, after more than a year of translation (Magnel added 10 more chapters to the book during translation) American publishers refused to publish the book as "they could not yet see a market for the product" (*4*).

FIGURE 3 Gustav Magnel. [Photo courtesy of Prestressed Concrete Institute (PCI).]

However, Concrete Publications Limited of London agreed to publish the book with a first edition of 6,000 copies in 1948 (Figure 4). The book immediately sold out as did a second revised expanded edition printing of 8,000 copies. A third further expanded edition was published in 1954. Finally recognizing the market, Mcgraw-Hill bought the reprint rights of the third edition late in 1954. It is generally believed that virtually all copies of the book, regardless of the publisher, made their way to the United States and Canada (*3*).

The simplicity and clarity of the book made it the practical tool to which engineering students and practicing engineers referred to for design and analysis of prestressed concrete structures. The impact of this book on the implementation of prestressed concrete in the United States can not be understated. As stated by Charles Zollman, "The basic principals, charts and nomographs generated the necessary confidence in the design of the new material and served as the basis for prestressed concrete publications and engineering practice for many years thereafter." Indeed it set the stage for the prestressed concrete bridges to come.

THE EARLY YEARS OF PRESTRESSED CONCRETE BRIDGES IN AMERICA

As mentioned earlier, Professor Magnel was introduced to representatives of the Preload Company during his initial visit to the United States in 1946. This visit, coupled with the firm's hiring of Magnel's former student Charles Zollman, set the stage for the Preload Corporation to enter the linear prestressing field. In 1948, the firm was working on a project in the general vicinity of Philadelphia and became aware of a planned project to build a new bridge in Fairmont

FIGURE 4 First edition of Magnel's book. (Courtesy of PCI.)

Park. They immediately enlisted the services of Magnel and presented their concept plans to the Principal Assistant City Engineer, R.R. Schofield. Schofield was so impressed with the concept he immediately requested approval from his superiors. Approval was granted to design and construct the Walnut Lane Bridge with contract documents being completed in 1948 and contract award in the early spring of 1949. When completed, the Walnut Lane Bridge was the first major prestressed girder bridge in the United States (*3*). [The term major prestressed girder bridge is used here because the Walnut Lane Bridge was not the first prestressed bridge constructed in the United States. That honor is generally given to Ross Bryan's machine-made concrete block bridge constructed in Tennessee in the late 1940s. These block bridges were not "girder" bridges and were built on secondary roads whereas the Walnut Lane Bridge was a girder bridge on a major parkway. As an added point of interest, the block bridges of Tennessee, as well as some built in Michigan about the same time, can claim the title as the first segmental bridges built in the United States (*4*).]

Very tight construction specifications were prepared by Magnel for the Walnut Lane project. Magnel specified steel forms and a concrete with only a two inch slump. The steel forms were desired to facilitate aggressive external vibration of the forms in addition to aggressive internal vibration. This extensive vibration was necessary due to the low slump coupled with the rather narrow web and the presence of the ducts passing through the web. For reasons of economy, wooden forms were substituted for steel resulting in a limitation in the amount of exterior vibration. Once construction began, slump crept up from 2 in. (a value that was a compromise from Magnel's originally desired zero slump) to 3½ in. These changes in form material and concrete slump resulted in project delays due to longer-than-anticipated times to develop desired concrete strength and the occurrence of numerous voids in the girder.

Because of the uniqueness of the structure, it was decided to test one of the girders to destruction (Figure 5). The test was conducted on October 25, 1949, in the presence of 300 engineers from 17 states and five countries and met all the design criteria. The presence of such a

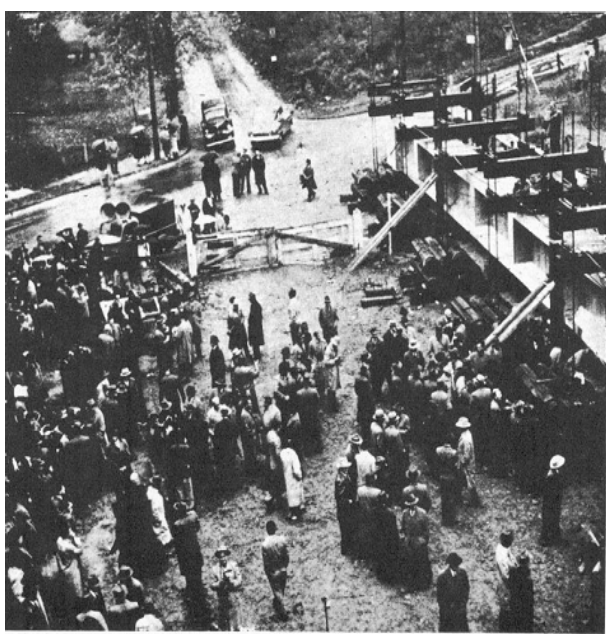

FIGURE 5 Testing of the Walnut Lane Girder. (Photo courtesy of PCI.)

large number of engineers indicates the level of interest developing in this new material. To the amazement of those present the girder had to be loaded to over 10 times the design working load before the top flange failed in compression (Figure 6).

While the Walnut Lane Bridge was under construction, the Preload Corporation learned of plans to construct a bridge across the lower Tampa Bay (Figure 7). The original plans for the bridge, bid several years earlier, called for a 17,424-ft bridge consisting of 36-ft reinforced concrete spans. The original bids were rejected as they came in well over budget. Preload realized that through the use of prestressed concrete the spans could be lengthened to 48 ft, thereby saving a substructure unit every 144 ft. In addition, this could be accomplished without appreciable increases in the superstructure concrete quantities. With this concept in hand, along with Magnel's former student Charles Zollman, Preload engineers traveled to Florida to meet with William Dean, Chief Bridge Engineer of the Florida Department of Roads.

The initial meeting with Dean did not go as expected as Dean had developed some concerns about the use of prestressed concrete. However, after discussions regarding the Walnut Lane Bridge as well as a presentation on the use of prestressed concrete at a local ASCE meeting, Dean became convinced that prestressed concrete should be used for the new Tampa Bay Bridge. Redesign of the bridge was undertaken and bids were received in 1951. At this time, it was the largest contract for prestressed members ever let in any part of the world (*5*).

Like the Walnut Lane Bridge, there was an extensive testing program and great interest in the project. Most of this testing was a result of Dean's realization that the success of the project, as well as prestressed concrete in general, was dependent upon both the potential and limitations of American labor. This understanding, in large part, led to Dean's position on the I-beam standards that were developed under his direction as chairman of a joint committee of the AASHO and the Prestressed Concrete Institute (PCI). Instead of the thin web members that were being used and advocated by the Europeans, not to mention advocated by the Bureau of Public Roads, Dean fought for "fat and sassy" standards that could be fabricated with less difficulty and more consistent quality.

Dean became an advocate for the use of prestressed concrete. He believed in practical detailing and thorough testing. Some of the changes that came about in the early days of prestressed concrete bridges were championed by Dean. He established through testing at the University of Florida that end blocks in pretensioned bridge beams were superfluous. He, along with many others, led a drive for larger strands. In 1957 he concluded that shear keys were not needed between the girders and slabs "if sufficient steel is provided to develop the vertical compression." So he extended the vertical reinforcing into the slab and eliminated the shear key. Before long the practice was followed nationwide.

In large part because of Dean's efforts in Florida, PCI became legally chartered June 18, 1954, in Tampa, Florida. The first PCI convention was held April 21–22, 1955, in Ft. Lauderdale, Florida. The first National Prestressed Concrete Short Course was held in September of that year at the University of Florida and the second PCI convention was held May 16–18, 1956 in Hollywood, Florida.

R.R. Schofield and Bill Dean each made a decision to step out and try this new technology of prestressed concrete. There was no obvious advantage for theses two civil servants to make this decision. If anything, they ran the risk of experiencing severe criticism from individuals advocating the status quo. The only possible explanation for their decision was that they saw the merits and the possibilities of the material. The possibilities of the material proved itself over the next 50 years through constant evolution and improvement.

FIGURE 6 Magnel and others admiring the completed Walnut Lane Bridge. (Photo courtesy of PCI.)

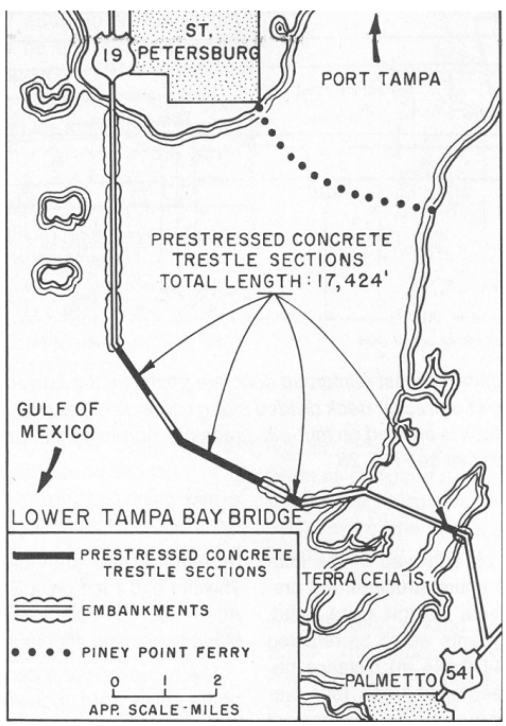

FIGURE 7 Lower Tampa Bay Bridge. (Courtesy of PCI.)

EVOLUTION OF PRESTRESSED CONCRETE BRIDGES IN AMERICA

High-Strength Wire

One of the key forces behind the successful implementation of prestressed concrete bridges in America was the development of high-strength steel wire. In fact some have said that construction of prestressed concrete could not have been possible without the availability of high-strength steel wires. The need for the high-strength steel was essential to maintain a specified compressive stress in the concrete, after prestress losses due to elastic shortening, creep, and shrinkage of concrete has occurred. In initial applications, single high-strength steel wires of Grade 240 ksi were used. Shortly thereafter, seven-wire strands with a guaranteed ultimate tensile strength (GUTS) of 250 ksi became available. By the late 1970s, grades of seven-wire strands were improved to 270 ksi, both stress-relieved and low-relaxation.

Optimization of Sections

In 1952 the Bureau of Public Roads, the forerunner to FHWA, published *Criteria for Prestressed Concrete Bridges,* the first formal prestressed concrete design specifications. Standardization of girder sections was another important factor leading to the growth in the use of prestressed concrete. In the earliest applications of prestressed concrete in bridges, designers developed their own ideas of the best girder sections. The result was that each contractor used slightly different girder shapes. It was too expensive to design custom girders for each project. As a result, representatives of the Bureau of Public Roads (now FHWA), AASHO (now AASHTO), and the PCI began work to standardize bridge I-girder sections. The AASHO–PCI standard girder sections Types I through IV were developed in the late 1950s. Types V and VI followed up in the early 1960s for longer spans. Simultaneously, standard precast, prestressed concrete boxes were developed. Today, adjacent- and spread-box bridges are popular in the Midwest and eastern states for shorter span bridges. There is no doubt that standardization of girders led to simplified design, wider utilization of prestressed concrete for bridges, and more importantly, increased speed of construction and economy.

With advancements in the technology of prestressed concrete design and construction, not to mention local capabilities, numerous states started to refine their designs and to once again develop their own standard sections. As a result, in the late 1970s, FHWA sponsored a study to evaluate existing standard girder sections and to determine the most efficient girders. This study concluded that bulb-tees were the most efficient sections. These sections afforded a reduction in girder weights up to 35% compared to the AASHTO Type VI, and cost savings of up to 17% compared to the AASHTO–PCI girders of equal span capability. Based on conclusions of the FHWA-sponsored study, PCI developed the PCI Bulb-Tee Standard, which was endorsed by the AASHTO bridge engineers at their 1987 annual meeting. In the mid 1990s, when the federal government set the deadline of September 30, 1997, to procure in metric units, the states of Nebraska, Florida, and New England developed their own metric bulb-tees.

An FHWA investigation in the 1990s evaluated the use of existing prestressed concrete girder sections relative to the use of high-strength concrete (*6*). The investigation concluded that the bulb-tee cross section should continue to be considered as a national standard for span lengths from 80 to 200 ft. For span lengths of 80 to 120 ft, sections developed by Colorado and Washington and the PCI bulb-tee were most economical. From 120 to 150 ft, the PCI bulb-tee was most economical and for span lengths from 150 to 200 ft, the Florida and University of Nebraska sections were most economical.

Plant Production

Prestressing of concrete can be accomplished using two basic techniques. In the first, pretensioned girders are manufactured in a plant, and then transported to the bridge site. In the second, girders are cast in place, and then post-tensioned. The latter is favored where weight or length limitations make it uneconomical to transport precast elements. In the United States, 49 states predominantly use precast, prestressed girders, while the state of California practices favors cast-in-place, post-tensioned bridges. The number of applications combining pretensioning and post-tensioning is increasing in the United States. Long-segment, spliced girders were introduced in the late 1950s in New York for spans of up to 320 ft. In the 1980s, this technique gained momentum to extend concrete bridge spans. By the late 1990s, spliced I-girder and bulb-tee spans are again matching the earlier record of 320 ft, set 40 years earlier.

High-Performance Concrete

In 1987, Congress initiated a 5-year Strategic Highway Research Program (SHRP) to develop various products for the highway system to improve constructability and reduce maintenance costs. One of these products was high-performance concrete (HPC). In general, HPC is defined as concrete designed for a certain application in a specific environment. The characteristics of HPC may include ease of placement; consolidation without segregation; high early age strength, long-term strength, and other mechanical properties; low permeability; higher density; lower heat of hydration; improved toughness; volume stability; and longer life in a severe environment. (7)

Implementation of HPC into actual bridges began in 1993 as a partnership between FHWA, state departments of transportation, AASHTO, local agencies, industry, and academia. The program included the construction of demonstration bridges and dissemination of the technology and results at showcase workshops. Eighteen bridges in 13 states were included in the national program. The bridges were located in different climatic regions and used different types of superstructures. Collectively, the bridges demonstrated practical applications of HPC. (*8*, *9*)

The 1998 Transportation Equity Act for the 21st Century (TEA-21) provided a continuation of funding to extend the service life of bridges through the Innovative Bridge Research and Construction (IBRC) Program. Forty-one HPC-related bridge projects were funded during the first 5 years. A 2003 FHWA survey indicated that 44 states have utilized HPC specifications to achieve high-strength concrete, low permeability concrete, or both (*7*).

High-strength concrete enables standard bridge girder cross sections to be designed for longer span lengths, wider girder spacings, or shallower sections. The use of longer span lengths reduces the number of spans and the number of substructures on multi-span bridges. The use of wider girder spacings reduces the number of girders needed for a specific bridge width. The use of shallower sections allows replacement bridges to have a higher load capacity without using a deeper section or roadway altering geometry.

For high-strength concrete to be used efficiently, it needs to be precompressed to the maximum value allowed by the design specifications. Therefore, as the specified concrete compressive strength increases, the prestressing force also needs to increase. The amount of force depends on the diameter, spacing, and strength of the strand and shape of the bottom flange of the beam. Once the bottom flange is full of strands, additional strands can only be placed in the web, which is less efficient because the strands are closer to the neutral axis (*10*). Various investigators (*6,11,12,13*) have shown that the maximum effective concrete strengths for readily available pretensioned beam shapes range from 8,000 to 11,000 psi with 0.5-in. diameter strand and 10,000 to 14,000 psi with 0.6-in. diameter strand. Today's challenge is how to take further advantage of even higher strength concretes using different girder cross sections, larger diameter strands, or higher strength strands.

High-performance concrete provides for improved durability through increased resistance to freeze–thaw damage, chloride penetration, reinforcement corrosion, and abrasion damage. The availability of supplementary cementitious materials, such as fly ash, ground granulated blast furnace slag, and silica fume, in combination with high-range water-reducing admixtures have facilitated the development of low permeability concrete. Unfortunately, the application of higher compressive strength concrete in bridge decks has in some cases, resulted in an increase in the amount of cracking. Today's challenge is to produce a crack-free low permeability concrete bridge deck with a life of at least 100 years. This is particularly important for owners and contractors with limited experience with HPC.

New materials will also find increasing demand in repair and retrofitting. As the bridge inventory continues to age, increasing the usable life of structures will become critical. Some innovative materials, although not economical for complete bridges, will find their niche in retrofit and repair.

Innovation Continues

Another step in the evolution of prestressed concrete applications in the United States is segmental bridges. Although the technique was used on some of the early applications, it gained fast acceptance following construction of the America's first precast concrete segmental box girder bridge in 1973—the John F. Kennedy Memorial Causeway Bridge connecting Corpus Christi and Padre Island, Texas. The evolution of this technology can best be shown through the adaptation of precast concrete segmental bridge technology to a large number of varied and unique applications. On the Long Key Bridge in Florida, the technology was adapted to repetitive span lengths using segments constructed in a factory like environment. When accessibility to the site became a major factor on the Linn Cove Viaduct in North Carolina, unidirectional cantilever erection was developed to allow the contractor to build from the previously completed structure (Figure 8). In Tampa Bay, the new cable stay system was adapted to segmental concrete for the first time in the United States to help the Sunshine Skyway to achieve increased clearance (Figure 9). The beautiful arches of the Natchez Trace Parkway were accomplished by adapting proven precast piers to produce the arched substructure (Figure 10).

Indeed, 30 years later, precast concrete segmental construction has produced some of the most technologically advanced, visually dramatic, and environmentally sensitive structures in North America as well as in many other countries throughout the world. In the mid 1990s, the American Segmental Bridge Institute (ASBI) joined hands with PCI, AASHTO, and FHWA to develop new standard sections for segmental construction. The AASHTO Highway Subcommittee on Bridges and Structures approved standard segmental box sections in 1998. Subsequently, AASHTO–PCI–ASBI Segmental Substructure Standards and Transverse Deck

FIGURE 8 Linn Cove Viaduct construction.

FIGURE 9 Sunshine Skyway construction.

FIGURE 10 Natchez Trace Parkway construction,

Post-Tensioning Standards were also published. The impetus for this effort has been the growing need to enable rapid construction of substructures as well as superstructures. The public demands least inconvenience, and thus, the need for speed of construction.

Late in the 1970s, with the completion of the Pasco–Kennewick Intercity Bridge in Washington State, cable-stayed construction raised the bar for concrete bridge spans to 981 ft. By 1982, the Sunshine Skyway Bridge in Tampa, Florida, set a new record for concrete bridges, with a main span of 1,200 ft. The next year, the Dames Point Bridge in Jacksonville, Florida, extended the record to 1,300 ft.

The growth in use of prestressed concrete in bridge applications in the United States is due to its economy and excellent performance. Prestressed concrete has enjoyed great success in competing in today's "lowest first cost" method of material selection in the United States. As steps are taken to consider life-cycle cost for bridges, it is anticipated that the low maintenance required for prestressed concrete will make it an even more desirable method of bridge construction in the United States.

TOMORROW'S NEEDS

Recently, the FHWA initiated a new program entitled "Highways for LIFE." This program looks to set new goals for highway and bridge engineering. The term LIFE is an acronym for Long Lasting, Innovative Technologies, Fast Construction, Efficient and Safe Pavements and Bridges

The bridge community will play a key role in the implementation of this program. This concept was summed up by former U.S. Department of Transportation Deputy Secretary Michael Jackson when he said, "We need to build them faster, have them last longer, have them be safer and at a lesser cost. Be BOLD and AUDACIOUS in your thinking."

The prestressed concrete industry has always been bold and audacious (The initial concept of prestressing was probably perceived this way by practicing engineers of the time.) The PCI community has accepted this challenge and is looking toward the future for ways to make it happen. The elements of the Highways for LIFE program are well suited for precast and prestressed products. Precast and prestressed concrete need to offer the following benefits:

- **Long Lasting:** Prestressed bridges must survive aggressive environments and last for at least 75 years with little or no maintenance.
- **Innovative Technologies:** Technologies will need to be developed or transferred from other industries to produce better bridges.
- **Fast Construction:** The concept of prefabrication should be extended from simple girder fabrication to complete bridge systems.
- **Efficient Bridges:** By incorporating these concepts we will strive to reduce both the initial cost and the life cycle cost of bridges.

To accomplish these ambitious goals, the industry will need to approach the program on several fronts as outlined below.

New Materials

New materials for use in bridge construction have been developed during the last decade. The development of HPC has significantly improved the strength and durability of highway bridges. In the future, new materials and concrete admixtures will need to be developed to further improve the strength and durability of concrete. The FHWA is already studying a new generation of ultra HPC, where concrete strengths, ductility, and durability are taken to the levels that were unimaginable just one decade ago.

Light-weight concretes will also play a key role in the development of longer spans. Reductions in girder weights also have a positive impact on seismic performance, fabrication, shipping, and erection costs.

Deterioration of internal mild reinforcement in prestressed beams can be a hindrance to long-term durability. New forms of internal reinforcements will need to be developed that reduce corrosion potential and improve performance. Many concepts have been studied in the past decade. This work will need to continue and expand. New reinforcing materials and products from other industries will need to migrate to the bridge market to help achieve long-term durability goals.

Methods of prestressing will also need to be explored. Span lengths and girder efficiencies are often limited by the amount of prestress force that can be introduced to a girder. Innovative concepts such as girder splicing and post tensioning have extended span lengths. New concepts will need to be developed to further increase the efficiency of longer span bridges.

Bridge Girder Configurations and Optimization

The past decade has seen a migration from the bridge designs using the original AASHTO girders to a new family of bulb-tee girders. These sections were developed to provide more efficient designs that are more commensurate with the development of new materials. Inevitably, the development of the next generation of materials will lead to larger prestressing forces and higher strength concretes. These increases will lead to newer beam sections that can take advantage of the new material properties.

New bridge framing concepts that can take better advantage of new materials and strengths are also possible. The conventional concept of lines of girders may prove to be less efficient as alternate framing concepts. Engineers will need to continue to push the envelope of conventional framing systems and explore other framing methods.

Prefabrication

One of the key thrust areas of the Highways for LIFE program is the development of rapid construction technologies. The Interstate Highway System envisioned by President Dwight D. Eisenhower has essentially been realized. The majority of the system is in place. Portions of the system that were constructed in the 1950s and 1960s have either become structurally deficient or functionally obsolete. This is especially true in urban areas where land development has led to an astronomical increase in highway traffic volume. Many of these highways are in need of replacement and widening. Designers are challenged to reconstruct roadways without stopping the existing traffic. This can be comparable to changing the oil in a car with the motor running.

On many roadway reconstruction projects, the construction of the bridge is often the critical path and cause of most of the disruption to the traveling public. The bridge industry must explore viable options for replacing bridges using time frames that were considered impossible a decade ago. At this time, technologies are migrating from other industries to the bridge market to facilitate this process. These technologies include innovative heavy load moving equipment developed by the petrochemical industry. For decades prefabrication has been used in that industry since on site construction of off-shore structures is virtually impossible.

As shown in Figure 11, bridges can now be installed overnight with as little inconvenience to the traveling public as a 20-min rolling roadblock.

It is already possible to construct simple bridges using precast concrete in days rather than months or years. Figure 12 shows a PCI award-winning design for a bridge in New Hampshire. This structure was constructed using all precast elements including the abutments, walls and the spread footings. Construction of this bridge from start of erection of footings to bridge opening took approximately 8 days.

Systems are currently being developed for more complex bridges that will allow for total bridge prefabrication on a larger scale. Currently, the cost for the use of a rapid construction technology in a region is frequently higher than for conventional construction. The challenge to future designers is to develop systems and designs that will be less expensive than conventional construction while employing the concepts of rapid construction using total bridge prefabrication. A change to incorporate delay-related user costs in the decision-making process is needed as well and this change alone may show in many cases the cost-effectiveness of accelerated bridge construction.

Research

The advancement of the concepts described above is only possible with a commensurate level of quality research. In order for the industry to advance, all stakeholders need to work together in a cooperative manner to develop concepts with lasting value. The Transportation Research Board and NCHRP will continue to play an important part in the future development of prestressed concrete technology. The collaboration of numerous departments of transportation, research experts, industry experts and private consultants insures that research is well founded, carefully

FIGURE 11 Complete bridge change-out.

FIGURE 12 Rapid bridge replacement.

executed, and conclusive. Without this approach, there can be duplication of effort and an inevitable loss in the quality and usefulness of the research effort.

The development of new materials and technologies require extensive research to verify that the new concepts can be applied with the same reliability and safety as predicted by existing code specifications. Research should also be used to expand code specifications to take full advantage of the new materials and technologies.

CONCLUSIONS

President Dwight D. Eisenhower was a visionary who envisioned a system of roadways that would interconnect the United States. Professor Gustav Magnel was a visionary who developed and promoted a new concept in concrete design. Men such as R.R. Schofield and William Dean were visionaries that were willing to take a chance on this new building system. Through their efforts and the efforts of the many other innovators who followed them an industry was spawned that has played a major role in the development of the interstate system. The bridge design community in general will need to follow the led of these individuals and be bold and audacious in the use of emerging materials and methods in order to provide the next generation with bridges that will be durable, economical and efficient.

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Condition Assessment of Highway Structures
Past, Present, and Future

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The Silver Bridge collapse of 1967 was the impetus behind the creation of the National Bridge Inspection Standards (NBIS), which mandated the periodic inspection of bridges on and off of the National Highway System (NHS). While the inspections associated with this effort provided useful information about the nation's bridge inventory and their condition, it was insufficient to make reliable programmatic decisions in the face of dwindling funding dollars. Subsequent revisions to federal bridge legislation created funding for bridge management systems (BMS), which advanced the NBIS information to the state where better decisions were possible. Despite the advances in BMS modeling, the condition assessment activities associated with NBIS and BMS still rely heavily on visual inspection, which inherently produces widely variable results. This paper presents a historic perspective of the condition assessment process for our nation's bridges, highlighting significant developments and shortcomings.

The bridge owners of today are faced with the same challenges that faced bridge owners at the inception of the Interstate highway system, that is how to deal with and manage an aging Interstate system, particularly, a system that is today 50-years old with approximately 30% of its bridge inventory rated as structurally deficient or functionally obsolete. The challenge today will be to develop better assessment methodologies that will generate better prediction models so owners can maintain and preserve their bridge inventories, under increased funding restraints. The paper presents highlights of a new initiative, the Long-Term Bridge Performance Program (LTBPP), which was included in the latest highway legislation. This program will provide quantitative data for network and bridge level management; ultimately serving to improve the safety assessment of our nation's bridges.

INTRODUCTION

Enactment of the Federal Highway Act of 1956 created the needed funding mechanism for construction of our nation's modern Interstate highway system. While the act created a 90–10 funding appropriation, with the federal government bearing the greatest burden, the engineering birth of the Interstate highway system was created more than 25 years prior with the issuance of a unified approach to highway design. Highway design originated with the publication of the ASCE's 1924 *Final Report on Specifications for Design and Construction of Steel Highway Superstructures* and later refined in the first edition (1931) of AASHO's *Standard Specification for Highway Design*. Subsequent advancements in engineering specifications through 1956 paved the way for construction of an integrated interstate system of roads and bridges. This system would achieve the initial vision of President Franklin D. Roosevelt who, in 1939, recommended “a special system of direct interregional highways, with all necessary connections through and around cities, designed to meet the requirements of the national defense and the needs of a growing peacetime traffic of longer range.” Primarily as a result of insufficient funding from 1939–1953, minimal progress was made toward construction of the interregional Interstate system. This was soon to change under the leadership of President Dwight D. Eisenhower, who championed the 1956 legislation which delegated $25 billion in federal funds to construction of a system of Interstate and defense highways. Prior to this time, just over $1 billion in federal funds had been allocated through earlier versions of the Federal-Aid Highway Act (*6*).

The 1956 act called for uniform Interstate design standards to accommodate 20-year traffic forecasts. The Bureau of Public Roads (BPR), the precursor of the current-day FHWA, would work with AASHO to develop minimal standards to assure uniformity of design, full-control access, and elimination of grade crossings. A key element of this effort was the integration of new construction with existing transportation systems, namely toll roads, bridges, and tunnels, to form an integrated national roadway system. So began, the “greatest public works program in the history of the world” [Secretary of Commerce Sinclair Weeks, 1956 (*6*)].

In the first year of the program, $1.1 billion was allocated to the states for construction, which was approximately equal to the funding allocated in the previous 10 years. Improvements completed prior to 1956 were woefully inadequate for anticipated 20-year traffic forecasts and in fact BPR reported only 24% of Interstate roadways to be adequate for present traffic levels. The next 50 years however saw fulfillment of Presidents’ Roosevelt and Eisenhower’s vision of a system of Interstate and defense highways. This accomplishment was not without trial and tribulation, and none more significant than the December 1967 collapse of the US-35 Silver Bridge between Point Pleasant, W.V., and Gallipolis, Ohio. This tragedy focused the nation’s attention on the deterioration of aging structures incorporated into the Interstate system.

The challenges posed by the Silver Bridge collapse are not dissimilar to the issues facing present day bridge owners, i.e., how to deal with and manage an aging Interstate system, particularly, a system that is today 50 years old with approximately 30% of its bridge inventory rated as structurally deficient or functionally obsolete. This paper presents the history of bridge condition assessment, which was precipitated by the Silver Bridge collapse, and describes how future research will address the reliability and safety of our nation’s inventory of bridges as we move into the next 50 years of the Interstate system.

INITIATION AND EVOLUTION OF THE NBIS

The tragic collapse of the Silver Bridge in 1967 resulted in 46 fatalities and nine major injuries and prompted national concern about bridge conditions and safety. Although many states such as California and New York had instituted bridge inspection programs since the 1920s, it was not until the Silver Bridge collapse that the critically important issue of bridge safety was brought into national focus. The sudden failure of an eyebar member of the Silver Bridge generated questions about the bridge inventory. How many bridges like the Silver Bridge are there in the country? Can this happen again? What are departments of transportation (DOTs) doing to prevent these types of failures? What are you doing to ensure the safety of the nation's bridges? There were no satisfactory answers to these questions.

Prior to the Silver Bridge collapse, there was no comprehensive, nationwide database of information about the number, type, location, and condition of our nation's bridges. Congressional hearings on the failure produced mandates requiring the U.S. Secretary of Transportation to develop and implement the NBIS and the National Bridge Inventory (NBI). The NBIS, developed by FHWA in cooperation with AASHTO, was enacted as part of the Federal Aid Highway Act of 1971. This landmark legislation established, for the first time in U.S. history uniform, national standards for bridge inspection and safety evaluation. The Act also designated funding for the replacement of deficient bridges on the federal aid highway system through the Special Bridge Replacement Program (SBRP). Under this program, structurally inadequate or functionally obsolete bridges on the federal aid system were eligible for replacement or rehabilitation funding.

The initial NBIS requirements provided for states to perform periodic inspection of bridges in excess of 20 ft (6.1 m) located on federal aid highway systems. Bridge inspection data and collection requirements were established. Qualifications for key bridge inspection personnel were also outlined. Training programs for bridge inspectors were developed and implemented.

Since its enactment, the NBIS has been fine tuned, additional inspection requirements have been added, and funding programs have been updated. The need to monitor safety and condition of all bridges led to the inclusion of all bridges in excess of 20 ft (6.1 m) located on public roads into the NBI. To accommodate this requirement, data collection was enhanced, and training programs were modified to incorporate knowledge gained through research and experience. Funding programs were also similarly expanded to permit the use of federal funds for replacement of both federal-aid and non-federal aid bridges. The expanded funding programs were outlined in the 1978 edition of the Federal Highway Act, or Surface Transportation Assistance Act as it was called. This act also provided for the replacement of the SBRP with the Highway Bridge Replacement and Rehabilitation Program (HBRRP).

Despite efforts to refine inspection programs and fund bridge replacement and rehabilitation, trial and tribulations continued when in 1983 a section of the Mianus River Bridge failed causing several fatalities and disruption of commerce in the heavily traveled I-95 corridor between New York and Connecticut. The failure precipitated significant research into fatigue behavior of steel connections and existing NBIS programs were modified to incorporate more rigorous inspection procedures for fracture critical bridges. Specialized training programs were also strengthened to increase the bridge inspection community's understanding of fatigue and fracture problems.

The safety of our nation's bridges was again brought into question on April 5th, 1987, when disaster struck with the collapse the New York State Thruway (I-90) Bridge across the Schoharie River. Localized flooding caused scour at a central pier, which was followed by a subsequent loss of bearing capacity at the foundation. This condition led to a catastrophic collapse and several fatalities. Other notable scour-induced failures occurred throughout the country, including the Hatchie River Bridge collapse in Tennessee on April 1, 1989. With more than 80% of public road bridges crossing waterways, these bridge failures represented a critical need to revise the NBIS to include scour assessment. As such, legislation was enacted to require periodic underwater inspection of all structures at risk and susceptible to scour damage.

It should be noted that improvements in the condition evaluation of our nation's bridges that were driven by the NBIS and advancements in the management of our bridge inventory through the NBI, were simultaneously accompanied by advancements in design specifications during the first 30 years of the Interstate system. The history of design specifications are highlighted in 50th anniversary papers prepared and presented by the Structures Section Committees (AFF00) of the Transportation Research Board (TRB) at the 85th Annual Meeting of TRB. These papers are contained in the TRB circular where this paper appears.

Lessons learned from these failures; subsequent and ongoing advancements in research; and experience gained from more than 40 years of bridge monitoring and inspection has produced an educated work force of bridge inspectors who proactively monitor the safety of our nation's bridges. These same metrics have produced advancements in the training, data collection, and management of our bridge inventory via the NBI. However, the shortcomings of the inspection and appraisal methodologies, which are rooted in the subjective and qualitative aspects of the visual inspection and data interpretation requirements, have not been upgraded. A summary of major bridge inspection and bridge program funding legislation is provided in Table 1.

THE NATIONAL BRIDGE INVENTORY

The NBI database contains the following types of information: Inventory Information, Condition Ratings, Appraisal Ratings, and Calculated Fields. Data for the NBI is collected per NBIS guidelines, which are outlined in *Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation's Bridges* published by FHWA (*4*).

One of the primary functions of the NBI is to maintain a complete inventory of bridge data related to location, type (superstructure design) and geometry (lengths, clearances, widths, spans, etc.), roadway or feature crossed, NHS designation, service carried and crossed, responsible owner, age, materials of construction, design load capacity, etc. With more than 7 billion bytes of data, the NBI database represents the most comprehensive source of bridge information available, and can be used to classify, review and search the nation's bridge inventory. For example, NBI data can be used to quantify the bridge types constructed over a given period. Figure 1 shows a distribution trend by year for type of construction. Prestressed bridge construction leads the way, with steel and reinforced concrete following.

The NBIS requires periodic visual inspection, with most structures (82%) evaluated once every 2 years. When safety concerns such exist such as from fatigue, scour, and advanced deterioration, etc., inspection intervals may be more frequent. Approximately 14% of the nation's bridges are inspected at intervals of less than 2 years. Similarly, for structures with characteristics that have historically been free of concern, the period of observation may be increased to 4 years. Only 2% of bridges are inspected at intervals of greater than 2 years.

The bridge owners (states, cities, municipalities, etc.) are responsible for these inspections, with oversight by state DOTs and FHWA. Information is collected on the bridge composition and conditions and reported to FHWA where the data is maintained in the NBI database. This information forms the basis of the bridge safety assurance efforts and provides the data necessary for funding apportionments to the states.

Through periodic safety inspections, condition state data is collected on the following structural components:

- The bridge deck, including the wearing surface;
- The superstructure, including all primary load carrying members and connections;

TABLE 1 Summary of Major Bridge Inspection and Bridge Program Funding Legislation and Noteworthy Changes

Act and Date	Requirements
Federal Aid Highway Act of 1970(P.L. 91-605)	• Inventory requirement for all bridges on the federal aid systems • Established minimum data collection requirements • Established minimum qualifications and inspector training programs • Established SBRP
Surface Transportation Assistance Act of 1978 (P.L. 95-599)	• Established HBRRP (extending funding to rehabilitation) to replace SBRP • Extended inventory requirement to all bridges on public roads in excess of 6.1 m • Provided $4.2 billion for the HBRRP, over 4 years
Highway Improvement Act of 1982	• Provided $7.1 billion for the HBRRP over 4 years
Surface Transportation and Uniform Relocation Assistance Act of 1987	• Provided $8.2 billion for the HBRRP over 5 years. • Added requirements for underwater inspections and fracture critical inspections • Allowed increased inspection intervals for certain types of bridges
Intermodal Surface Transportation Efficiency Act of 1991 (ISTEA)	• Provided $16.1 billion for the HBRRP over 6 years • Mandated state implementation of BMSs
National Highway System Designation Act of 1995	• Repealed mandate for management system implementation
Transportation Equity Act for the 21st Century (TEA-21)	• Provided $20.4 billion in HBRRP funding over 6 years
Safe, Accountable, Flexible, Efficient Transportation Equity Act: A Legacy for Users (SAFETEA-LU)	• Provided funding for systematic preventive maintenance, in addition to bridge replacement and rehabilitation • Appropriates $21.6 billion in bridge program funding over 5 years

Source: FHWA (*1*)

- The substructure, considering the abutments and all piers;
- Culverts, recorded only for culvert bridges; and
- Channel and channel protective systems, for all structures crossing waterways.

The NBIS uses a 10-point rating system, with 9 representing excellent, as-new condition and 0 representing a failed condition. A summary of the rating system is provided in Table 2. Ratings of 3 and less classify the component as being deficient.

Bridge inspectors assign condition ratings based on experience, training and visual review of the subject component. The condition ratings are used to describe the existing, in place status of a component, not its as-built state. Engineers assign condition ratings by evaluating the severity of deterioration or disrepair and the extent to which it is widespread throughout the component being rated. This methodology is highly subjective and produces variable results. Research findings from FHWA's study of the reliability of the visual inspection method found significant variability of condition rating assignments using visual inspection (*5*). While

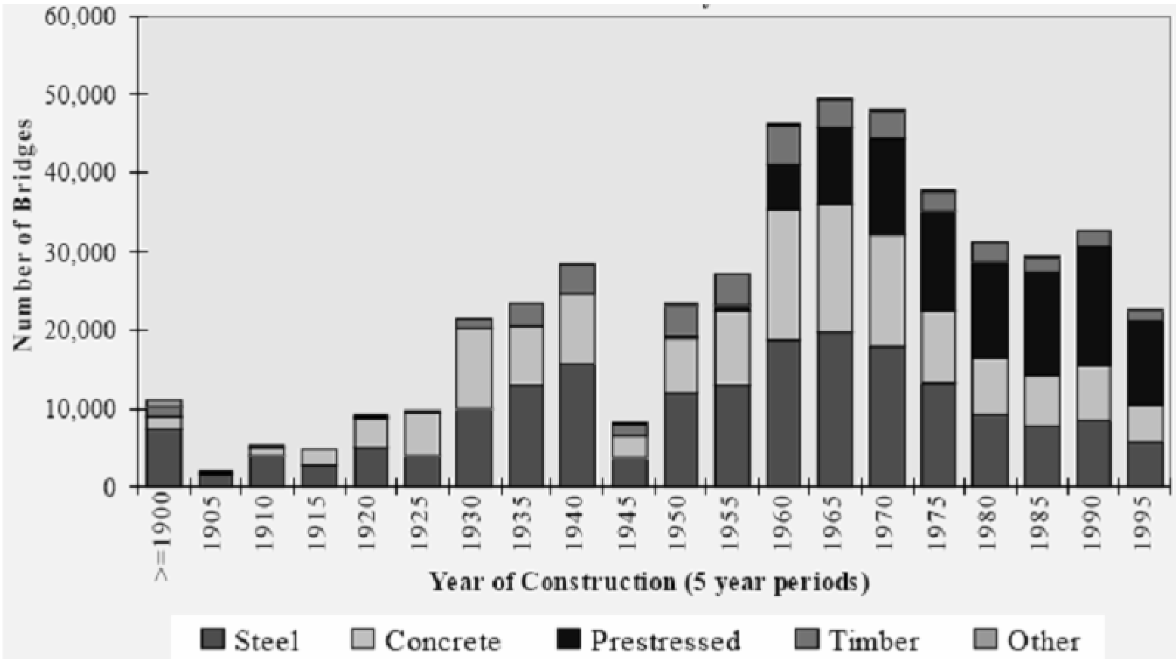

FIGURE 1 Distribution of bridges by year of construction and type of material.

TABLE 2 Bridge Condition Ratings

Rating	Category	Description
9	Excellent condition	
8	Very good condition	
7	Good condition	No problems noted.
6	Satisfactory condition	Some minor problems.
5	Fair condition	All primary structural elements are sound but may have minor section loss, cracking, spalling, or scour.
4	Poor condition	Advanced section loss, deterioration, spalling, or scour.
3	Serious condition	Loss of section, deterioration, spalling, or scour have seriously affected primary structural components. Local failures are possible. Fatigue cracks in steel or shear cracks in concrete may be present.
2	Critical condition	Advanced deterioration of primary structural elements. Fatigue cracks in steel or shear cracks in concrete may be present or scour may have removed substructure support. Unless closely monitored, it may be necessary to close the bridge until corrective action is taken.
1	Imminent failure condition	Major deterioration or section loss present in critical structural components, or obvious loss present in critical structural components, or obvious vertical or horizontal movement affecting structure stability. Bridge is closed to traffic but corrective action may put back in light service.
0	Failed condition	Out of service; beyond corrective action.

Source: FHWA (*4*)

extensive training is offered by the FHWA to assist in calibrating the nation's inspectors, the subjective nature of the ratings and inspection process affects the reliability of the data being collected.

Functional adequacy is also a concern in the bridge inventory. Following collection of the inventory information and condition ratings, appraisal ratings are calculated to assess the adequacy of the structure to provide the required service. Appraisal ratings are quantified for

- Structural evaluations (load carrying capacities);
- Deck geometry (indicating constrictions which affect safety);
- Underclearances (which, if insufficient, results in detours); and
- Waterway adequacy (the ability of the opening to handle the flow-rates).

Appraisal rating scales range from 0 to 9, with 9 being superior to present desired criteria and 0 being bridge closed. Based on the appraisal rating assigned, a bridge may be considered structurally deficient or functionally obsolete. Appraisal ratings of 3 or less classify a bridge as deficient or obsolete. Structural deficiencies result from poor condition ratings or from low load ratings. Functional obsolescence results from low appraisal ratings or from low design load capacities. Inadequate waterway adequacy can be a contributing factor for either structural deficiency or functional obsolescence.

STATUS OF OUR NATION'S BRIDGES

The status of our nation's bridges is based on the information contained in the NBI, which includes nine different appraisal ratings. The various appraisals used to classify the service state of our bridges are shown in the vertical axis of Figure 2. The horizontal bars represent the number of bridges classified as either deficient (solid) or non-deficient (hatched) by the NBIS.

The appraisal ratings are essentially a comparison to current design standards. Many of the appraisal-rating protocols are based on subjective criteria while others are based upon numeric criteria. For example, a two-lane bridge subjected to two-way traffic with an average daily traffic of less than 100 vehicles per day (vpd) is assigned a deck geometry appraisal rating of 3 if it is less than 18 ft wide. Appraisal ratings increase as width increases. The association between these criteria and bridge safety or traffic flow is subjective. A bridge is classified as deficient if the appraisal rating is 3 or less.

By law, NBI data is used to appropriate HBRRP funds to each state on the basis of total needs for bridge improvement projects. The eligibility of funds for a specific bridge project depends on an assessment of various factors and the sufficiency rating. The sufficiency rating formula is described in the *Recording and Coding Guide for the Structural Inventory and Appraisal of the Nation's Bridges* (*4*).

The sufficiency rating formula yields a score of 0 to 100%. Bridges with a sufficiency rating of 80% or less are eligible for HBRRP funds for rehabilitation; a bridge is eligible for replacement if the sufficiency rating falls below 50%. The overall rating is based on three factors: structural adequacy and safety which accounts for 55% of the rating; serviceability and functional obsolescence which accounts for 30% of the rating; and essentiality for public use which accounts for the remainder. The sufficiency rating can be reduced by up to an additional 13% if certain factors such as detour length, traffic safety features or structure type are detrimental to the public.

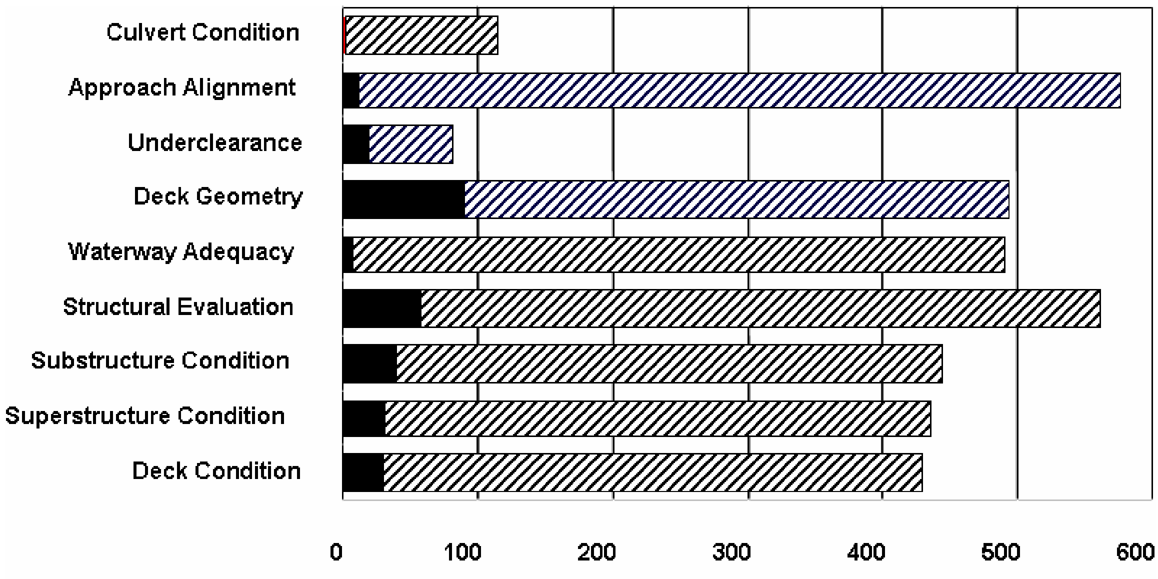

FIGURE 2 Summary of bridge deficiencies from 2002 NBI data (*1*).

So what has the NBI revealed about the condition of our nation's bridges? Approximately 30% of the structures included in the NBI are either structurally deficient or functionally obsolete. Of the nearly 600,000 structures in the inventory, 80% are bridges and 20% are large culverts. Looking at the bridge-only inventory, 80% are off the NHS. So of the 100,000+ bridges on the NHS, nearly 28% are classified as deficient. Of this 28%, 20% are classified as functionally obsolete and 8% are structurally deficient. Note that bridges classified as structural deficient, may also be functionally obsolete but are not included in both totals if this is the case.

Figure 3 shows the deficiency classifications for the nine appraisal ratings. Focusing on deficient bridges, the most frequent deficiency is a narrow roadway, close to 90,000 bridges. The second most frequent and most important from a bridge safety perspective is the structural appraisal rating. Almost 60,000 bridges have load ratings which are well below current standards. Waterway appraisal is not a major contributor, yet failures during flooding are the single greatest cause of bridge collapse. Vulnerability in general is not considered.

MANAGEMENT OF OUR BRIDGES

In spite of various shortcomings, the NBI has proven adequate for administration of the HBRRP in terms of equitably allocating funds to the States. The subjective, non-quantitative and gross nature of the NBI data has proven to be inadequate for owner-level bridge management. In fact, most states augment the NBI data for their own purposes. The NBI is also severely limited in providing adequate measures of bridge performance in support of continuing quality improvement and asset management initiatives. Transportation legislation of 1991 (ISTEA) mandated implementation of a BMS. Several BMSs have been developed, with PONTIS and BRIDGIT being the most widely employed (*3*).

It is commonly recognized that available funding levels authorized by Congress have

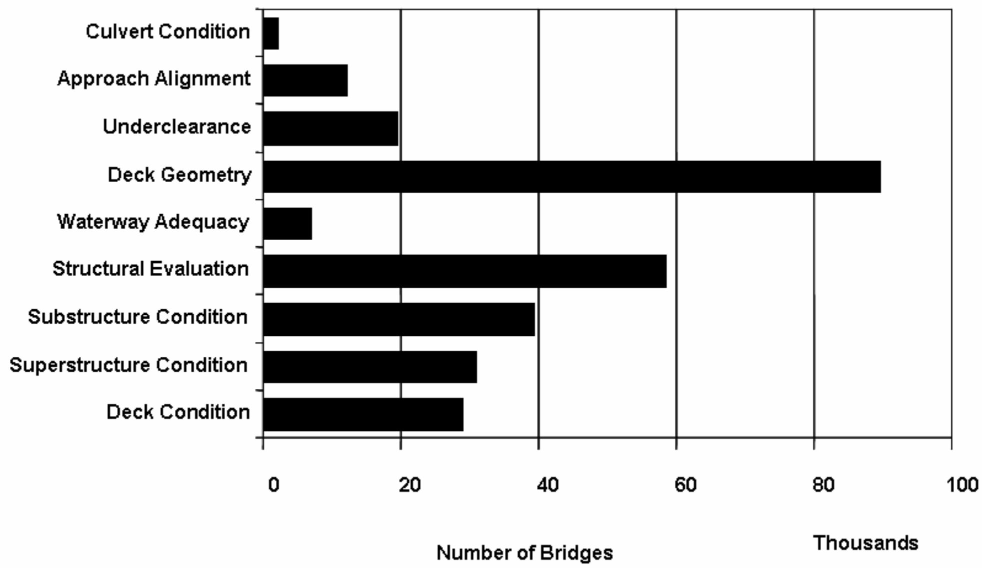

FIGURE 3 Number of deficient bridges in each appraisal category (*1*).

been insufficient to keep pace with the number of NBI-classified deficient bridges requiring replacement or rehabilitation. The subjective and qualitative nature of the NBIS fueled concerns that optimal funding decisions were not possible using this data. Hence, BMS development was pursued.

BMS are element-level inspection systems that are based upon a more refined bridge description wherein the bridge is defined as a collection of elements, such as steel girders, concrete abutments, bearings and a concrete deck. A total of 160 standard elements can be used to describe the bridge. The condition state of each element is then determined via visual inspection using standard protocols that are more discrete than the NBIS. Here the condition of each element is defined in engineering terms and are on a scale from 1 to 5 for most elements. For example, if 12 ft of a 24-ft unpainted steel open girder is in excellent condition with little or no corrosion, then 50% of the girder is in condition state 1.

BMS information is used to predict future conditions on an element by element basis. Repair, replacement or maintenance scenarios are evaluated under different budget constraints to determine the impacts of the various alternatives. The “what-if” scenarios are evaluated using deterioration, cost and optimization models.

Deterioration models predict the condition of bridge elements at any given point in the future and may be deterministic or probabilistic in nature. A deterministic model predicts that a bridge will deteriorate at a known, given rate. A probabilistic model takes into consideration that the actual deterioration rate is unknown, and includes a probability that the bridge will actually deteriorate at a certain rate. In addition, most deterioration models are patterned as a Markov process. This type of model predicts deterioration in a probabilistic fashion and on the basis of the current, not historical condition of the element.

A BMS typically estimates two types of costs: improvement and agency. Improvement costs are estimated to determine the cost of a maintenance action to improve the condition of an element. The expected user cost savings for safety and serviceability improvements should also be estimated.

From the results of the cost and deterioration models, the optimization models determine the least-cost maintenance, repair, and rehabilitation strategies for bridge elements using life-cycle cost analysis or some comparable procedure. These more advanced BMS have allowed agencies to optimization budget funding, but the data collection and management of these system has proven cumbersome.

While the BMS represents a significant improvement over the NBIS, the condition assessment task is still being driven by visual inspection, which has been shown to produce variable and unreliable results.

SHORTCOMINGS AND NEEDS

As our nation's inventory of bridges aged, it became apparent in the early 1990s that available funding for bridge replacement and rehabilitation was insufficient to catch-up on the backlog of deficient bridges. To make better use of limited funding, transportation legislation funded advancements in probabilistic and quantitative systems for bridge management. This was a significant step forward in funding appropriation and management of our bridge inventory.

Even now with the passage of the most recent transportation legislation (SAFETEA-LU), the nation remains faced with a shortfall in bridge replacement and rehabilitation funding. To help off-set this shortfall, SAFETEA-LU legislation includes mechanisms for preventative maintenance appropriations which will serve to drive owners to maximize the useful life of bridges. SAFETEA-LU, like the bill before it, also provides for research advancements in the quantitative assessment of our bridge inventory. These research efforts are expected to yield better modeling tools that will allow better management of our limited funds in addressing bridge deficiencies, which will be defined by quantitative methodologies.

The key to improving our bridge management and funding allocation tools is to recognize that visual inspection is the limiting variable in both NBIS and BMS. As mentioned earlier, research conducted by the FHWA Nondestructive Evaluation (NDE) Validation Center has shown that visual inspection results are widely variable and unreliable from bridge to bridge and from inspector to inspector (*5*). This study showed that the range of condition ratings assigned by multiple teams inspecting the same bridge will vary by three or four numerical values. When considering the NBIS rating system has nine condition states, and nine and zero are new and collapsed respectively, the variability of results is concerning. This study and others like it indicate that NBIS and BMS condition states are grossly defined and so subjective in nature that accurate quantification of bridge deterioration and its change over time is impossible. This is only a portion of the shortcomings of the visual-based systems currently employed in the assessment of our bridge inventory.

Visual inspection also does not capture hidden deterioration or damage. There are many types of damage and deterioration which need to be detected and measured in order to determine if a bridge is safe or if repairs are required. Many of these are difficult to detect visually unless the damage or deterioration is severe. For example, it is not possible to look at a bridge and determine if it has been overloaded or if it has settled unless the damage is so severe as to cause the lines of the bridge to change. Frozen bearings, corrosion, and fatigue damage can exist with no visible indications.

Improvements in detection and quantitative measurement methodologies are needed to optimize BMS. These detection and measurement improvements need to come in four areas: deterioration, damage, operations, and service.

Deterioration assessment methodologies need to be capable of detecting and measuring corrosion, fatigue, water absorption, loss of prestress, unintended structural behavior, to name just a few. Damage assessment methodologies need to be capable of detecting and measuring impact, overload, scour, seismic, fracture, settlement, loss of section, inoperative bearings, and others.

Operational assessment methodologies need to be capable of detecting and measuring average daily traffic, weigh-in-motion, stress, strain, deflection, and displacement. Service assessment methodologies need to be capable of detecting and measuring congestion, accidents, traffic bottleneck, rerouting due to clearance issues, etc. Operational performance of bridges is currently not measured and vulnerability to damage or failure due to extreme or random events is not considered.

There is a need to more accurately quantify the operational performance of highway bridges. The performance measures which are most relevant to the traveling public are congestion, accidents, and service. These same performance measures can help to value the assets in terms of user costs and benefits. These need to be measured and included in the appraisal methodology. It is a basic tenet of modern management that if you can’t measure it you can’t manage it.

RESEARCH NEEDS FOR THE NEXT 50 YEARS

SAFETEA-LU will bring needed funding to research so that shortcomings of NBIS and BMS can be studied and overcome. Key initiatives in SAFETEA-LU include the Long-Term Bridge Research Program wherein the Innovative Bridge Research and Deployment Program is continued, with a new set-aside for high performance concrete bridge technology research and development. In addition, several new initiatives to address bridge life and performance have been funded. These include long-term bridge performance, high performing steel bridge research and technology transfer, and steel bridge testing. Additional funding is also allocated to the Turner–Fairbank Highway Research Center to continue research in NDE technologies.

These programs, and more specifically the LTBPP, will develop and validate technology that can provide the quantitative and objective information necessary to move beyond our current subjectively based BMSs. The need to evolve to a more quantitatively driven management approach has been identified and emphasized by the United States National Academies of Science and Engineering, stakeholders from the public, private, and academic communities, and DOTs (*2*). The need for reliable and timely data and information is recognized as critical to the efficient management of the nation’s highways.

Technology development must be accompanied by improvements in decision support tools to integrate probabilistic life-cycle analysis into infrastructure management. This will require improvements in the way infrastructure assets are valued and benefits of the system are quantified. A long-term bridge monitoring program can address these needs.

The safety assurance of highway structures for extreme events would also be greatly enhanced by short- and long-term monitoring and measurement of the loading and structural response during extreme events. As already demonstrated, assessment and management of bridges and other structures demand the quantitative measurements provided by the monitoring of structures. It is not possible to develop enhanced specifications without long-term observation and quantitative measurement of structural behavior and deterioration.

Finally, the basic information necessary to support the automation of design, construction and maintenance, identified as a national priority for infrastructure research and development, must be provided by sensing and measurement technology.

The LTBPP will serve as the platform to develop better BMSs; systems that are founded on quantitative methodologies for detection and measurement. The program would be modeled after the Long-Term Pavement Performance Program, and will include detailed inspections and periodic evaluations conducted on a representative sample (in the thousands) of bridges to monitor and measure their performance over an extended period of time (at least 20 years). It is anticipated that the resulting database will provide high quality, quantitative, performance data for highway bridges to support improved designs, improved predictive models, and better bridge management systems. Furthermore, the data will be instrumental in improving design specifications.

A second component of this LTBPP will be a subset of instrumented bridges (in the hundreds) that can provide continuous long-term structural bridge performance data. The third component of the program will include detailed forensic autopsies of several hundred bridges each year (out of the several thousand bridges that are decommissioned each year). These forensic autopsies could also be conducted for bridges that fail unexpectedly. The intent is to collect valuable performance data on corrosion, overloads, alkali-silicate reaction, and all other deterioration processes.

The LTBPP will be designed to accommodate and develop both general trend information, and information specific to a variety of bridge types, locations, and environmental exposures.

To summarize, long-term bridge monitoring can provide quantitative data for network and bridge-level management. This could contribute to a much greater level of reliability and utility of data necessary for asset management. Bridge safety, especially during extreme events, is enhanced by measurement and monitoring of critical bridge components. Enhanced safety, reliability and efficient maintenance can result from improved incident detection and assessment. Global bridge health and performance assessment in support of asset management and enhanced specifications must be, and arguably can only be, accomplished using quantitative measurement methods. The subjective assessment methodologies employed in the last 50 years are simply not adequate to meet the needs of our bridge condition assessment efforts over the next 50 years.

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SEISMIC DESIGN OF BRIDGES COMMITTEE

Seismic Design and Retrofit of Bridges Using Load and Resistance Factor Design

Past, Present, and Future

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For almost 70 years, bridges in the United States have been designed using a working stress design specification, updated at regular intervals to reflect recent research results and field experience. Nevertheless, growing concern with inconsistencies in the specifications, and the fact that variability in the loads and materials were not explicitly addressed, an entirely new specification was developed in the early 1990s based on load and resistance factor design (LRFD). This new document is the *LRFD Bridge Design Specifications* and is the subject of this paper. In particular, the seismic design provisions are presented and the selection of load and resistance factors is discussed. Bridge performance in recent earthquakes is then reviewed and it is concluded that refinements are necessary to the LRFD methodology in order to explicitly consider response for multilevel earthquakes. Performance-based design (PBD) is introduced, and recent efforts to implement PBD for new and existing bridges are discussed. Specifically, efforts to introduce dual-level PBD for the design and retrofit of bridges, by AASHTO and FHWA respectively, are summarized.

AASHTO LRFD SPECIFICATION

Brief History

It is generally recognized (*1*) that the first national standard for the design and construction of bridges in the United States was published in 1931 by AASHO, predecessor to AASHTO. With the advent of the automobile and the establishment of highway departments in all of the American states dating back to the turn of the last century, the design, construction, and maintenance of most U.S. bridges was the responsibility of these departments and, more specifically, the chief bridge engineer within each department. It was natural, therefore, that these engineers, acting collectively as the subcommittee on bridges and structures would become the author and guardian of this first bridge standard.

This first publication was entitled *Standard Specifications for Highway Bridges and Incidental Structures*. It soon became the *de facto* national standard and, as such, was adopted and used not only by the state highway departments but also by other bridge-owning authorities and agencies in the United States and abroad. Within a few years, the last three words of the original title were deleted and the specification has been reissued in consecutive editions, at approximately 4-year intervals ever since as the *Standard Specifications for Highway Bridges.* The 17th edition appeared in 2002 (*2*).

The body of knowledge related to the design of highway bridges has grown rapidly since 1931 and continues to do so. Theory and practice have evolved greatly, reflecting advances in understanding the properties of existing and new materials, in more rational and accurate analysis of structural behavior, in the advent of computers and rapidly advancing computer technology, in the study of natural hazards such as seismic events and stream scour, and in many other areas. Advances in these areas have increased dramatically in recent years.

To accommodate this growth, not just in the United States but around the world, the AASHTO Subcommittee on Research, at the request of the Bridge and Structures Subcommittee, undertook an assessment of current U.S. bridge design practice and specifications, reviewed foreign specifications and codes, and reexamined the philosophies underlying the *Standard Specifications.* This assessment was completed in 1987 and its principal recommendation was to develop an entirely new specification based on LRFD, a philosophy that has gained wide acceptance elsewhere in the world and particularly in Canada and Europe.

From its inception until the early 1970s, the design philosophy embedded within the *Standard Specifications* was one known as working stress design (WSD). WSD establishes allowable stresses as a fraction of a given material's load-carrying capacity and requires that calculated design stress be less than these allowable stresses. Beginning in the early 1970s, WSD began to be adjusted to reflect the variability of certain load types, such as traffic loads and wind forces, by introducing load factors, a design philosophy referred to as load factor design (LFD). Both WSD and LFD are reflected in the current edition of the *Standard Specifications* (*2*).

But a disadvantage of both WSD and LFD is that neither considers the variability in the properties of the structural elements in a direct and explicit manner, as is done with the applied loads. However, LRFD offers a mechanism to include this variability in an explicit manner. To do so LRFD makes extensive use of statistical methods and represents a major step forward in the design and analysis of highway bridges. Advantages include superior serviceability, enhanced long-term maintainability, and more uniform levels of safety.

The decision to develop an LRFD specification for highway bridges led to a 5-year effort, involving all 20 technical committees of the AASHTO Bridge Subcommittee, a team of 50 researchers and practitioners, and a project advisory panel. After five successive drafts and numerous trial designs by 14 state departments of transportation (DOTs), the first edition was published in 1994. The second edition was released in 1998 (*1*).

Scope

The AASHTO *LRFD Specifications* are intended for the design, evaluation, and rehabilitation of both fixed and movable bridges. They emphasize the importance of redundancy and ductility, and protection against extreme events such as scour, collision and earthquake. Nevertheless the provisions are intended to be the minimum requirements necessary for public safety. Owners must specify more rigorous requirements when higher levels of performance are required, such as for critically important bridges.

Philosophy

The basic philosophy of the specifications is that bridges shall be explicitly designed for specified limit states to achieve the objectives of constructability, safety, and serviceability with due regard to issues of ease of inspection, economy, and aesthetics [Art 1.3.1, (*1*)].

Each component and connection in a bridge shall be designed for the following limit states:

- *Service limit state*, which shall be taken as restrictions on stress, deformation, and crack width under normal service conditions, as determined by experience and which cannot always be derived solely from strength or statistical considerations [Art 1.3.2.2, (*1*)].
- *Fatigue and fracture limit state*, which shall be taken as restrictions on stress range as a result of single design truck occurring at the number of expected stress range cycles; the fracture limit state is based on a set of material toughness requirements of the AASHTO Material Specifications [Art 1.3.2.3, (*1*)].
- *Strength limit state*, which shall be taken to ensure that strength and stability both local and global are provided to resist the statistically specified significant load combinations that a bridge will experience in its design life [Art 1.3.2.4, (*1*)].
- *Extreme event limit states*, which shall be taken to ensure the structural survival of a bridge during a major earthquake or flood, or when impacted by a vessel, vehicle, or ice flow, possibly under scoured conditions [Art 1.3.2.5, (*1*)].

For each limit state, the demand on each component and connection due to self weight and external loads shall be less than or equal to the capacity of these elements to resist these demands. In other words the following equation must be satisfied:

$Q \leq R_r$

where

Q = total factored force effect (i.e., total demand in the given limit state)
= $\Sigma\, \eta_i\, \gamma_i\, Q_i$;
R_r = factored resistance (actual capacity);
= $\phi\ R_n$;
η_i = load modifier, a factor related to ductility, redundancy and operational importance $\eta_D\, \eta_R\, \eta_I \geq 0.95$, when an upper bound is used for γ_i;
η_D = a factor relating to ductility;
η_R = a factor relating to redundancy;
η_I = a factor relating to operational importance (0.95 – 1.05);
γ_i = load factor, a statistically based multiplier applied to force effects (Q_i) to account for the variability of loads, the lack of accuracy in the analysis, and the probability of simultaneous occurrence of different loads;
Q_i = force effect including a stress, stress resultant (axial force, shear force, flexural or torsional moment), or deformation, caused by the applied loads, imposed displacements or volumetric changes;

ϕ = resistance factor, a statistically based multiplier applied to nominal resistance, to account for variability of material properties, structural dimensions, workmanship and uncertainty in the prediction of resistance;
R_n = nominal resistance of a component or connection to force effects as indicated by the dimensions specified in the contract documents and by the permissible stresses, deformations, or specified strength of materials; and
Σ = summation over all load cases comprising the limit state (load combination) under consideration.

The response of structural components or connections beyond the elastic limit can be characterized by either brittle or ductile behavior. Brittle behavior is undesirable because it implies the sudden loss of load-carrying capacity when the elastic limit is exceeded. Ductile behavior is characterized by significant inelastic deformation before any loss of load-carrying capacity occurs. Ductile behavior provides warning of structural failure by large inelastic deformations. Under repeated seismic loading, large reversed cycles of inelastic deformation dissipate energy and have a beneficial effect on structure survival. η_D factors are selected accordingly, in the range 0.95–1.05.

Also of great benefit during extreme events is structural redundancy. Multiple load paths and structural continuity are essential to survival under ultimate conditions. An element whose failure is expected to cause collapse of the bridge shall be designated as failure-critical, and the associated system is non-redundant. η_R factors are selected accordingly, in the range 0.95–1.05.

Load and Resistance Factors for the Seismic Limit State

Three service, one fatigue, five strength, and two extreme event limit states must be considered and load case combinations for each are specified in the provisions [Art 3.4, (*1*)].

The load combination for Extreme Event I is the only combination involving earthquake effects. Loads assumed to be acting at the same time as the seismic loads, are the permanent loads and a portion of the live load. The simultaneous occurrence of other limit states and extreme events is considered very unlikely and are omitted from the seismic event.

Load factors for the more common permanent loads vary from 1.25 to 1.50, and for live load from 0.0 to 0.5, depending on where the bridge is located (in an urban area with heavy traffic volumes, $\gamma_{EQ} = 0.5$). The load factor for earthquake effects is 1.0, reflecting the fact that the seismic design coefficients are based on an unfactored response spectrum for a 500-year earthquake. In summary the total factored force effect is given by:

$$Q = \eta \, \Sigma \, \gamma_i \, Q_i$$

$$= \eta \, [\gamma_p \, \mathrm{DL} + \gamma_{\mathrm{EQ}} \, \mathrm{LL} + 1.00 \, \mathrm{EQ}]$$

where

η = modifier for the ductility of the component or connection and for the redundancy and importance of the bridge;
γ_p = load factor for permanent load effect;

γ_{EQ} = load factor for portion of live load assumed to be on bridge at time of earthquake (usually $\gamma_{EQ} = 0.0$, but up to 0.5 is recommended for bridges in urban areas with heavy traffic volumes);
DL = permanent load effect including dead load, wearing surfaces, earth pressures and the like;
LL = live load effect on the component or connection; and
EQ = earthquake effect on the component or connection.

Resistance factors for the seismic limit state are as for conventional design except that for reinforced concrete columns in flexure, the resistance factors decrease from 0.90 to 0.50 as the axial load on the column increases from 0 to $0.2f'_cA_g$. This requirement reflects the reduction in ductility capacity of the column as the axial load increases.

These resistance factors should not be confused with overstrength factors, which are used in seismic design to calculate likely upper bounds on member strength so as to ensure that ductile action occurs at preferred locations. For example, flexural plastic hinging is preferred in a column before inelastic shear occurs, mainly because one is ductile and the other tends to be brittle. In order to assure that flexural hinging will occur before the limit state in shear is reached, the flexural overstrength of the column must be calculated. Overstrength factors for concrete and steel sections are specified at 1.30 and 1.25 respectively; both are applied to the nominal resistance of the column [Art 3.10.9.4.3, (1)].

AASHTO SEISMIC DESIGN PROVISIONS

Characterization of the Seismic Hazard

Historically, the United States and many other countries have used a single-level earthquake to seismically design bridges and other structures. This earthquake, usually called the design earthquake, is intended to represent the largest earthquake that could reasonably be expected to occur during the life of the bridge. Inherent in such a statement is the notion of 'uniform risk' since the design level is intended to be an earthquake with the same probability of exceedance from one region to another, rather than using the maximum historical event for each region, which may have a very low probability of occurrence.

The *Standard Specifications for Highway Bridges* in the United States (*2*) adopted this uniform risk approach following the 1989 Loma Prieta earthquake, and uses a level of hazard that has a 10% probability of exceedance (PE) in a 50-year exposure period. This corresponds to an event with a return period of about 500 years (actually 475 years).

With the development of the *LRFD Specifications* (*1*) the exposure period was adjusted to 75 years, corresponding to the assumed life of a normal highway bridge in the *LRFD Specifications*. Accordingly the probability of exceedance was raised to 15%, so as to maintain, approximately, the same return period (500 years).

Seismic Design Philosophy

At the same time as adopting this uniform risk approach, a corresponding set of performance expectations were included in the philosophy of the AASHTO *Standard Specifications* (*2*) and carried over to the *LRFD Specifications* (*1*). These are given in Art. C3.10.1 of the *LRFD Specifications* and summarized below:

- Small to moderate earthquakes should be resisted within the elastic range, without significant damage.
- Realistic seismic ground motion intensities and forces are used in the design procedures.
- Exposure to shaking from large earthquakes should not cause collapse of all or part of the bridge. Where possible, damage that does occur should be readily detectable and accessible for inspection and repair.

A set of basic concepts for seismic design follows from this philosophy, and these are summarized below:

- Hazard to life to be minimized.
- Bridges may suffer damage but must have a low probability of collapse.
- Function of essential bridges to be maintained.
- Ground motions used in design should have a low probability of being exceeded in the normal lifetime of the bridge.

Characterized by a lack of specificity, these criteria were nevertheless a significant advance over the then prevailing requirements for seismic design.

Seismic Design Methodology

The seismic design provisions in the LRFD Specification and the steps involved in completing the seismic design of a bridge are illustrated in Figure 1. The provisions apply to bridges of conventional slab, slab and girder, box girder and truss superstructure construction with spans not exceeding 150 m. For other types of construction and spans exceeding 150 m, the owner shall determine site-specific provisions to be used for design. The potential for soil liquefaction and slope failure must also be considered.

Despite the fact that inelastic response is expected during a moderate to large earthquake, bridges are designed using elastic methods of analysis. But since yielding will limit the maximum force demands on components and connections, calculated elastic effects are reduced using response modification factors, to obtain design forces. Displacement demands, however, are not reduced by the *R*-factors, and design support lengths must satisfy the greater of these demands and a set of specified minima, to prevent girder unseating.

Minimum design requirements are specified with respect to the seismic zone in which the bridge is located and these range from no analysis and default minima in the lowest zones of the United States to rigorous analysis and detailed design requirements in the highest zones. Regardless of zone, single span bridges need only meet default minima.

The design process has three major steps:

1. Determine the seismic demand;
2. Analyze the bridge; and
3. Design components and connections such that their capacity exceeds the demand.

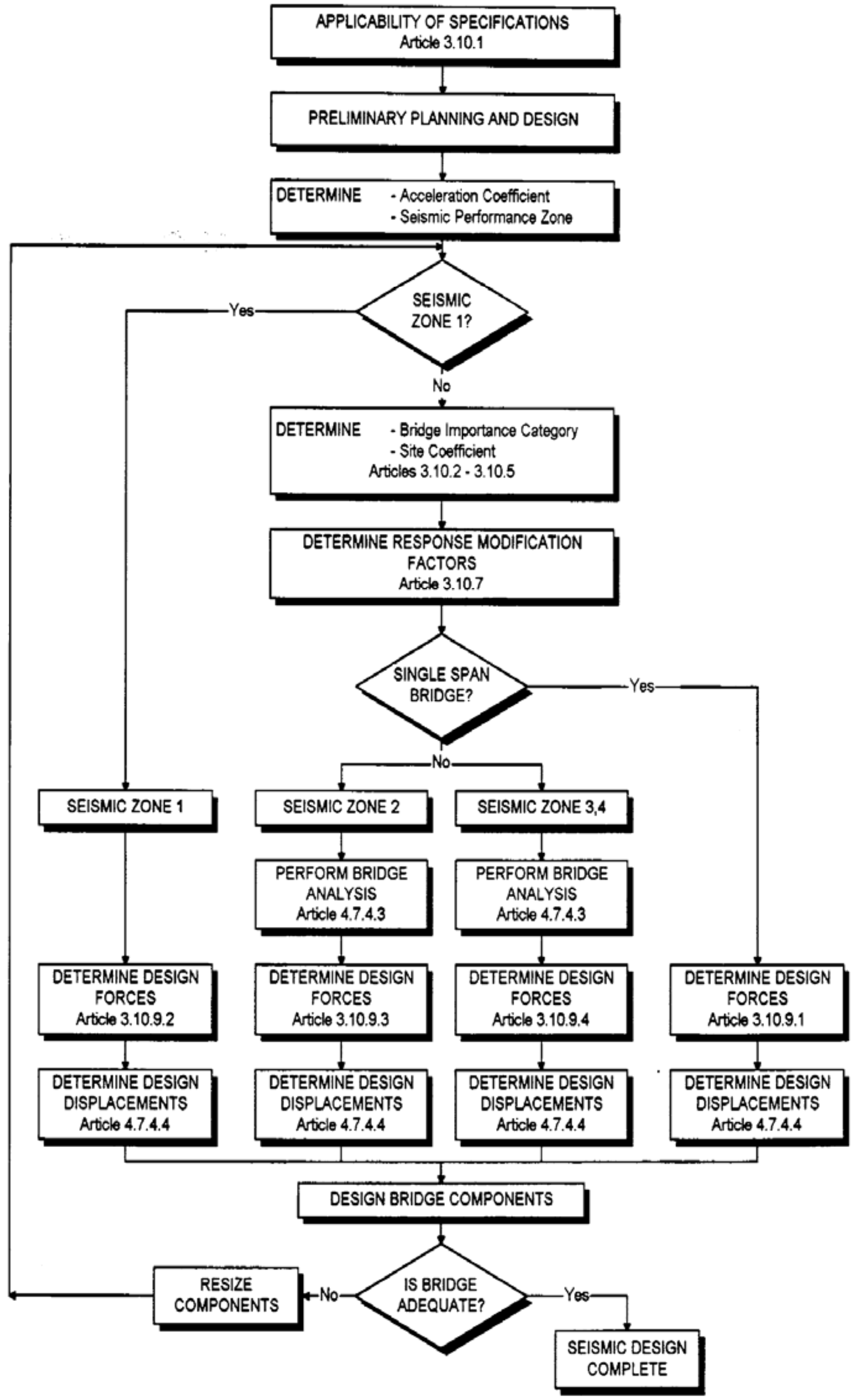

FIGURE 1 AASHTO LRFD bridge design specifications: seismic design procedure (*1*).

Exceptions are permitted, such as for single span bridges and all bridges in Seismic Zone 1, but generally the seismic design process follows the above steps, which are further described below.

Step 1: Loads

The earthquake load (F) to be applied horizontally to the bridge, is proportional to the weight of the bridge (W) and is given by:

$$F = C_S W$$

where

C_S = seismic response coefficient,
= $1.2\, A\, S / T^{2/3} \leq 2.5\, A$,
A = acceleration coefficient,
S = site coefficient, and
T = period (s) of the bridge in the direction under consideration.

The acceleration coefficient, A, represents the peak ground acceleration that is likely to occur within the 500-year return period described above. Values of A are read from maps developed by the United States Geological Survey (USGS), but special studies are required if:

- The site is in the near-field of an active fault, or
- Long-duration earthquakes are expected in the region, or
- The bridge is of particular strategic importance to the region that earthquakes with longer return periods be considered.

The above acceleration coefficients are for rock sites and a site coefficient, S, is used to include the amplification effects of overlying soils, if any. Four soil profile types are defined, and these are:

1. Rock and shallow deposits of stiff soils (Profile Type I);
2. Stiff cohesive or deep cohesionless soils (Profile Type II);
3. Soft to medium-stiff clays and sands (Profile Type III); and
4. Deep deposits of soft clays and silts (Profile Type IV).

Table 1 gives values of the site coefficients for these four soil types.

TABLE 1 Site Coefficient, *S*

Soil profile type	**I**	**II**	**III**	**IV**
Site coefficient, S	1.0	1.2	1.5	2.0

Step 2: Analysis

Every bridge is assigned to a seismic performance zone, which reflects the variation in seismic hazard across the country. These zones are used to permit different requirements for methods of analysis, minimum support lengths, column design details and foundation and abutment design procedures. As shown in Table 2, they are determined by the acceleration coefficient at the bridge site.

Bridges in Zone 1 need not be analyzed for seismic effects and this analysis step may be omitted. Similarly single span bridges do not need to be analyzed, regardless of their seismic zone. Nevertheless minimum design requirements are specified for these bridges, and these are discussed in the next step.

Four methods of analysis for multi-span bridges are specified, two are single-mode methods and two are multi-mode methods. In increasing order of rigor, they are:

- Uniform load elastic method,
- Single-mode elastic method,
- Multimode elastic method, and
- Time history method (elastic or inelastic).

Minimum analysis requirements are also specified based on seismic zone, bridge regularity and importance. In general, more rigorous analysis is required for bridges in higher seismic zones, than for those that are 'irregular', or of greater importance.

Regularity is a function of the number of spans and the distribution of weight and stiffness. Regular bridges have less than seven spans; no abrupt or unusual changes in weight, stiffness, or geometry, and no large changes in these parameters from span-to-span or support-to-support, abutments excluded. Any bridge which is not regular is irregular.

For the purpose of identifying bridge importance, three types of bridges are defined as follows:

1. Critical bridges,
2. Essential bridges, and
3. Other bridges.

Essential bridges are those that should, as a minimum, be open to emergency vehicles and for security/defense purposes immediately after the design earthquake (the 500-year event).

TABLE 2 Seismic Performance Zones

Acceleration Coefficient	Seismic Zone
$A \leq 0.09$	1
$0.09 < A \leq 0.19$	2
$0.10 < A \leq 0.29$	3
$0.29 < A$	4

Some bridges, however, must remain open to all traffic and be usable by emergency vehicles and for security/defense purposes immediately after a very large earthquake, e.g., a 2,500-year event. These bridges are critical structures. Bridges that are neither essential nor critical are classified as other.

Step 3: Design

Design *forces* are obtained from the results of the elastic analyses in the previous step. They are generally set equal to the peak elastic force, as calculated in the analysis, divided by a response modification factor (also called an *R*-factor). These factors are based on three properties: first the ability of the component or connection to deform in a ductile manner once yield is reached; second the importance of the bridge; and third the degree of redundancy in the bridge substructure.

The higher the ductility capacity of an element, the higher the *R*-factor, and the more important the bridge, the lower the factor. A high value for *R* implies that significant yield will occur during the design earthquake with corresponding large plastic deformations in the hinge zones of columns and wall piers. These deformations will likely cause concrete cover to spall and permanent residual strains to occur in steel elements that have yielded. Whereas collapse will be avoided, the bridge may need to be closed until repairs can be completed. Such action may be in conflict with performance expectations for essential and critical bridges, and a limitation is therefore imposed on the *R*-factors for these bridges.

Redundant load paths in bridge substructures are considered to be good engineering design and thus non-redundant systems are penalized in the specification. For example, single column substructures in other bridges have *R*-factors that are 60% of the factors for multi-column bents. Representative *R*-factors are given in Table 3.

Where an elastic analysis is not required, such as for single span bridges and bridges in Seismic Zone 1, minimum design forces and details are specified. In most cases these refer to the design forces for the connections between the superstructure and abutments (for a single-span bridge) and at intermediate supports (for multispan bridges). Minimum design details generally include minimum longitudinal and reinforcement ratios in concrete columns together with minimum confining steel requirements.

Design *displacements* are the greater of the calculated elastic displacements (not reduced by an *R*-factor) and the minimum support length (*N*) calculated as below:

TABLE 3 Response Modification Factors, *R*

Substructure Type	**Importance Category**		
	Critical	**Essential**	**Other**
Wall-type piers	1.5	1.5	2.0
Single-column piers	1.5	2.0	3.0
Multi-column piers	1.5	3.5	5.0
Superstructure connections	0.8	0.8	0.8
Column connections	1.0	1.0	1.0

$$N = (200 + 1.67\,L + 6.67\,H)(1 + 125\,S^2\,10^{-6})$$

where

N = minimum support length measured normal to the bearing centerline (mm);
L = length of bridge deck adjacent to seat at expansion joint or end of deck (m);
H = effective column height adjacent to seat (m); and
S = angle of skew measured from a line normal to the span (deg).

The value for N calculated above is applicable to bridges in Seismic Zones 1 and 2, but for Zones 3 and 4, N is increased by a factor of 1.5

Observations on the LRFD Methodology for Seismic Design

The assumption is made in single-level design (and retrofit) that if performance at the design event is satisfactory, it will be satisfactory at all other levels, both smaller and larger. Such an assumption is generally not true, as seen in recent earthquakes in California, Costa Rica, Japan, Turkey, and Taiwan. It would be true for smaller events if elastic performance was required at the design event, and it may also be true for larger events, if the design event was sufficiently large and a generous degree of conservatism used in the design. But under the design event, inelastic performance (damage) is explicitly intended (in most bridges), and provided life safety is preserved, the consequential restrictions on access are considered to be tolerable.

However, these restrictions become unacceptable, if they were to occur on a more frequent basis such as during a smaller earthquake. Since this is a nonlinear problem, assurances regarding performance during smaller earthquakes cannot be obtained simply by scaling performance at the design event and thus explicit design (or at least a design check) should be made at this level, to gain this assurance.

Similarly, performance during a larger event cannot be estimated by scaling upwards and relying on reserve strength. Without explicit quantification, this approach is unreliable because it is based on engineering judgment and an experience database that is thin and largely unverified, especially in low to moderate seismic zones such as in the central and eastern United States (CEUS).

The argument is thus made, that to avoid adverse performance, such as seen in Loma Prieta, Northridge, Kobe, and Taiwan, explicit consideration of bridge performance during at least two levels of earthquake (and perhaps more) should be undertaken. Furthermore, the expected level of performance during these earthquakes should be stated with a greater level of specificity than has been the case in the past, and assurances given that these performance levels will be met. This argument leads to the consideration of performance-based engineering for the seismic design and retrofit of bridges.

PERFORMANCE-BASED ENGINEERING AND DESIGN

Performance-based engineering (PBE) has been defined as the selection of design criteria and structural systems (layout, proportioning and detailing), and the assurance and control of construction quality and long-term maintenance, such that the structure will not be damaged beyond certain limiting states or other usefulness limits, at specified levels of ground motion and with defined levels of reliability (*3*). This definition has been paraphrased from that developed for buildings in the Vision 2000 Project of the Structural Engineers Association of California, where PBE was explored and its potential for improving the seismic performance of new buildings was clearly demonstrated.

Application of the design phase of PBE, called performance-based design (PBD), requires two fundamental issues be addressed:

1. Selecting the specified ground motions (hazard levels) and the corresponding damage states (performance objectives), and
2. Developing methods of evaluation for the verification of damage states and performance objectives.

The first of these issues requires that the ground motion be determined with a high degree of confidence (i.e. the seismic design coefficient is known within acceptable limits for a bridge with a fundamental period T, and given site class), which may be true for the west coast of the United States but is not necessarily so for the CEUS.

The second issue implies a level of sophistication in analysis that may be difficult to implement with ease and reliability. Furthermore, the relationship between some damage states and performance objectives (such as crack width to lane closures) is not known with any certainty, except perhaps for typical bridges in California.

Performance Objectives

If dual events are considered and two bridge types identified, performance objectives may be formatted in a 2 x 2 matrix with the rows assigned to the earthquake level and the columns to bridge type. Four performance levels (PL) and four damage levels (DL) corresponding to the two earthquake levels and two bridge types may then be specified but, in practice, duplication among the PLs and DLs is common and the number of separate and distinct levels may be as few as two. Recent efforts by California Department of Transportation (Caltrans) and AASHTO to implement such a set of objectives are described in the following sections.

Caltrans Experience

Immediately following the 1989 Loma Prieta earthquake, Caltrans moved towards dual-level PBD. Endorsed by the Caltrans Seismic Advisory Board and an independent review by the Applied Technology Council (*4*), these criteria were customized to the seismic hazard in California and Caltrans prevailing practice. Two bridge classes were identified (ordinary and important) and the rare and frequent earthquakes identified as the safety evaluation earthquake (SEE) and the functional evaluation earthquake (FEE) respectively. The adopted criteria are shown in Table 4.

The SEE is obtained deterministically by the California Division of Mines and Geology (CDMG) for a bridge site and is called the maximum considered earthquake (MCE). In some circumstances this rare earthquake is determined probabilistically, using a 1,000–2,000-year return period. The functional evaluation event (the frequent earthquake) is also a probabilistic event with about a 40% probability of exceedance within the life of the bridge (about a 200-year event).

TABLE 4 Caltrans Performance Criteria

Earthquake	Bridge Type 1 (Ordinary Bridge)	Bridge Type 2 (Important Bridge)
Functional Evaluation Earthquake	PL2	PL2
(Frequent Earthquake)	DL2	DL3
Safety Evaluation Earthquake	PL1	PL2
(Rare Earthquake)	DL1	DL2

Where PL1 is Performance Level 1, defined as limited access being possible in days; full access within months; PL2 is Performance Level 2, defined as immediate and full access to normal traffic almost immediately after the earthquake; DL1 is Damage Level 1, defined as significant damage which may result in closure but not collapse; DL2 is Damage Level 2, defined as repairable damage which may be executed with minimum loss of functionality; DL3 is Damage Level 3, defined as minimal damage with essentially elastic performance.

An important bridge is one satisfying any of the following:

- Would create a major economic impact if closed for restoration of functionality,
- Required to provide secondary life safety, or
- Designated in a local emergency response plan as critical.

An ordinary bridge is any bridge not classified as important. It is noted that for important bridges, the same performance level is required for both earthquakes, but greater damage is tolerated for the rare earthquake (SEE) than for the frequent event (FEE). For ordinary bridges, less stringent performance is required for the SEE than for the FEE and a greater level of damage is also tolerated. There is a similar differential between the performance and damage levels for ordinary and important bridges.

AASHTO EXPERIENCE

In response to growing concern about the adequacy of the LRFD specifications for seismic design, AASHTO recently sponsored the development of a new set of LRFD seismic design specifications (5). Conducted by the National Cooperative Highway Research Program (NCHRP 12-49) of the Transportation Research Board, the work was undertaken by a Joint Venture between the Applied Technology Council (ATC) and the Multidisciplinary Center for Earthquake Engineering Research (MCEER). As part of this effort, a dual level, performance-based design philosophy was developed with the intent of nationwide applicability. The proposed performance criteria are similar in principle to those being used by Caltrans except that the frequent and rare events are both probabilistic in nature (100- and 2,500-year events) and the bridge types are based only loosely on importance. Minimum performance expectations are used instead.

Two performance levels are again used and three damage levels. Performance levels PL1 and 2 are defined as disrupted and immediate access respectively. Damage levels DL1, 2, and 3 are defined are significant, minimal, and none. Table 5 summarizes these criteria.

In addition to the above criteria, the new recommendations greatly expand the number of evaluation methods (for demand as well as capacity) to provide more realistic methods for the assessment of damage states and verification of performance objectives. Traditional methods are force-based using modified forces from elastic models (i.e. the *R*-factor methods) whereas the new methods are displacement-based and include the following (*6*, *7*):

- Capacity spectrum method, in which demand and capacity evaluation are combined in a single procedure. Method is restricted to very regular structures, which can be modeled as single degree-of-freedom systems; is the basis of the AASHTO guide specification for isolated bridges (*8*).
- Nonlinear static displacement capacity verification methods (pushover analysis), in which the displacement capacities of individual bridge substructures are determined from lateral load-displacement analyses taking into account the nonlinear behavior of their components.
- Nonlinear dynamic analysis methods, in which force and displacement demands are found from step-by-step time-history analyses using ground motion records and taking into account the nonlinear behavior of various bridge components.

In June 2002, the AASHTO Bridge and Structures Subcommittee deferred adoption of these recommendations as a new specification, and asked that the selection of 2,500 years for the return period of the rare earthquake (Table 5) be revisited. This revision is currently being undertaken by AASHTO through NCHRP (*9*).

SEISMIC RETROFITTING OF HIGHWAY BRIDGES

PBD can also be applied to the retrofit of highway bridges and the 1995 FHWA *Manual for Seismic Retrofitting* (*10*) has recently been revised to include these principles (*11*).

TABLE 5 Proposed AASHTO Performance Criteria (*5*)

Earthquake	**Bridge Type 1 (Life safety required)**	**Bridge Type 2 (Full operation required)**
100-year earthquake	PL2	PL2
(Frequent earthquake)	DL2	DL3
2,500-year earthquake	PL1	PL2
(Rare earthquake)	DL1	DL2

Where PL1, 2 are Performance Levels 1 and 2, and defined as disrupted access and immediate respectively; DL1, 2, 3 are Damage Levels 1, 2 and 3 and defined as significant sustained damage, minimal, and none.

The new manual is in two parts, one for bridges and the other for retaining structures, slopes, tunnels, culverts and roadways. Whereas Part 1 maintains the basic format of the retrofitting process described in the 1995 manual, major changes have been made to include advances in earthquake engineering such as PBD, field experience with retrofitting highway bridges, and the performance of bridges in recent earthquakes.

Performance criteria are given for two earthquake ground motions with return periods of 100 and 1000 years (Table 6). As for new bridges, higher performance levels are required for earthquakes with the shorter return period. Criteria are recommended according to bridge importance and anticipated service life, with more rigorous performance being required for important, relatively new bridges, and a lesser performance for standard bridges nearing the end of their useful life. Note that damage levels are not used in setting criteria for retrofitted bridges. Minimum recommendations are made for screening, evaluation and retrofitting according to an assigned Seismic Retrofit Category. These categories are determined by the PL required and the seismic hazard at the site. Bridges in Category A need not be retrofitted whereas those in Category B need not be evaluated in detail provided certain requirements are satisfied. Bridges in Categories C and D require more rigorous evaluation and retrofitting, as required. Various retrofit strategies are described including a range of retrofit measures such as restrainers, seat extensions, column jackets, footing overlays, and soil remediation.

CONCLUSIONS

For almost 70 years, bridges in the United States have been designed using a working stress design specification, updated at regular intervals to reflect recent research results and field experience. Nevertheless, growing concern with inconsistencies in the specifications, and the fact that variability in the loads and materials were not explicitly addressed, an entirely new specification was developed in the early 1990s based on LRFD. The *LRFD Bridge Design Specifications* is a limit state based specification and is a considerable step forward in the design

TABLE 6 Proposed FHWA Performance Criteria for Retrofitting (*11*)

Earthquake	**Bridge Type 1 Importance: Standard**			**Bridge Type 2 Importance: Essential**		
	ASL 1	**ASL2**	**ASL3**	**ASL1**	**ASL 2**	**ASL 3**
100-year Earthquake						
(Lower Level earthquake)	PL0	PL3	PL3	PL0	PL3	PL3
1000-year Earthquake	PL0	PL1	PL1	PL0	PL1	PL2
(Upper Level earthquake)						

Where PL0, 1, 2, 3 are required Performance Levels:0 (no minimum performance level required, 1 (significant disruption to service), 2 (operational for emergency vehicles), and 3 (immediate and full access); ASL1, 2, 3 are ranges of Anticipated Service Life: 1 (0–5 years remaining), 2 (16–0 years remaining), and 3 (more than 50 years remaining).

and analysis of highway bridges. Advantages include superior serviceability, enhanced long-term maintainability and more uniform levels of safety. Seismic design provisions are included and the selection of load and resistance factors for all extreme events has been rationalized.

However the performance of bridges in recent earthquakes has been less than satisfactory and although life safety is generally preserved, public frustration with closures and limited access has been widespread. As a consequence, a major review of performance criteria for bridges has been undertaken and a move towards performance-based, multi-level seismic design has begun in the United States. The process has required the selection of appropriate levels for the design ground motions (i.e., return periods), corresponding performance objectives for each level, and the development of reliable analytical methods for the verification of these objectives.

Dual level criteria and improved techniques for evaluation, using modified force-based methods or true displacement-based methods, have now been used in California for a decade and applied explicitly to several major structures, and implicitly to hundreds of 'ordinary' bridges. Advantages are expected to be improved performance at all levels of ground shaking, from small frequent events, to large rare events. Application to other seismic regions of the United States seems practical, despite the higher levels of uncertainty with respect to the ground motions and the lack of damage data for typical bridge configurations found in the central and eastern United States. This paper has reviewed the progress made towards implementing performance-based seismic design in the United States, as well as note some of the progress made towards applying PBD to seismic retrofitting.

ACKNOWLEDGMENTS

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The ongoing development of a performance-based seismic design methodology for bridges is the work of many people at the present time. Major contributions have been made by Caltrans, the FHWA, and AASHTO. For example, the AASHTO-funded NCHRP Project 12-49 is a landmark achievement, and the authors recognize the major contributions of Ian Friedland (FHWA, formerly MCEER and ATC) and Christopher Rojahn (ATC) the co-principal investigators for this project.

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50 Years of Tunnels on the Interstate Highway System

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The United States has approximately 72 mi (116 km) of road tunnels. About 29 mi (47 km) of these tunnels are along the Interstate highway system. The tunnels along the U.S. Interstate highway system have been constructed over many years utilizing a variety of construction methods, equipment, and contract mechanisms. Construction methods varied primarily according to the subsurface geologic conditions encountered, current tunneling methods, and economic considerations. The majority of tunnels were constructed using the design–bid–build contracting methods. However, some were constructing using design–build methods especially in the recent past. These tunnels were required to go under a number of natural obstacles, most often rivers and mountains. Occasionally, depressing highways underground in congested urban areas have presented additional opportunities for surface reclamation or air rights developments. The shortest Interstate highway tunnel is the Reverse Curve Tunnel on Interstate 70 (I-70) approximately 600 ft (183 m) long. The longest Interstate highway tunnel is the Brooklyn Battery Tunnel on I-478 approximately 1.7 mi (2.8 km) long. The Brooklyn Battery Tunnel is also the longest subaqueous tunnel on the Interstate highway system. The highest highway Interstate tunnel in the United States is the Eisenhower Tunnel on I-70 at a maximum elevation of 11,152 ft (3,401 m). This paper identifies several tunnels that have been constructed along the Interstate highway system over the past 50 years. Specific examples of the various tunneling methods and contracting practices used are highlighted.

INTRODUCTION

Numerous tunnels have been constructed for the Interstate highway system over the past 50 years. The United States has approximately 72 mi (116 km) of road tunnels. About 29 mi (47 km) of these tunnels are along the Interstate highway system. The tunnels along the U.S. Interstate highway system have been constructed over many years utilizing a variety of construction methods, equipment, and contract mechanisms. Construction methods varied primarily according to the subsurface geologic conditions encountered, current tunneling methods, and economic considerations. The majority of tunnels were constructed using the design–bid–build contracting methods. However, some were constructing using design–build methods especially in the recent past. These tunnels were required to go under a number of natural obstacles, most often rivers and mountains. Occasionally, depressing highways underground in congested urban areas have presented additional opportunities for surface reclamation or air rights developments. The shortest Interstate highway tunnel is the Reverse Curve Tunnel on I-70 approximately 600 ft (183 m) long. The longest Interstate highway tunnel is the Brooklyn Battery Tunnel on I-478 approximately 1.7 mi (2.8 km) long. The Brooklyn Battery Tunnel is also the longest subaqueous tunnel on the Interstate highway system. The highest highway Interstate tunnel in the United States is the Eisenhower Tunnel on I-70 at a maximum elevation of 11,152 ft (3,401 m). Several tunnels that have been constructed along the Interstate highway system over the past 50 years are discussed below. Specific examples of the various tunneling methods and contracting practices used are highlighted.

ROADWAY TUNNELS IN THE UNITED STATES

Most of the roadway tunnels in the United States are not on Interstate highways (Table 1). But it's important to note that most if not all roadway tunnels are constructed using the same methods and are designed to the same standards as Interstate highway tunnels. Listed in this paper are 36 tunnels constructed on the Interstate highway system (Table 2). The Interstates that have tunnels include I-10, I-64, I-70, I-71, I-76, I-77, I-78, I-90, I-94, I-95, I-264, I-279, I-376, I-478, I-495, I-540, I-579, I-664, I-895, and H-3.

TUNNEL CONSTRUCTION METHODS

Tunnels are constructed through soils (softground) and bedrock (rock). Typically, softground tunnels used to be constructed underneath rivers using compressed-air mined tunneling methods like what was used for New York City's vehicular crossings of the Hudson River (Figure 1). These methods took long construction times and were fairly hazardous to the workers. More modern methods for subaqueous crossings include immersed tube tunneling and earth pressure balance tunnel boring machine, precast concrete segmental lining methods. A very innovative multidrift method was used for the Mt. Baker Ridge Tunnel as discussed elsewhere in this paper. Cut-and-cover tunneling methods are often used for tunnels in soil as well. Often these methods are used in congested urban environments requiring pre-installation of the sidewalls (slurry walls) and roadway decking prior to underground excavation. Rock tunneling methods generally include drill-and-blast tunneling methods using explosives and rock reinforcement methods (rock bolts, straps, mesh, shotcrete, etc.) for initial support. Mechanized excavation methods of rock also exist including the use of roadheaders and tunnel boring machines when geologic conditions permit and size/shape considerations are met. A final concrete lining with membrane waterproofing is commonly used after the excavation has been made. Examples of these tunneling methods are given below.

Drill-and-Blast Tunneling Through Rock

Five recent highway tunnel projects in rock were completed in the 1990s. The tunnels on these projects include the Reverse Curve and Hanging Lake Tunnels in Colorado, the Trans-Koolau Tunnel in Hawaii, the Cumberland Gap Tunnel on the border of Kentucky and Tennessee, and the Bunyard Tunnel in Arkansas. These tunnels had some common elements reflecting emerging trends in underground construction at the end of the 20th century. These included detailed evaluations of geologic conditions through borings, in situ testing, and exploratory tunnels; advanced methods of rock reinforcement, lining and waterproofing; and advanced contracting practices recommended by the ASCE (*1, 2*) and other agencies. These practices generally included differing site conditions clauses, escrow bid documents, geotechnical baseline reports, and dispute review boards.

TABLE 1 Approximate Length of Roadway Tunnels in the United States

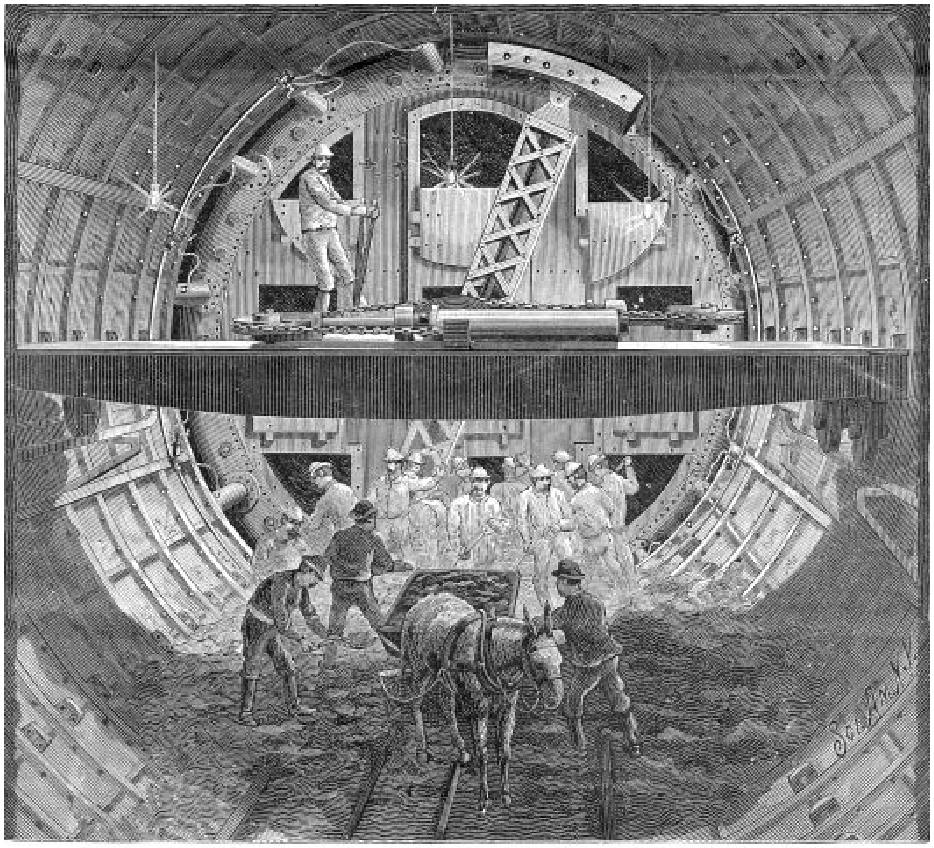
States That Have Roadway Tunnels	Approximate Total Length of Roadway Tunnels (miles)*	Approximate Total Length of Interstate Highway Tunnels (miles)*
Alaska	2.5	
Alabama	1.2	0.6
Arizona	0.5	0.5
Arkansas	0.3	0.3
California	11.2	
Colorado	2.7	2.5
District of Columbia	1.1	
Delaware	0.3	0.3
Florida	0.6	
Hawaii	1.8	1.1
Illinois	4.3	0.3
Kentucky	0.4	
Massachusetts	5.5	2.9
Maryland	2.8	2.8
Michigan	1.1	
Minnesota	0.3	0.3
Missouri	0.3	
Montana	0.8	
Nevada	0.3	
New Jersey	1.2	0.8
New York	9.4	3.7
North Carolina	3.5	
Ohio	0.5	0.2
Oregon	0.7	0.2
Pennsylvania	8.4	7.2
Tennessee	0.4	
Texas	0.7	
Utah	1.0	
Virginia	7.4	4.3
West Virginia	0.5	0.5
Washington	1.5	1.5
Total	73.1	29.1

*1 mi = 1.6 km

TABLE 2 Interstate Highway Tunnels in the United States

Tunnel	State	Approximate Length (miles)*	Interstate
Wallace	Alabama	0.6	I-10
Hance Deck Park (Papago Freeway)	Arizona	0.5	I-10
Bunyard (Bobby Hopper)	Arkansas	0.3	I-540
Eisenhower Memorial	Colorado	1.7	I-70
Reverse Curve	Colorado	0.1	I-70
Hanging Lake	Colorado	0.7	I-70
Penn's Landing	Delaware	0.3	I-95
Hospital Rock	Hawaii	0.1	H-3
Trans-Koolau (Tetsuo Harano)	Hawaii	1.0	H-3
Kinzie Street	Illinois	0.3	I-90/94
Ted Williams/I-90 Extension	Massachusetts	2.6	I-90
Fort Point Channel	Massachusetts	0.3	I-90
Baltimore Harbor	Maryland	1.4	I-895
Fort McHenry	Maryland	1.4	I-95
Lowry Hill	Minnesota	0.3	I-94
Carlin	Nevada	0.3	I-80
Brooklyn Battery	New York	1.7	I-478
Holland	New York/New Jersey	1.6	I-78
Lytle	Ohio	0.2	I-71
Tooth Rock	Oregon	0.2	I-84
Tuscarora Mountain	Pennsylvania	1.0	I-76
Kittatinny Mountain	Pennsylvania	0.9	I-76
Lehigh	Pennsylvania	0.8	I-76
Blue Mountain	Pennsylvania	0.8	I-76
Liberty	Pennsylvania	1.1	I-579
Squirrel Hill	Pennsylvania	0.8	I-376
Fort Pitt	Pennsylvania	0.7	I-279
Allegheny	Pennsylvania	1.1	I-76
Big Walker Mountain	Virginia	0.8	I-77
Monitor Merrimac Memorial	Virginia	0.9	I-664
Hampton Roads	Virginia	1.4	I-64
Downtown (1st Norfolk Portsmouth)	Virginia	0.7	I-264
East River Mountain	Virginia/W. Virginia	1.0	I-77
Mount Baker	Washington	0.6	I-90
Martin L. King Lid	Washington	0.4	I-90
First Hill Lid	Washington	0.5	I-90

* 1 mi = 1.6 km



THE HUDSON RIVER TUNNEL—THE "ERECTOR" AND THE HYDRAULIC SHIELD.

FIGURE 1 Early methods of tunneling across rivers.

I-70 Glenwood Canyon Project

In Glenwood Canyon, Colorado, 12 mi (19.4 km) of U.S. Route 6 was converted into a four-lane section of I-70 (*3*). The project included 5.7 lane mi (9.2 km) of viaduct, 1.6 mi (2.5 km) of tunnels, and 17.8 lane mi (20.7 km) of roadway including grade-separated sections. The project cost approximately $350 million. The Colorado Department of Highways owns the project. The FHWA funded approximately 92% of the project. The two tunnel projects consisted of the westbound Reverse Curve Tunnel, a 600-ft (183 m), two-lane bore through a nose of rock, and the Hanging lake Tunnel, twin 40-ft (12.2 m) diameter, 3,500-ft-long (1,067 m) mined tunnels, and a 500-ft-long (152 m) cut-and-cover tunnel and ventilation and control building. The project is located approximately 149 mi (226 km) west of Denver.

Interstate H-3 Project

The Interstate Route H-3 Project in Hawaii consisted of a 10-mi-long (16.1-km) segment of new highway that traverses the Koolau mountain range on the island of Oahu (Figure 2). The project included tunneling through this mountain range as well as several long viaducts, hillside road cuts, and a short cut-and-cover tunnel through pristine tropical conditions that had historical and religious significance to local natives. The one-mile-long (1.6 km) twin-bore Trans-Koolau

FIGURE 2 Waterproofing membrane installation in Interstate H-3 Trans Koolau Tunnel.

Tunnel has two lanes in each direction and a roadway width of 38 ft (11.6 m) including shoulders. The Hawaii Department of Transportation (DOT) owns the $1 billion-plus project and the majority of funding came from FHWA.

I-540 Bunyard Tunnel Project

The Bunyard (Bobby Hopper) Tunnel is near Winslow, Arkansas, along I-540 running between Alma and Fayetteville, Arkansas. The 47.9-mi- (77.2-km-) long I-540 project was built along one of the most dangerous highway corridors in America to improve safety and enhance mobility and economic growth. The twin bore tunnel is 0.3 mi (0.5 km) long and utilizes a longitudinal ventilation system. The tunnel was mined out of shale and sandstone using drill-and-blast excavation methods with a top heading and bench. The tunnel is owned by the Arkansas State Highway and Transportation Department.

U. S. Route 25E Cumberland Gap Project

Technically not an Interstate, this project still merits mention in this paper because of similarities to other Interstate highway tunnels that were constructed in the same period. Most notably, this tunnel utilizes longitudinal ventilation, the first application in the United States on any major federally funded highway tunnel project. The decision to allow longitudinal ventilation was largely supported by the extensive research carried out at the West Virginia abandoned Memorial Tunnel by FHWA during the mid-1990s (*4*). Prior to this project being opened, U. S. Highway 25E between Cumberland Gap, Tennessee, and Middlesboro, Kentucky, carried a traffic load of more than 18,000 vehicles per day over a steep, winding, two- and three-lane route that passed over the Cumberland Gap in the Cumberland Gap National Park. The historical nature of the route, coupled with the technical problems of roadway improvements in such difficult terrain, led to relocating the highway into twin two-lane tunnels through Cumberland Mountain. FHWA served as the design and construction manager for the project on behalf of the owner, the U. S. Department of Interior, National Park Service. The twin tunnels are about 4,100 ft (1,250 m) long, with a width of about 40 ft (12.2 m).

Softground Tunneling

Softground (soil) tunneling for highways in the United States has traditionally been used at river and other waterway crossings. Several notable highway tunnels were built in the late 1800s and early 1900s using shields or compressed air (caisson) tunneling methods. Heavy cast iron and steel segments were bolted together to line these tunnels. The Holland, Lincoln, and Brooklyn–Battery Tunnels in New York are typical examples. More recently immersed tube tunneling has been used for this purpose as discussed below. More modern methods of softground mining involve the use of specially designed tunnel boring machines (TBMs) that balance the earth pressure (EPBs) at the tunnel face in a pressurized bulkhead at the front of the machine. The tunnel miners are then in a free air environment behind the bulkhead avoiding the unhealthy aspects of compressed air mining. Metal linings have given way to more economical precast concrete segmental linings. Other innovative tunneling methods have been used for the Interstate highway system such as stacked drifts and tunnel jacking as described below.

I-90 Mount Baker Ridge Tunnel

One of the more innovative tunnels constructed in softground during the last 50 years was the Mount Baker Ridge Tunnel on I-90 in Washington. This 82.7-ft (25.2-m) outside diameter, 63.6-ft (19.4-m) inside diameter, 1,332-ft (406-m-) long tunnel was constructed in 1986 to carry five lanes of traffic and a bicycle–pedestrian lane through a 262-ft (80-m) high ridge along the east side of Seattle (Figure 3). Twenty-four horseshoe-shaped stacked drifts form the circular lining that is a semi-flexible, unreinforced compression ring having a minimum thickness of 4.9 ft (1.5 m). The 9.8-ft (3 m) diameter drifts were excavated with a horseshoe-shaped digger shield through hard glacial clay. After placement of the ring of drifts, the soil core was removed and three decks were constructed for carrying traffic (*5*).

I-90/I-93 Interchange on the Central Artery Tunnel Project (Jacked Tunnels)

The purpose of the Central Artery project in Boston was to cure massive traffic congestion in the heart of the historic city by replacing the elevated Central Artery section (opened in the 1950s) with a new eight- to 10-lane highway running mainly underground. The I-90/I-93 interchange links the Central Artery to the Ted Williams immersed tube tunnel leading to Logan Airport.

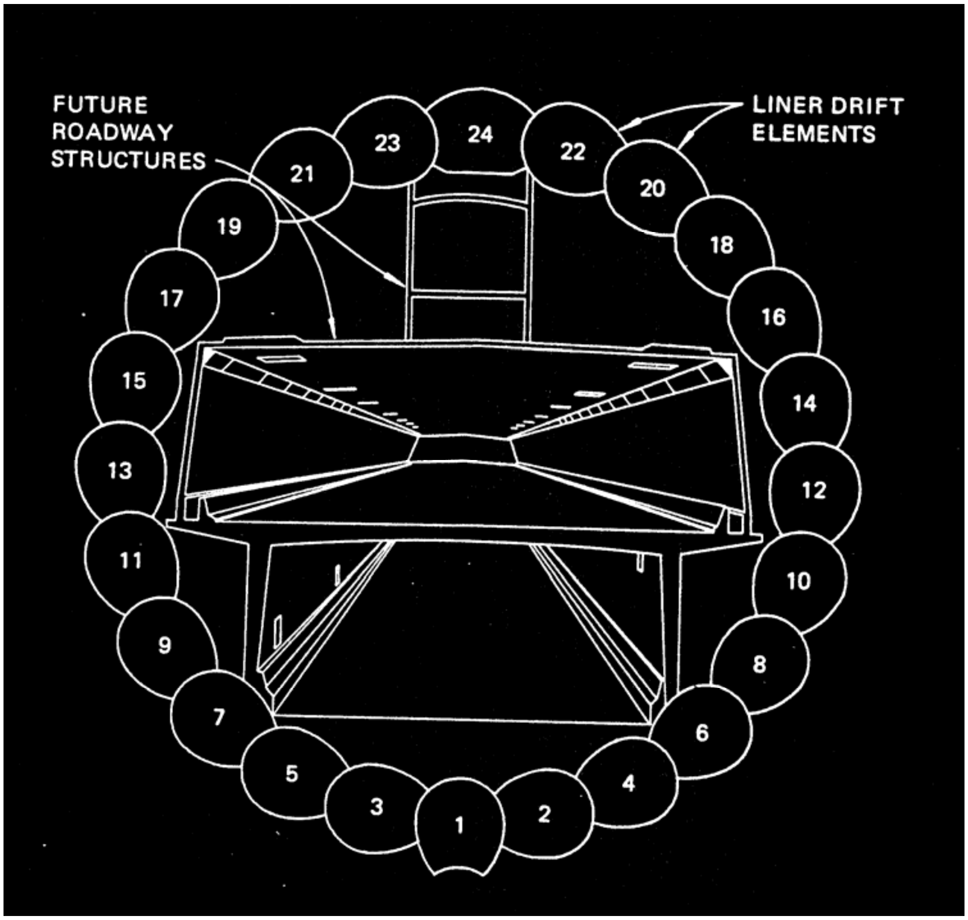


FIGURE 3 Mount Baker Ridge Tunnel.

This multilevel Interstate interchange is one of the most complex sections of the project and involves three crossings beneath eight active railway tracks providing Amtrak and MBTA service into South Station. Jacked tunnels form the deepest section of the multilevel Interstate interchange that also includes an open boat section, at-grade roadways, several levels of viaduct that tie into the adjacent roadways, and immersed tube tunnels. Early design concepts would have made interruption of rail traffic inevitable. Alternatively, a tunnel jacking solution was developed that allowed construction without interrupting train service, saving millions of dollars of railway operating revenues (Figure 4). This part of the project included three jacked tunnel sections, three jacking pits where the to-be-jacked tunnels were constructed on-site and jacked from, and 1,200 ft (366 m) of cut-and-cover tunnels as approaches to the jacked tunnel segments. The jacked tunnels are the largest and most complex ever constructed in the world. The largest of the three was 79 ft (24 m) wide by 36 ft (11 m) high by 370 ft (113 m) long. The deepest tunnel section had 24 ft (7.3 m) of cover. Combinations of dewatering and ground treatment methods including ground freezing and jet grouting were used to control and minimize settlement during excavation. Soil conditions consisted mostly of reclaimed land, Boston Blue Clay, thin layers of fine sand, and glacial till overlying Cambridge Argillite bedrock.

FIGURE 4 Central Artery Tunnel jacking pit.

Immersed Tube Tunneling

Immersed tube tunneling consists of constructing huge steel or concrete sections that can be fabricated away from the project site, floated to the location by tugboats, and then sunk into place by the addition of ballast. A trench is pre-excavated in the waterway bottom along the tunnel alignment and is backfilled after the tunnel segments are in place. After the tunnel segments are placed next to each other and sealed, the tubes are dewatered and outfitted for service. Notable U. S. immersed tube highway tunnel projects in the past 50 years include the I-64 Hampton Roads, I-95 Fort McHenry, and the I-90/I-93 Central Artery Tunnels. These tunnels used construction methods that were very similar to one another, as described below.

The I-95 Fort McHenry Tunnel joins the Baltimore Harbor Tunnel (I-895) and the Francis Scott Key Bridge (I-695). The tunnel is 1.4-mi (2.2-km) long and eight lanes wide. Construction consisted of dredging a trench with a bottom width of 180 ft (54.9 m) across the harbor bottom, excavating a trench at each shore area near the harbor shoreline, transporting 3.5 million cubic yards of spoil material by underwater slurry pipeline to the Canton–Seagirt Disposal Site a couple of miles away in East Baltimore, fabricating the 32 individual 320-ft- (97.6-m-) long tunnel sections at Wiley Manufacturing at Port Deposit, Maryland, concreting and sinking the tunnel sections in the trench, connecting the tunnel sections to each other, backfilling soil over the tunnel elements to restore the former harbor bottom contour, and finishing the tunnel by building inside the tunnel the ceiling slabs and installing reflective tiles on the tunnel walls. Each steel tube tunnel element was 82 ft (25 m) wide and 42 ft (12.8 m) high. The alignment near Fort McHenry and below the shipping channel required both horizontal and vertical curves. The materials removed from the trench excavation were placed at the Canton–Seagirt dredge disposal site that was converted to the 136-acre Seagirt Marine Terminal. During placement, 76 cellular cofferdams contained the fill, each 62 ft (18.9 m) in diameter, stretching 5,600 linear ft (1,707 m) around the containment area. The tunnel and terminal facilities are owned by the Maryland Transportation Authority (*6*).

Cut-and-Cover Tunneling

Cut-and-cover tunnels are often built within an open cut excavation. If sufficient room exists and soil and groundwater conditions permit, the excavation can be laid back to stable slopes and the reinforced concrete tunnel structure built from the base of the excavation up and then backfilled. If ground conditions or available space do not permit a sloped excavation then some type of excavation support and bracing must be used. The excavation support may include soldier piles and lagging, steel sheet piles, shotcrete, or slurry walls. The bracing may include struts, soil nails, or tiebacks. Ground improvement methods such as deep soil mixing can be used to enhance the strength of the ground so it can help support itself. When this type of construction is used in congested urban environments, decking is usually installed that rests on pre-installed shoring so that traffic is not impeded during excavation underneath the decking. The Central Artery Project used many of these methods.

The downtown I-93 portion of the Central Artery Project included approximately 23,000 linear ft (7,000 m) of Soldier Pile Tremie Concrete (SPTC) slurry wall with steel wide-flange soldier piles used to reinforce the slurry walls spaced at 4 to 6 ft (1.2 to 1.8 m) on center. The 3- to 4-ft (0.9- to 1.2-m) thick slurry walls form the sidewalls of the highway tunnels, which were excavated beneath the six-lane I-93 viaduct prior to demolition. The viaduct was underpinned with grade and needle beams supported by the slurry walls. In most places, the concrete slurry walls act as lagging spanning between the structural piles. A concrete base slab is rigidly connected to the SPTC walls and the composite roof is pin connected to the walls to form the tunnel box section (*7*).

CONTRACTING METHODS

Most of the tunnels described above were done using the common design–bid–build contracting practices. However, some projects such as the Whittier Access (Anton Anderson Memorial) Tunnel used design–build as the contracting mechanism quite successfully. The Whittier Access Project–Tunnel Segment is the conversion of a 2.5-mi (4.0-km) single-track hard rock rail tunnel to a dual-use rail–highway tunnel, forming part of the highway connecting the port of Whittier to the Seward Highway and the commercial heart of Alaska. Although the tunnel is not along an Interstate highway, it is the longest road tunnel in North America. Its dual function is accomplished with a precast concrete roadway surface with integral rails (Figure 5) and a sophisticated traffic control system to both control the single-lane highway traffic flow direction and enable safe operation of train traffic. This landmark project boasts several other "firsts." It is the first combined-use rail-highway tunnel in North America, first tunnel to use a ventilation system that combines jet and portal fans in the United States, and first tunnel designed to operate in temperatures down to –40°F (–40°C), and winds up to 150 mph (242 kph), and portal buildings able to withstand avalanches. Challenges on the project included designing a surface that would accommodate both automobiles and trains, rock excavation—enlargement of the tunnel constructed in 1941–1942, design and construction of the portal structures—able to withstand avalanches and "A-frame" to split snow slides, a unique two-tiered drainage system, and systems to assure all vehicular traffic has cleared the tunnel prior to allowing opposite direction traffic and/or train traffic. This tunnel is owned by the Alaska DOT.

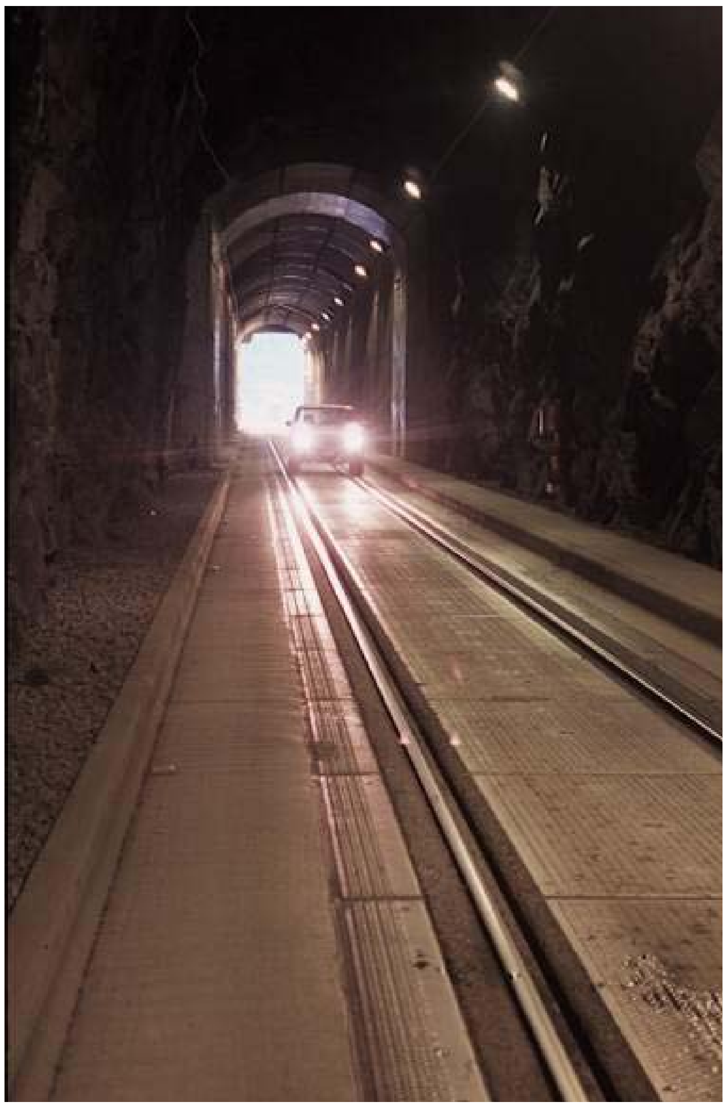


FIGURE 5 Composite roadway and rails in Whittier Tunnel.

OTHER DEVELOPMENTS

Over the past 50 years, there have been many other major developments related to highway tunnel ventilation systems, lighting, architectural finishes, communication devices, fire life safety, and security monitoring and management. Many of these systems require significant operational and maintenance attention that impose complex requirements during design and construction especially after the fiendish events of September 11, 2001, and ongoing terrorist threats and acts that are on the world's mind today. Recently on October 19, 2005, the Fort McHenry tunnel was closed for more than 2 h due to concern about an alleged terrorist threat to detonate explosives in the tunnel (*8*). As a result of the ever-changing technical and security environment, many of the tunnels discussed in this paper have undergone significant renovations and upgrades to bring them up to modern and safe standards.

FUTURE INTERSTATE HIGHWAY AND NON-INTERSTATE ROADWAY TUNNELS

Many significant Interstate highway and non-Interstate roadway tunnel projects have been completed in the past 50 years. Many more are being planned, designed, or constructed utilizing many developments that have occurred to date as well as lessons learned. Some of these projects include the Port of Miami Tunnel in Florida, Devil's Slide Tunnel in California, Gowanus Expressway (possibly a tunnel) in New York, 3rd Hampton Roads Crossing in Virginia, LBJ Freeway in Texas, Drumanard Tunnel in Kentucky, Fourth Caldecott Tunnel in California, the Alaska Way Tunnel in Washington, and possibly a tunnel along Route 9A in Lower Manhattan, New York.

CONCLUSIONS

There have been several major Interstate highway tunnel construction projects over the past 50 years in the United States. In addition to Interstate highway tunnel projects, several other tunnels have been built along non-Interstate roadways using similar construction methods and contracting practices. Many of the tunnels were required to carry traffic across waterways or through mountains. Others were constructed to provide grade separation with intersecting transportation routes, replacement of old and decrepit viaducts, or to reclaim surface space at-grade. Significant strides have been made to make this type of facility both economic and safe. It is expected that these trends will continue well into the future.

ACKNOWLEDGMENTS

Much of this paper is based on tunnel projects that the author has participated in while employed at the firms of Hatch Mott MacDonald and Parsons Brinckerhoff Quade & Douglas. FHWA, several state DOTs, construction contractors, and expert consultants were also instrumental in the success of these projects. The author gratefully acknowledges the advice and knowledge shared freely by his colleagues at the Transportation Research Board and all of these entities who are too many to name herein.

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Culverts
Hidden Structures, Critical Infrastructure Elements

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The U.S. transportation system is vast. Roadways and railroad beds cross small rivers, streams, and drainage channels millions of times, and each crossing requires a culvert to provide drainage without disrupting the flow of traffic. Numbering in the millions, culverts form a critical element of the national infrastructure.

Guidelines for design and installation of culverts have evolved from manufacturers' trade associations to the current detailed AASHTO specifications, which control routine drainage culverts as well as small bridges with spans of 50 to 80 ft (15 to 24 m). AASHTO has developed these specifications through extensive federal, state, and industry research.

Challenges exist in keeping the existing culvert population functioning and in improving the service life of new culverts. Key areas for improvement include improved asset management to monitor the culvert population, improved joint design and testing, and development of culvert inlet, outlet, and barrel rehabilitation materials and procedures. Vigilance must be maintained to see that good practice is used in construction of culvert installations. Through the use of improved materials, design practices, and construction procedures, more durable culvert installations can be achieved.

This paper explores the growth of culvert design, construction, and material practices; assesses the existing condition of the culvert population; and identifies critical issues for keeping that population and our highways, healthy for the coming years.

INTRODUCTION

There are numerous locations where small rivers, streams, and drainage channels must cross roadways and railroad beds. Every one of these locations requires a culvert to provide drainage without disrupting traffic. Culverts are buried, and like water pipelines, they are often forgotten until there is a failure and a road is closed, yet, numbering in the millions, culverts form a critical element of infrastructure.

When the U.S. Interstate system was first initiated 50 years ago, the design and installation of culverts was left largely to manufacturers' trade associations. However, as culvert sizes grew larger, new materials become available, and our understanding of the importance of proper design and construction increased, AASHTO developed detailed design, construction, and material specifications for metal, concrete, and thermoplastic culverts. These procedures continue to be used for routine drainage culverts as well as for large culverts with spans of 50 to 80 ft (15 to 24 m) (Figure 1). AASHTO's understanding of culvert-soil interaction has grown through federal, state, and industry research.

As the Interstate system reaches its 50th birthday, the culvert population is aging as well. Many culverts are nearing the end of their service life and need repair and rehabilitation. Many states are becoming aware that culverts require asset management programs just as have been implemented for bridges. However, the issues for culverts are largely different than for bridges in that individual culverts are less critical than individual bridges, but the numbers of culverts is huge relative to the number of bridges. For example Utah has 2,800 bridges, but more than 47,000 culverts. Some states do not know how many culverts are in their systems or where they are.

This paper explores the growth of culvert design, construction, and material practices; assesses the current condition of the culvert population; and identifies critical issues for keeping that population healthy.

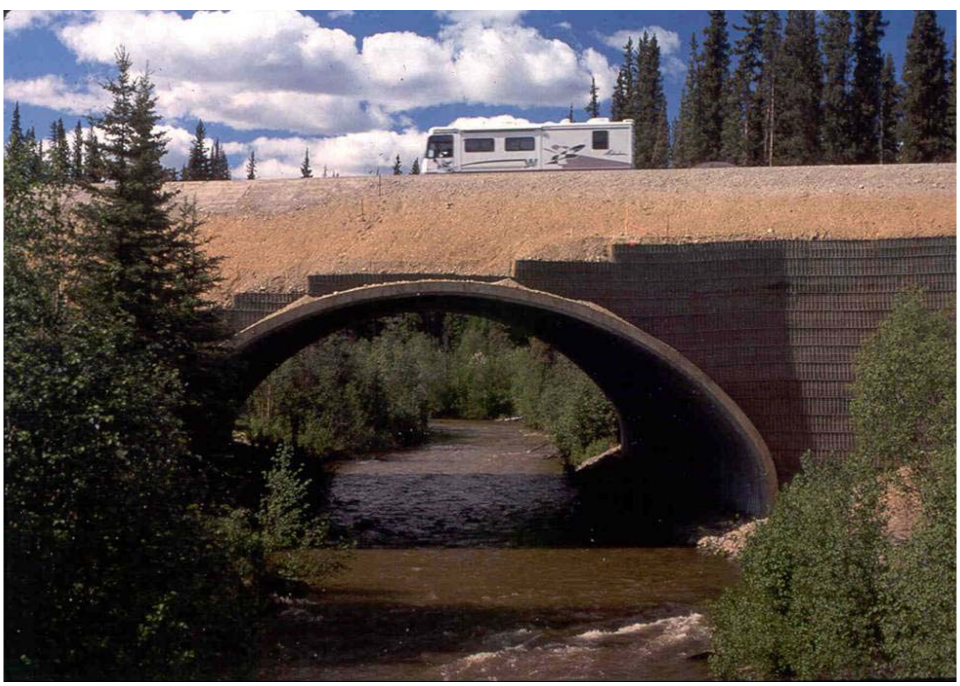

FIGURE 1 Large-span culvert used for stream crossing.

HISTORICAL DEVELOPMENTS

Culverts vary widely in size and shape. Culverts provide small-diameter underdrains to long-span stream crossings; have shapes including round, rectangular, elliptical, and pear; and may have open or closed inverts. Culvert materials include metal, concrete, and thermoplastics. All culverts are designed under the culvert sections of AASHTO specifications *(1, 2)*. Large-span culverts [spans greater than 20 ft (6m)] are treated as bridges for inspection and safety issues.

Metal culverts are classified as flexible culverts. They depend on proper installation to control deflections. Small metal culverts are usually quite stiff, while larger sizes can be extremely flexible. Long-span metal culverts require careful shape control during installation. Because of their ductility, steel and aluminum culverts are generally allowed to yield in bending under service earth loads.

Thermoplastic culverts are also flexible culverts. Thermoplastic culverts were first investigated by AASHTO in the 1970s *(3)* and have seen increasing use since that time. They are lightweight and corrosion resistant.

Concrete culverts are classified as rigid and have the least dependence on soil support to provide structural performance. In small diameters and under low fill heights, there are rarely structural issues with concrete pipes. In larger diameters and under higher fills, concrete pipes depend on soil support to provide good structural performance and installation procedures become much more critical. Concrete culverts are mostly precast, but some are cast-in-place.

Other types of culverts have been used in the Interstate system, such as brick and stone masonry. These culverts have provided good service, but there are no active AASHTO specifications for them.

The following provides a brief background on the developments of metal, thermoplastic, and concrete culverts that are in current AASHTO specifications.

Metal Culverts

Corrugated metal pipe (CMP) has been manufactured and used as culverts and other drainage structures for approximately 110 years. Early in the 20th century, the base steel for CMP was improved and coated with zinc to improve durability. By 1912, the first AASHTO specification for CMP was introduced. Asphalt coatings were introduced for added durability around 1920.

The industry realized the need for larger structures and developed structural plate that could be shipped unassembled and field erected. Starting in the early 1930s, structural plate evolved from round bolted structures to many varying configurations used as stream crossings and grade separations. During the last 50 years, field experience and improved design procedures have allowed structural plate pipe to be furnished in much greater opening sizes and spans. Today, structural plate culverts can be furnished with spans up to 80 ft (24 m), under high fills, and under live loads substantially exceeding normal highway loading. Corrugated metal box culverts (Figure 2) were developed with essentially rectangular openings to be used as low-wide culverts and storm sewers under minimum cover installations.

Extended service life requirements necessitated the corrugated pipe industry to find improved coatings for steel pipe. An aluminized coating placed over steel was commercially introduced in the late 1970s and has a predicted service life up to 75 years and more. In the early 1970s, a polymeric film placed over galvanized steel was introduced. This film has proven to protect corrugated steel pipe in a wide range of environments with a projected service life of 100 years.

FIGURE 2 Metal box culvert with metal end sections.

The corrugated pipe industry introduced its first hydraulically improved pipe around 1980. This pipe had the standard corrugated exterior with a smooth metal liner integrally attached and is identified as double-wall corrugated pipe. The liner provides hydraulic equivalency to other smooth interior drainage pipe. By the late 1980s, “spiral rib” pipe was developed with external rectangular box ribs and a smooth interior for improved hydraulics. Both of these pipes can be furnished in all available coatings required for the project service life requirements.

CMP can be furnished in long joint lengths that facilitate faster installation. With relatively light weights, moderate-sized equipment can be used for installation. Additionally, positive connections can be furnished to improve structure stability and integrity on steep slope applications. On many bridge replacements or large culvert projects, road shut down time is critical. This is especially true for emergency routes, school bus routes, or high traffic areas. Many times, structural plate can be field assembled near the site prior to opening a road. Once traffic is stopped, structure removal can proceed immediately. Upon removal, bedding is placed and the new plate structure can be installed and backfilled in a relatively short time. Road shutdown time is minimal.

Today, many drainage structures are nearing the end of their expected service life. Culvert relining is an option that many transportation departments favor to avoid the traffic interruption and high installation costs associated with replacement options. CMP is being used in many of these applications. The pipe can be sized to individual project diameter requirements and designed to carry high fills. Minimal hydraulic losses are attained through using spiral rib or double wall pipe.

Thermoplastic Culverts

Thermoplastic pipes were initially used over 60 years ago in Europe and North America for water and sewer applications. Polyethylene (PE) and polyvinyl chloride (PVC) pipes have been used for drainage applications in the United States for over 35 years and for highway pavement underdrain applications for more than 30 years. Since their inception, both PVC and corrugated PE pipes have proven to be a structurally viable, non-corrodible, chemically resistant, erosion-resistant pipe for use in the full range of road culvert applications. As a result, shipments of PE and PVC pipe have increased steadily for the past several decades and the demand for them is continuing to increase.

Derived from the agricultural industry, the first use of corrugated PE pipe (Figure 3) for highway applications was on Interstate 80 (I-80) in Iowa in 1972. Following this application, the Iowa Department of Transportation (DOT) added the product to their specifications and an AASHTO standard (M 252) was developed shortly thereafter. As technology and demand for the product continued to increase, larger sizes were developed, with 30-in. (760-mm) and 36-in. (900-mm) diameters entering the market in the mid-1980s, along with a new AASHTO Standard Specification for large-diameter corrugated PE pipe (AASHTO M 294). Dual-wall pipe, corrugated exterior with a smooth interior, was also first manufactured in the 1980s, offering better hydraulic capabilities than the single-wall products. The 1990s saw the addition of diameters up to 60 in. (1,500 mm).

FIGURE 3 Corrugated PE installation.

The first corrugated PE DOT crossdrain was installed by Ohio DOT in 1981. The pipe material was chosen due to its resistance to chemical attack at an acidic site. Other applications demonstrating the capabilities of the product include the Pennsylvania Deep Burial Study installed in 1987, where nearly 600 ft (183 m) of 24-in. (600-mm) diameter corrugated PE pipe was installed at depths up to 104 ft (32 m) underneath I-279 north of Pittsburgh. This research project, a cooperative effort between the Pennsylvania DOT and University of Massachusetts, demonstrated PE pipe's ability to be installed at great depths with proper installation.

PVC is a dominant pipe material for both sanitary sewer and water distribution pipes in the United States. PVC pipes have been used for sewer and drainage applications since the 1930s in Europe and the 1950s in North America. The first AASHTO standard for PVC pipe was M 278. Larger sizes and the introduction of structured-wall or profile-wall technologies resulted in the development of AASHTO Standard M 304, which includes diameters up to 48 in. (1,200 mm).

Corrugated PE pipes and PVC pipes used in culvert applications today are manufactured with an integral bell-and-spigot joining system that, when used with an elastomeric gasket, can provide a watertight joint that prevents soil and water infiltration and exfiltration. PE and PVC pipes are relatively lightweight and easy to handle. They are generally manufactured in lengths of 10 to 20 ft (3 to 6 m).

PE and PVC pipes have also become a good option for the relining and rehabilitation of culverts in need of repair. High resistance to corrosion and abrasion wear (erosion) enables these products to greatly extend the lives of aging culverts. The service lives of PVC and PE culverts have been projected beyond 100 years.

The development of a new design methodology for PE and PVC culverts is nearing completion. While the current AASHTO design methodology is adequate, this new approach is expected to enhance the proper selection and installation of PE and PVC pipes.

Concrete Culverts

Precast concrete pipe has been used under U.S. roadways for more than a century and in the American sanitary sewer and drain tile industries since the mid-1800s. The last 50 years have been a time of improved consistency and performance of the product, as well as improved understanding of the pipe–soil structure.

Research and testing at Iowa State University from 1913 to 1946 developed concrete pipe design procedures that were used until the 1970s. Beginning in the early 1970s, further research using improved analytical methods and full-scale tests was commenced to improve understanding of pipe–soil interaction and the impact of modern construction techniques. The current AASHTO design and construction procedures, called "Standard Installations," provide new direct design procedures for concrete pipe as well as improvements to the traditional indirect design method. Jointing for concrete pipe evolved from the basic mortar joint to flexible joints using rubber gaskets.

In 1970, a computerized program incorporating structural design concepts for precast concrete box culverts was developed. In 1974, the first ASTM standard for precast reinforced concrete box culverts was published. This was followed in 1976 by another ASTM standard specifically addressing precast box culverts carrying highway loads under shallow fills. Similar to the concrete pipe standards, these standards were later adopted by AASHTO. The ability to use precast box culverts as both a conduit and a bridge-type structure reduces construction time, in turn leading to increased use by DOTs (Figure 4).

Driven by the economy of precast concrete culverts, three-sided culverts were developed to provide longer spans. The open-bottom design of these culverts also addressed increased environmental demands to provide natural stream beds in culverts. These structures are manufactured both with flat tops, which provide a good roadway surface, and with arch tops, which are more efficient structural shapes. The three-sided industry expanded rapidly as DOTs realized the ease, speed, and economy of using modular set-in-place construction to update and repair the aging bridge infrastructure. Now, the industry has expanded beyond the typical small short-span bridges, using multiple cells or prestressed slabs to increase the effective span and precast pedestal walls to increase the rise.

While a better understanding of the design of precast concrete culverts was occurring, improvements to the materials and manufacturing of the products were also being made. Computer automation and testing has found its way into the production line of precast products. The benefits are increased output, greater manufacturing consistency, and reduced manpower. In more modern plants, a handful of employees can produce a precast product that was once labor intensive.

With advancements now being made, the next revolution in concrete pipe will be the use of ultra-high-performance materials to improve constructability, durability, and mechanical properties. Concrete pipe manufacturers will offer flexible or rigid concrete pipe depending on the soil conditions.

FIGURE 4 Concrete pipe and box sections.

Concretes using silica and chemically activated fly ash exhibit rapid strength development, high acid resistance, and high ultimate strength. Macro-defect-free cement, another option for the future, is a fine-grained mix produced under pressure to develop flexible strengths up to 60 times greater than normal cast concrete. Reactive powder concrete (RPC) can achieve compressive strengths up to 100,000 psi (700 MPa). In addition, the improved strength of RPC provides extremely high shear capacity, permitting thinner sections and a wider variety of shapes. Versions of RPC have already been tested in certain precast products.

Precast structures that can reduce the amount of construction time required for the overall completion of the project are a major benefit by reducing both the project cost and social cost of unhappy motorists. Precast structures that can serve as the bridge structure as well as the culvert bring an additional benefit.

CURRENT CULVERT RESEARCH

In spite of all the advancements in culvert technology, there are still needs to be addressed to improve culvert manufacturing, design, and performance. Current activities include the following:

- Improved understanding of live-load effects on culverts. While culverts have been successfully designed for live loads for many years, there remains uncertainty about the how live loads attenuate through fills and interact with culverts. The AASHTO Standard and Load and Resistance Factor Design (LRFD) Specifications for Bridge*s* have incorporated very different approaches to this matter. The NCHRP has instituted Project 15-29: Design Specifications for Live Load Distribution to Buried Structures to investigate this behavior.
- Finite element analysis of buried culverts has provided great insights into pipe–soil interaction and has proved to be invaluable in both research and design. FHWA developed the finite element pipe–soil interaction program CANDE, but this program has not been updated since 1989 and is not run in a graphical interface. NCHRP has instituted Project 15-28: Modernize and Upgrade CANDE for Analysis and LRFD Design of Buried Culvert, to provide a modern user interface, to design in accordance with current LRFD specifications , and to include additional enhancements to simplify culvert design.
- One of the pressures felt by state DOTs, as with virtually all public-sector organizations building new facilities, is that less time and budget are available for inspection during construction. To address this, AASHTO substantially upgraded its requirements for post-construction inspection of culverts in 2005. For the first time, AASHTO Construction Specifications now demand thorough inspections of all types of culverts before they are accepted for use. These inspections include barrels, inlets, outlets, joints, and, for flexible culverts, deflections. Inspections must be conducted at least 30 days after installation to allow initial settlement to occur.

FUTURE CHALLENGES

Maintaining the existing population of culverts and improving the expected service life of new culverts, all while controlling costs, is a great challenge. Huge numbers of culverts with an expected service life of 50 years were installed during the construction of the Interstate system; thus, as the Interstate system nears its 50th anniversary, those culverts are nearing the end of their design service life. Areas of concern are discussed here.

Remaining Service Life of Existing Culverts

Many culverts are performing well and can be expected to provide adequate service well beyond their original expected design life. However, increasing numbers of culverts will need rehabilitation or replacement in the near future (*3*,*4*). Engineers need either analytical tools to evaluate remaining service life or asset management programs to monitor changes in culvert conditions to allow advance planning for culvert replacement.

Many equations have been produced to predict the service life of metal culverts based on corrosion rates under various environmental conditions. The variety of predictions suggests that this is at best an imperfect science. Also, site conditions vary significantly over short distances, so obtaining quality input for the prediction equations for each culvert can be difficult. Predicting service life in many states is complicated by the fact that good records do not exist to establish current service times; thus, knowing total service life does not necessarily establish remaining service life. Relying on advance predictions of service life of culverts is difficult at best and would likely underestimate the life of culverts. A better solution to estimating service life is to establish asset management programs that monitor culvert conditions over time. Such programs could establish deterioration rates that are specific for each culvert. Even with occasional inspections, an asset management system will collect enough data over time to predict the end of a culvert service life well in advance, allowing adequate planning time for replacement or rehabilitation under normal contracting procedures.

A culvert management program brings with it inspection costs that could be substantial. With tens of thousands of culverts in every state, it would take several full-time staff to conduct the inspections and to maintain the database. McGrath and Beaver (*5*) have suggested a solution to this. They proposed that only critical culverts be inspected routinely. They classify critical culverts as those with large spans [culverts with spans of 20 ft (6 m) or more are already inspected routinely under the National Bridge Inventory Standards], at locations under critical roadways, or, for some other reason, that have a significant consequence of failure. Other culverts should be inspected and rated whenever road sections are repaved or reconstructed. Since most culverts would be classified as non-critical, the cost of inspections is spread out and incorporated into the cost of roadway maintenance.

Improved Characterization of Site Conditions

Improved service life is always a concern with culverts. All culvert industries are continually striving to develop improved material properties to extend service life. Through improved raw materials and coatings the service life of culverts has been extended significantly; however, serviceability problems still occur. These can be addressed in part by improved materials, and as noted above, all industries are working on this constantly. However, increased emphasis by DOTs and their consultants on evaluating site environmental conditions prior to construction would also improve culvert material selection and corrosion protection issues. Experience indicates that generalized maps of soil conditions are not sufficiently detailed to make corrosion predictions for specific sites. Design should include site-specific evaluations of in situ conditions for this purpose. Proper understanding of site conditions provides the information necessary to select the most durable and economical material for a project. Significantly improved service lives can be realized with more effort devoted to this task.

Improved Joint Design

Historically, many culverts joints have been allowed to pass water, provided they did not allow the passage of soil. This has been called a silt tight joint. However, quantifying joint tightness as soil tight or water tight is problematic. Specifications vary for different products, and it is difficult to compare expected joint performance for different products during design. Culvert or roadway failures do occur as a result of soil passing through joints, causing loss of support. Alternatively, water passing out of culvert joints can lead to a loss of support due to piping. These failures may be the result of design issues, product design or quality, or installation. An important goal is to standardize joint design to provide reliable and predictable performance. Particular goals should be standardized joint specifications for water tight–silt tight for all types of products and standardized joint qualification tests. These two important items will allow designers to make better decisions on joint selection. With the availability of improved gasket systems, engineers should revisit if it is still necessary to allow soil tight joints. This requires a cost–benefit analysis.

Improved Approach to Design and Construction

One of the challenges AASHTO currently faces is providing rational, yet consistent, design procedures for each type of culvert and then insisting on construction procedures and inspection that ensure that the specified design is achieved. This latter issue is critical. AASHTO is already addressing this through the aforementioned post-construction inspection requirements, but more can also be done during construction.

Since most culvert design procedures were developed by manufacturers and trade associations, they address specific behaviors that are pertinent to those products. The result is that, while suitable for design of a specific product, DOTs find it difficult to make rational comparisons of different products. While there are many performance limits where this approach is needed to address varying pipe behaviors and material properties, there are certain aspects of culvert design that are applicable to all culverts. Soil models is one of these, and in 2000, AASHTO instituted design procedures for thermoplastic culverts that are based on the same soil model as the AASHTO concrete culvert design procedures. This appears as a small enhancement, but is actually a significant step forward as it ensures a similar view of soil properties for both products, which has not always been the case in the past. AASHTO needs to identify additional design enhancements in the future to further align design methods. Greater similarity could be achieved between metal and thermoplastic culverts.

Construction of culvert systems must remain a significant focus of attention. Culverts are pipe-soil systems, needing both culvert strength and soil support to provide good performance; that is, the pipe and soil together constitute the structure. Good backfill material and placement specifications must be in place and enforced to expect good culvert performance.

Culvert Rehabilitation

As the culvert population ages, more and more emphasis is placed on developing ways to rehabilitate them in place, rather than excavation and complete replacement. Rehabilitations include inlets and outlets as well as culvert barrels. Inlet and outlet structures are often exposed to harsher conditions and deteriorate at a faster rate than culvert barrels. Culvert barrels are ideally suited for relining as they are generally readily accessible at each end. Insertion and cured-in-place liners have been used in the past. In some instances, invert replacements alone should be economical. Specifications and details, especially at inlets and outlets, need to be developed.

Durability

The need for durable culverts becomes clearer as our infrastructure ages. Subsurface structures such as culverts should be even more durable than the road surface above. The disruption to traffic when a road has to be ripped up to replace a culvert is significant, and unlike a pavement, the users do not see the improvement. In addition, problems in culverts that are rarely seen, and perhaps rarely inspected, can result in unexpected hazards. Culvert durability results from proper material selection based on site conditions, proper installation in quality backfill materials, and proper maintenance.

SUMMARY

During the 50 years of the Interstate system, the culverts have matured right along with bridges. New materials now stand alongside traditional materials as tools in the engineer's arsenal to provide more durable culverts with longer service lives. Coatings and other corrosion protection schemes have been developed to extend the life of base materials. Current culvert research activities were discussed. Challenges exist in keeping the existing culvert population functioning and in improving the service life of new culverts. Among the key areas for improvement include

- Improved asset management to monitor culvert population,
- Improved joint design and testing, and
- Development of culvert inlet, outlet, and barrel rehabilitation materials and procedures.

In addition to these items, we must maintain our vigilance in executing proper construction in the field and working for more durable culvert installations.

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Fiber-Reinforced Polymers for Transportation and Civil Engineering Infrastructure
Reality and Vision

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This paper reviews the reality and vision of the use of fiber-reinforced polymer (FRP) composite materials in civil engineering and transportation infrastructure applications. Due to their superior characteristics these materials have become a commonly accepted construction material for both new construction and for rehabilitation and strengthening of existing bridges and structures. This paper presents a number of example field projects which demonstrate the level of acceptance of FRP materials in infrastructure applications. While the use of FRP composites has become common practice in civil engineering, their use to date has largely been an imitation of conventional construction materials. Only recently have technologies been developed which effectively utilize the beneficial characteristics of these materials. A future vision for the use of FRP materials in civil engineering infrastructure applications is underway at many university research institutions and in private industry. This paper reviews a number of the current concepts and innovative technologies that are currently being developed with the vision of optimizing the uses of FRP materials. This paper demonstrates that FRP materials have a demonstrated history of superior past performance and a bright future vision in civil engineering applications.

INTRODUCTION

In recent years, FRP composite materials have emerged as an acceptable construction material with application for solutions to several problems associated with transportation and civil engineering infrastructures. The proven record of the superior characteristics of FRP in terms of strength, durability, resistance to electrochemical corrosion, and versatility of fabrication have made these materials attractive to civil engineers. However, their use for the last decade was mainly an imitation of the use of other conventional construction materials rather than fully utilizing their potential and unique characteristics. This paper reviews the current level of the use of these materials for civil engineering infrastructure and transportation as well as the vision for the use of these new innovative construction materials to their full potential.

In the United States the actual development and research activities into the use of these materials started in the 1980s through the initiatives and vision of the National Science Foundation (NSF) and FHWA, who supported research at different universities and research institutions. Due to the excellent performance of these materials in laboratory trials, the use of composites progressed into various field applications, and their use has become commonplace for new construction, as well as in repair and rehabilitation of structures and bridges.

The development of FRP composites has opened up a number of opportunities in civil engineering. However, due to their relatively high cost, the use of composites is only economical in certain applications. For very long bridge structures, the high strength to weight ratio of composite materials is particularly advantageous (Meier 1986). Meier's vision is to construct an all-composite bridge across the Strait of Gibraltar at its narrowest point. Meier (1986) outlined the feasibility of using various types of cables to support long span bridges. He estimated the limiting span for classical suspension type bridges using steel cables, defined as the span at which the structure could barely support its own weight, to be 4,490 m. Using carbon FRP (CFRP) materials, the limiting span was estimated to be 14,580 m. However, to achieve such long spans significant advances must be made to utilize these materials to their full potential.

This paper describes the reality and the vision of composites in infrastructure. The first part of this paper discusses the current common practice in the use of FRP as a construction material for civil engineering and transportation applications. A number of example field projects are presented which demonstrate their advantage in solving several infrastructure problems. These practices have become standard operating procedures for the use of composites worldwide. The innovations discussed in the first section of this paper are currently being used as common practice for new construction of highway bridges and structures and for repair of existing infrastructure such as the extensive U.S. Interstate highway system. FRP materials represent an effective means to retrofit the existing structures on the 50-year old Interstate highway system thereby further extending their serviceable lives. The use of FRP materials for new construction can also help to ensure rapid construction and long-term durability of new structures throughout the highway system.

The second part of this paper briefly reviews the future vision of using FRP to their full potential based on selected current research work. A number of technologies are presented which are still under development for future industrial implementation. These innovative technologies represent an optimization of the use of composite materials which is necessary to achieve wide spread embracing and utilization by practicing engineers.

REALITY

Due to their versatility, FRP materials have gained widespread use in a number of civil engineering and transportation applications. As outlined in this section, composites are used both for new construction and in strengthening and rehabilitation of existing bridges and structures.

New Construction

Throughout the 1980s and to the present the use of composite reinforcements in new construction proliferated throughout Europe, Asia, and North America. As engineers have gained confidence in the use of composite materials, their use as a replacement for steel reinforcements for reinforced and prestressed concrete structures and bridges has become common practice. The following highlights selected applications of the use of FRP for new construction.

Reinforcement for Concrete Structures

One-dimensional CFRP and glass FRP (GFRP) bars are commercially fabricated and used as reinforcements for concrete structures. Due to their excellent corrosion resistance, FRP reinforcing bars have been extensively used as reinforcement for a number of concrete structures including bridges, underground precast chambers, highway pavements, maglev rails and structures housing magnetic resonance imaging (MRI) equipments.

FRP reinforcing bars were used exclusively in the construction of a portion of the concrete deck and barrier wall of a highway bridge (Rizkalla et al., 1998). The five-span bridge, which was constructed in 1997, consisted of eight 33-m-long, simply supported AASHTO-type prestressed concrete girders. A 16 m x 8 m portion of the deck was exclusively reinforced with 10 mm diameter CFRP bars. The CFRP bars were placed in two layers with bars oriented in both the longitudinal and transverse directions to create a two-dimensional reinforcement grid. Casting of the deck is shown in Figure 1*a*. A 14.2-m-long portion of the concrete barrier wall was also reinforced using two layers of 15-mm GFRP rods since these type of elements are subjected to severe environmental conditions including wet and dry cycling as well as splash of salt from the roadway.

To facilitate rapid construction, FRP materials are also fabricated as two-dimensional grids, in a similar fashion of conventional steel welded wire fabric, which were used as reinforcement for a concrete bridge deck as shown in Figure 1*b* (Benmokrane et al., 2000). Portions of the concrete barrier and sidewalk were also reinforced using different CFRP and GFRP reinforcements. Various types of gauges were used to monitor the behavior of the bridge including structurally integrated optical sensors to monitor the behavior under field conditions.

Shear stirrups represent another application of FRP in bridges since these particular reinforcements are the most susceptible to deterioration due to corrosion. Typically a minimum concrete cover is provided for shear reinforcements. Moisture ingress into cracked cover concrete can cause corrosion of the stirrups and spalling of the cover concrete. Two of the precast AASHTO girders used for construction of the Taylor Bridge were also reinforced using two different types of CFRP stirrups. The stirrups were extended out of the top surface of the

(a) **(b)**

FIGURE 1 FRP-reinforced concrete bridge decks: (*a*) Taylor Bridge and (*b*) Joffre Bridge.

girders to act as shear connectors and provide composite action between the prestressed girders and the concrete deck slab. The fully assembled reinforcing cage for one of the girders is shown in Figure 2.

Buried structures are continuously exposed to moist soils and aggressive corrosion-causing minerals which accelerate the deterioration of the conventional steel reinforcing bars and can lead to cracking of the concrete structure. Thousands of steel-reinforced chambers require replacement on an annual basis due to corrosion of the conventional reinforcing steel. Due to the limited access to these structures, replacement can be costly and time consuming. Typical chambers constructed using GFRP reinforcements and installed underground are shown in Figure 3 (Benmokrane, 1999). The chambers were constructed to demonstrate the feasibility of using composite materials to reduce the need for replacement of underground structures.

FIGURE 2 CFRP stirrups for prestressed girders.

FIGURE 3 GFRP reinforced underground precast concrete chambers.

Smooth GFRP dowels are commonly used as a replacement for epoxy-coated dowels to provide shear transfer across transverse joints in concrete highway pavements (Eddie et al., 2001). The non-corrosive GFRP dowels can reduce the need for future maintenance due to deterioration of the concrete near the joint caused by expansion of corroded steel dowels. A field application of 38-mm diameter GFRP dowels installed in concrete highway joints is shown in Figure 4. The performance of the highway joints that use the 38-mm GFRP dowels is comparable to the performance of joints that use 32-mm epoxy coated steel dowels. In a similar project the Ohio DOT installed GFRP dowels and conventional epoxy coated steel dowels along portions of I-77 and SR 7 in 1983 and 1985, respectively (Busel, 1999). After 13 years in service the steel dowels showed signs of deterioration due to corrosion while the GFRP dowels did not exhibit any decrease in load-transfer efficiency.

Prestressing of Concrete Structures

The high-strength characteristic of one-dimensional FRP products has been effectively used for prestressing concrete bridge girders. Several CFRP and aramid FRP (AFRP) rods and tendons that are produced worldwide are used in prestressing applications. The German bridge Ulenbergstrasse in Düsseldorf, which was constructed in 1986, was the first vehicular bridge in the world to use composite prestressing tendons (Mufti et al., 1991). In 1993 the Yamanaka Bashi Bridge was the first newly constructed bridge in Japan to be entirely prestressed by AFRP cables (Tezuka, 1994). Since then, numerous other prestressed bridges have been constructed using FRP tendons throughout Europe, Japan, and North America.

The first bridge in North America which used bulb-tee bridge girders prestressed with CFRP strands and rods was opened to traffic in 1993 (Rizkalla and Tadros, 1994). Six of the total 26 girders were prestressed by two different types of CFRP tendons as shown in Figure 5. Posttensioned steel cables extending along the entire length of the bridge provided continuity of the two spans. The CFRP prestressing was designed to ensure identical behavior under service loading conditions to the remaining girders, which were prestressed with steel tendons.

FIGURE 4 Installation of GFRP highway dowels (Gentile et al., 2001).

FIGURE 5 Centre Street–Beddington Trail Bridge bulb-tee girders.

FIGURE 6 The award-winning Taylor Bridge.

In 1997, two different types of draped CFRP tendons were used in a field application (Rizkalla et al., 1998) to prestress four of the forty AASHTO type girders, each with a span of 33 m. Two girders were prestressed by a total of 38 straight and 18 draped leadline CFRP tendons while another two girders were prestressed by a total of 32 straight and 14 draped CFCC tendons. The completed bridge, shown in Figure 6, was awarded the PCI Harry H. Edwards Industry Advancement Award in 1998.

Precast FRP Piles

Concrete-filled FRP tubes (CFFT) provide an effective alternative to conventional prestressed concrete piles. The composite piles consist of a filament wound GFRP outer shell which acts as a stay-in-place formwork and non-corrosive reinforcement for a concrete core. The FRP shell also protects the concrete core from exposure to harsh environmental conditions and provides confinement for the concrete thereby increasing the strength and ductility of the pile. CFFT piles were selected to support an entire bent of a highway bridge in 2000 (Fam et al., 2003b). The bent, shown in Figure 7*a*, consists of a reinforced concrete cap beam supported by 10 battered circular composite piles which were installed using typical conventional pile driving techniques.

There have been at least eight reported marine applications throughout the United States for which CFFT piles were utilized (Fam et al., 2003a). In 1998, CFFT piles were used to construct a naval station which houses electronic equipment that is sensitive to the presence of metallic structures. CFFT piles were installed as structural piles and as fender piles and dauphins (pile groups) which are used to protect other structural components against impact damage as shown in Figure 7*b*. The piles had sufficient strength to survive a recent hurricane. In other applications CFFT piles were selected over conventional piles due to their environmental durability, their resistance to marine borers and also due to their lack of toxic treatment chemicals.

FRP Bridge Decks

A number of commercially available pultruded FRP bridge deck systems have been used as a replacement for conventional highway bridge decks. As of 2003, there were at least 123 reported bridges constructed using FRP bridge decks worldwide, 90 of which were open to vehicular traffic (MDA, 2003). Most commercially available decks consist of assemblies of adhesively bonded pultruded shapes. Continuous pultruded shapes are fabricated using well-established processes and assembled into modular panels as shown schematically in Figure 8. The panels are transported to the site and installed onto the existing superstructure. Typically construction of the deck is completed by installation of a wearing surface to protect the deck from mechanical damage. In 1997 the Laurel Lick Bridge was one of the first highway bridges in the United States to be constructed with a pultruded FRP deck system (Lopez-Anido, 1997). The deck was part of an all-composite bridge structure that also included wide-flange pultruded stringer beams and piles.

(a) **(b)**

FIGURE 7 Applications of CFFT piles (*a*) completed Virginia DOT Route 40 bridge pier (Fam et al., 2003b) and (*b*) fender piles and dauphins at the Ingleside Naval Station (Fam et al., 2003a).

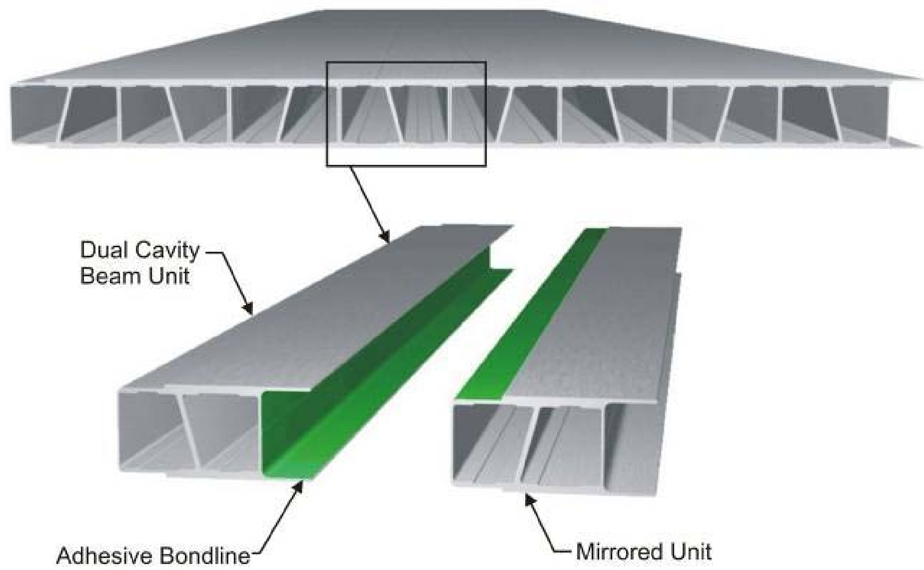


FIGURE 8 Fabrication of typical pultruded FRP bridge decks (Martin Marietta Composites, 2005).

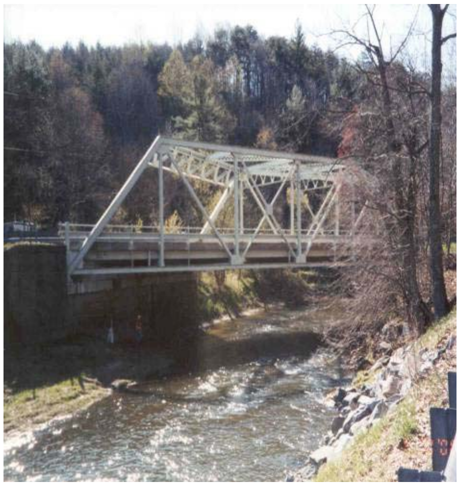
Another project which highlights the advantages of using modular FRP bridge decks is the replacement of the bridge on Maryland Route 24 over Deer Creek, shown in Figure 9*a* (FHWA, 2004). The use of the lightweight panels facilitated rapid installation of the panels using a forklift. Due to the limited access to the site because of the presence of overhead through-truss members, as shown in Figure 9*b*, crane access was not feasible. The use of the modular FRP bridge deck panels allowed the forklift to immediately drive over the erected portion of the bridge deck to install subsequent panels as shown in Figure 9*c*. The use of the lightweight panels also reduced the self-weight of the structure therefore increasing the allowable live load level for the load-restricted historic bridge.

Pedestrian Bridges

FRP materials are frequently used to construct all-composite pedestrian bridges. The lightweight FRP components facilitate rapid, environmentally friendly assembly and installation in remote locations such as those shown in Figure 10. The FRP components are transported by road, using specially designed trolley systems or even by helicopter resulting in a minimum disturbance to the natural environment.

In 1992 the longest FRP pedestrian bridge was constructed in Scotland (Busel, 1995). The 113-m-long cable-stayed bridge, shown in Figure 11, consists of a GFRP deck and two 17.5-m tall GFRP A-frame towers. The deck is supported from the towers by AFRP cables. The structure was constructed in less than 8 weeks by an average sized team of six university students without using any crane facilities.

While the various components of composite building systems are typically attached to one another by adhesive bonding, some systems make use of FRP mechanical connectors to simulate bolted construction of steel structures. The Japanese Public Works Research Institute constructed a 20-m-long three-span continuous composite bridge in 1996 (Sasaki, 1997). The



(a)

(b)

(c)

FIGURE 9 (*a*) Maryland Route 24 bridge over Deer Creek; (*b*) overhead through-truss structure; and (*c*) installation of the FRP bridge deck panels using a forklift.

(a)

(b)

FIGURE 10 Remote installation of FRP pedestrian bridges.

FIGURE 11 Aberfeldy Footbridge across the Tay River in Perthshire, Scotland.

joints are fastened using FRP bolts and nuts. The connections were designed to allow disassembly of the bridge for transportation and reconstruction at another location. The weight of each of the individual bridge members does not exceed 150 kg.

Special Structures

The rooftops of the Aerial Train Station in Stone Mountain, Georgia, shown in Figure 12, were fabricated from a number of commercially available FRP components (Clark, 1996). The roof's 64 rafters and 17 columns consist of pultruded FRP structural shapes and the roof structure is covered by GFRP building panels. The components were connected using FRP composite studs and nuts. GFRP components were selected for the roof structure to avoid electromagnetic interference with radio transmissions while maintaining the aesthetics of the park.

Strengthening and Repair

While composites are commonly used for new construction another practical, effective and promising use of FRP materials is for the strengthening and rehabilitation of concrete bridges and structures. For flexural strengthening of concrete members, FRP can be externally bonded using wet lay-up of sheets or strips as well as and near surface mounted applications using strips and bars. Externally bonded composites can also be used for shear strengthening of girders and retrofit of columns. A number of special structural applications have also been reported which involved the use of composites for repair. Most recently, externally bonded FRP has been used also for repair of metallic structures including cast iron and aluminum.

FIGURE 12 Aerial Train Station, FRP roof structure in Stone Mountain, Georgia.

Flexural Strengthening of Bridges

Externally bonded FRP sheets and pre-cured laminates are commonly used for flexural strengthening of concrete structures. FRP sheets are typically used for structures with severe surface irregularities or geometric discontinuities since the dry fibers can be easily shaped to match the variation of the surface profile (Shahawy, 1995a; Shahawy, 1995b). The fibers are impregnated in-situ, as shown in Figure 13*a* using a thermoset epoxy resin which acts as both the matrix for the FRP material and as the adhesive that bonds the FRP to the existing substrate. In applications requiring higher levels of strengthening, precured FRP laminates are attached to the surface using a thermoset adhesive as shown in Figure 13*b*. Premature debonding can be prevented by ensuring proper preparation of the concrete surface and by installing transverse U-shaped wraps to help anchor the longitudinal fibers as shown in Figure 13*a*.

Another method for strengthening structures is the near-surface mounting (NSM) techniques. This procedure involves embedding composite bars or strips in narrow adhesive-filled groves which are cut into the bottom of the existing structure as shown in Figure 14. NSM repair methods may be appropriate in applications when environmental exposure of externally bonded composites may adversely affect the bond between the repair and the substrate concrete. NSM bars are also commonly used to repair timber bridge girders. In 2000, 70 timber stringer beams of a bridge were strengthened using NSM GFRP bars as shown in Figure 14*b* (Gentile et al., 2002).

Shear Strengthening of Bridges

Externally bonded FRP composites are also used for shear strengthening of concrete structures. Typically fiber sheets are used to conform to the geometry of the surface of the beam. Externally bonded CFRP sheets were used for shear strengthening of two prestressed concrete bridges in 1999 (Hutchinson et al., 2003). Two girders of a bridge were strengthened using carbon fiber

(a) (b)

FIGURE 13 Externally bonded FRP: (*a*) sheets and (*b*) strips for flexural strengthening of prestressed concrete girders

(a) (b)

FIGURE 14 NSM strengthening of (*a*) prestressed concrete beams and (*b*) timber beams.

sheets. The sheets were attached to the girders with the fibers oriented in a vertical direction as shown in Figure 15*a*. A second layer of sheets was attached with the fibers oriented horizontally to anchor the vertical fibers. The shear capacity of the girders was increased by between 20% and 25%. A similar strengthening project was undertaken at the John Hart Bridge in Prince George, British Columbia, where 42 I-shaped girders were strengthened using CFRP sheets that were placed with a diagonal orientation, as shown in Figure 15*b*. The diagonal orientation was selected to optimize the use of the fibers by placing the fibers parallel to the direction of principal tension. The shear capacity of the girders was increased by between 15% and 20%.

(a)

(b)

FIGURE 15 Shear strengthening of (*a*) the Maryland Bridge and (*b*) the John Hart Bridge using externally bonded CFRP sheets.

Rehabilitation of Columns

Wrapping concrete elements with FRP sheets is currently a common technique used for the rehabilitation of concrete columns. The wraps provide confinement for the concrete thereby increasing the strength and ductility of the column. This concept is also commonly used to enhance the seismic resistance of bridge piers and columns of buildings throughout California and Japan. The presence of the externally bonded FRP also helps to prevent ingress of moisture and improve the durability of the column. Nine circular columns of a bridge were repaired using CFRP and GFRP wraps in 1996 (Neale and Labossièrre, 1998). The columns were in need of repair due to corrosion caused mainly by the proximity to the highway lanes as shown in Figure 16*a*. Five of the columns were wrapped using GFRP materials while the remaining four columns were wrapped by CFRP. The repair process is shown in Figure 16. Optical fibers were installed on four of the columns to monitor the behavior of the repair system. Since the system was installed primarily to enhance the durability of the members, the fiber optic sensors were monitored regularly and no signs of deterioration were observed for the retrofit columns.

Special Structures

The Gentilly-1 nuclear power plant, currently acts as a containment structure for a moderately contaminated nuclear reactor. The prestressed concrete containment structure was in good condition except for the secondary concrete. The secondary concrete fills the recesses and protects the terminations of the tendons against corrosion making it critical for the durability of the structure. Differential shrinkage caused cracking and debonding of the secondary concrete which had to be removed and replaced due to the effect of freeze-thaw cycling. The ring-beam at the top of the containment structure was severely affected because the numerous tendons of the roof terminate at that level. A retrofit of the ring beam was conducted in 2000–2001, as shown in

(a) (b) (c) (d)

FIGURE 16 Repair of concrete columns by wrapping with FRP sheets (*a*) damaged column; (*b*) column repaired by GFRP; (*c*) installation of CFRP sheets; and (*d*) column repaired by CFRP.

Figure 17, by first replacing the secondary concrete with high-quality shrinkage-compensated mortar and concrete, followed by FRP wrapping (Demers et al., 2004).

Repair of Metallic Structures

Due to the success of using externally bonded composites for the repair of concrete structures, the technology has been adapted for the repair of many metallic structures. Several applications have been reported throughout Europe and the United States in which metallic structures, including cast iron, wrought iron, and aluminum, were repaired using FRP materials.

A number of metallic bridge girders (Holloway and Cadei, 2002) and compression struts (Hill et al., 1999; Leonard, 2002) have been retrofit with FRP materials throughout Europe. In one application a historical cast-iron bridge, shown in Figure 18, was strengthened with composite materials to increase its capacity and to allow full highway loading of the structure (Holloway and Cadei, 2002). To accommodate the curved profile of the girders, wet lay-up of CFRP sheets was selected for the strengthening. The composites were cured at an elevated temperature between 50°C and 60°C. The CFRP was isolated from the cast iron using a polyester drape vale to prevent galvanic corrosion. At least two sets of European guidelines on the design and installation of composites for strengthening of metallic structures have been published (Moy, 2001; Cadei et al., 2004).

The k-joints of aluminum overhead sign structures suffer excessive fatigue induced cracking particularly at welded details. The full strength of the joints can be restored by externally bonding CFRP sheets to the joints (Fam et al., 2005). Externally bonded CFRP sheets were used to repair the cracked welded k-joints of an aluminum overhead sign structure in 2003. Longitudinal FRP layers were bonded to the diagonal truss members and wrapped around the main chord in alternating v-patterns. The diagonal truss members and the main chord were then

FIGURE 17 Repair of the Gentilly 1 nuclear containment structure.

FIGURE 18 The cast-iron Tickford Bridge in Newport Pagnell, United Kingdom (http://www.telfordsites.co.uk/telford/tickfdbr.html).

wrapped with additional circumferential layers of CFRP sheets to anchor the fibers and provide additional strengthening as shown in Figure 19.

VISION AND CHALLENGES

The future vision for the use of FRP materials for civil engineering infrastructure and transportation is underway as is evident by the amount of research currently in progress at many university research institutions and the private industry. This section of the paper reviews the current concepts and innovative approach under consideration as well as the future vision to achieve certain desired characteristics to optimize the use of these materials.

Strengthening of Steel Bridges

The vision of using FRP for the repair of steel structures began in the mid-1990s through research conducted at the University of North Florida (Sen and Libby, 1994) and at the University of Delaware (Mertz and Gillespie, 1996). This research demonstrated that externally bonded CFRP laminates could be used to repair damaged and deteriorated steel and steel-concrete composite girders. The use of CFRP materials as externally bonded reinforcement for steel structures was the focus of continued research efforts (Liu et al., 2001; Tavakolizadeh and Saadatmanesh, 2003; Patnaik and Bauer, 2004) as well as being the subject of field demonstration projects (Miller et al., 2001; Mosallam, 2005). Research indicates that the ultimate capacity of steel and steel-concrete composite beams can be increased substantially using externally bonded CFRP laminates.

The development of high-modulus CFRP (HM CFRP) materials has brought the vision of using composites for the retrofit of steel structures closer to reality. The use of HM CFRP is currently being investigated by the NSF Industry/University Cooperative Research Center on

FIGURE 19 Repair of an aluminum overhead sign truss structure (Fam et al., 2005).

Repair of Buildings and Bridges with Composites at North Carolina State University. The experimental research program included the selection of an appropriate adhesive to bond HM CFRP strips to steel structures (Schnerch, 2005), large-scale testing of steel-concrete composite beams strengthened with HM CFRP laminates, as shown in Figure 20 (Schnerch, 2005), and testing of strengthened beams under overloading and fatigue loading conditions (Dawood, 2005). The strengthened beams exhibited a reduction of the residual deflection of approximately four times under overloading conditions as compared to an unstrengthened beam at the same load level. The strengthened beams were capable of surviving 3 million loading cycles with an increase in the live load level of 20% as compared to an unstrengthened beam.

External Prestressed FRP

Research at various universities and research institutions is being conducted to consider the concept of prestressing FRP laminates before bonding to the tension side of steel and concrete structures. The development of a suitable anchorage system to apply the prestress force to the flat FRP laminates provides an interesting challenge which must be addressed before the technique can gain widespread use. Due to the flat configuration of the FRP strips, the techniques used to anchor steel tendons cannot be adopted for use with prestressed laminates. Also, since the anchorage can only be attached to one face of the FRP strips, the resulting prestress force is eccentrically applied to the laminate. Schnerch (2005) used a specially designed steel fixture to install a prestressed HM CFRP laminate onto the tension flange of a steel-concrete composite beam as shown in Figure 21. This technique shows promise to more effectively use the properties of FRP materials in flexural strengthening and rehabilitation applications.

FIGURE 20 Strengthening steel bridge girders with HM CFRP laminates.

FIGURE 21 Prestressing of externally bonded CFRP strips for a steel-concrete composite beam.

FRP Cables

There are a number of technical challenges which must be met before composites can be used for long span bridges. The development of FRP cables, such as the one shown in Figure 22, is one of the advancements which has been recently used for special applications. The cables, have a tensile capacity of up to 12 MN and are anchored using a specially designed anchorage (Meier, 2000). The anchorage consists of a conical steel shell which is used to form a molded epoxy cone on the end of the cable. The modulus of the epoxy is gradated along the length of the cone becoming softer near the tip to provide more consistent shear transfer along the length of the anchorage. FRP cables have been used in a number of demonstration projects throughout Europe, including as cable stays and as external prestressing for a number of bridges (Meier, 2000). In many of these applications the strands were outfitted with structurally integrated optical fibers, which were used to monitor the behavior of the cables. However, the high cost of these types of cables and the lack of long-term performance data makes their widespread use still only a vision.

Unbonded FRP Straps

A novel concept for an unbonded composite strap is being developed at the Swiss Federal Laboratory for Material Testing and Research in Switzerland (Meier, 2000). The straps consist of layers of thin, flexible CFRP thermoplastic tape which are wrapped as shown in Figure 23. The outermost layer is bonded to the second layer to form a closed loop. The inner layers remain unbonded allowing them to slide relative to one another until a uniform state of stress is achieved. The innovative system more effectively utilizes the capacity of the fibers by eliminating stress concentrations which can otherwise form in bonded laminates. The straps may be wound around pairs of circular pins to transfer tensile loads between adjacent components.

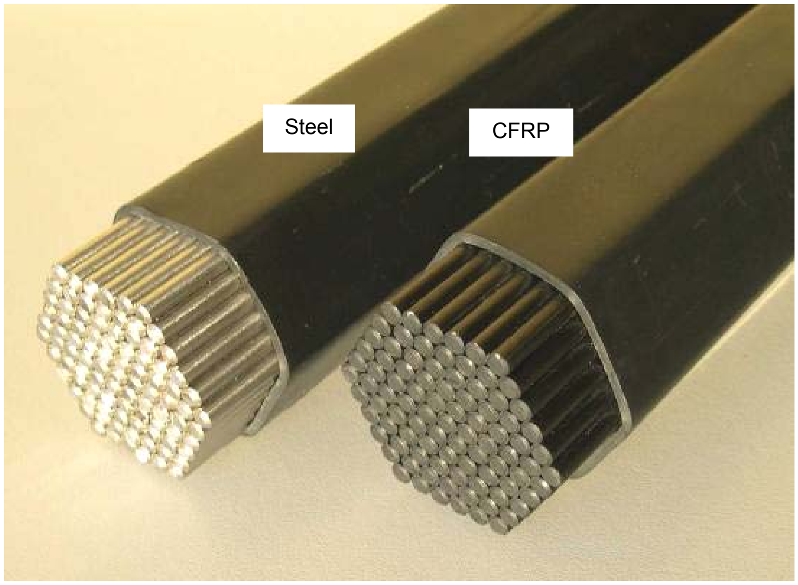


FIGURE 22 Steel and CFRP parallel wire strands.

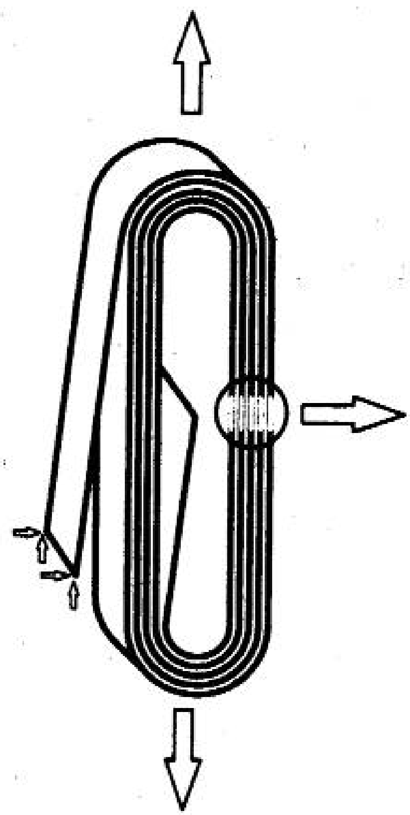


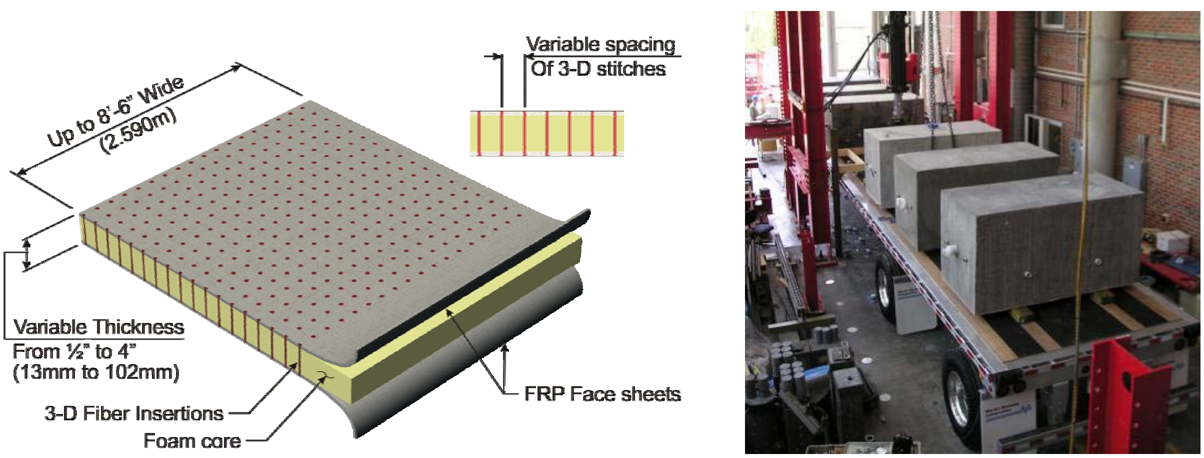
FIGURE 23 Schematic of novel unbonded FRP straps (adapted from Meier, 2000).

They have also been used as external prestressed shear reinforcement for concrete beams (Lees et al., 2000). Further development and analysis needs to be conducted before the system can be widely used for infrastructure applications.

FRP Sandwich Panels

The use of composite sandwich construction is gaining widespread use in civil infrastructure applications. Traditional composite sandwich panels exhibit low transverse stiffness, susceptibility to in-plane shear, face-to-core delamination and buckling instability. Stitched composite sandwich panels are currently being developed with the vision of improving on the effectiveness of conventional composite sandwich construction. The innovative panels incorporate through-thickness fibers that connect the top and bottom face sheets as shown schematically in Figure 24*a*. The three-dimensional fibers prevent delamination of the face sheets and increase the shear rigidity of the panel. The panels can serve in a variety of applications including as pedestrian bridge decks, trench covers, airfield matting and parking decks.

One potential application of the three-dimensional sandwich panels is as a lightweight construction material for tractor-trailers. An all-composite prototype trailer was constructed using the innovative new sandwich panel technology. The structural components of the deck and the main girders were fabricated completely from three-dimensional composite sandwich panels resulting in a significant reduction in the self-weight of the trailer as compared to conventional steel trailers. Testing of the trailer is shown in Figure 24*b*.

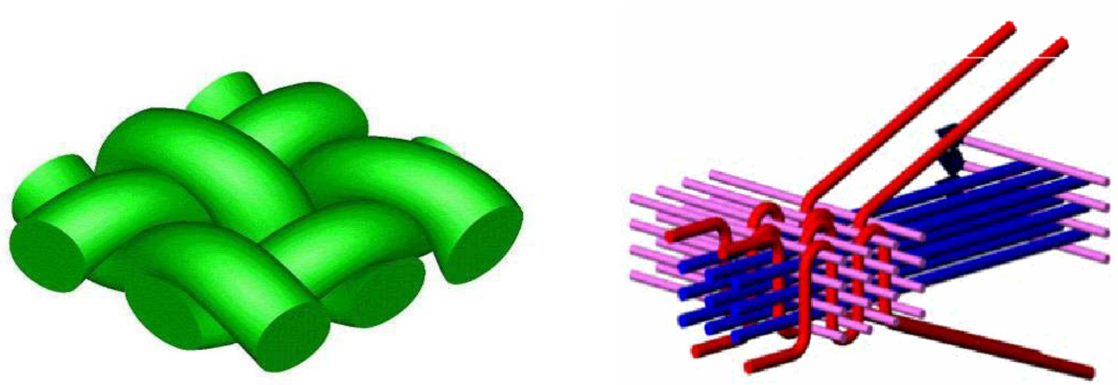


(a) (b)

FIGURE 24 Three-dimensional stitched-composite sandwich panels: (*a*) schematic of stitching configuration, and (*b*) testing of a prototype composite trailer.

Woven Fiber FRP Composite Bridge Decks

An innovative three-dimensional weaving process is under development at North Carolina State University. Conventional two-dimensional weaving can result in fiber waviness, as shown in Figure 25*a*, which can reduce the strength of the FRP material. The innovative new three-dimensional weaving process results in a composite structure in which fibers are un-crimped as shown in Figure 25*b*. The fibers in the third direction also act as through-thickness reinforcement that provides additional resistance to crack propagation and eliminates delamination as a failure mode. The vision of researchers is to utilize the advancements in three-dimensional weaving technology to develop an innovative GFRP bridge deck system. An experimental program is currently underway at the Constructed Facilities Laboratory at North Carolina State University to optimize the deck configuration to efficiently utilize the mechanical properties of the fibers. Fabrication of a prototype deck is shown in Figure 26.



(a) **(b)**

FIGURE 25 Two-dimensional and three-dimensional weaving technologies: (*a*) waviness of two-dimensional woven fibers, and (*b*) schematic of three-dimensional weaving process.

FIGURE 26 Three-dimensional weaving of a prototype glass fiber bridge deck.

Pseudo-Ductile FRP Composites

Researchers are investigating the use of pseudo-ductile hybrid FRP (HFRP) systems with the vision of mimicking the elastic-plastic tensile stress-strain relationship of conventional steel (Grace et al., 2005). HFRP woven fabrics consist of various glass, carbon and high modulus carbon fibers which can be oriented in different directions. The different failure strains and orientations of the various fibers result in a gradual failure of the hybrid system which is comparable to the elastic-plastic behavior of steel. When used as an externally bonded reinforcement for concrete structures, the longitudinal fibers primarily provide flexural strengthening for the structure while the diagonal fibers simultaneously provide shear strengthening. This eliminates the need for multiple layers of FRP with different fiber orientations. The diagonal fibers also help to anchor the longitudinal fibers thereby reducing the likelihood of debonding of the composite. The development of pseudo-ductile HFRP systems is a promising vision for the future of composites in infrastructure applications.

Thermomechanical Properties of Adhesives

Most epoxy adhesives have a glass transition temperature between 100°C and 150°C. While this is adequate for most infrastructure applications in the northern hemisphere, it is not sufficient for applications in hotter climates. Temperature variations can also dramatically affect the behavior of uncured adhesives. Allen et al. (1982) indicate that an increase of temperature of 8°C will approximately halve the cure time of an adhesive while a decrease of temperature of 8°C will approximately double the cure time. At very low temperatures, approaching 0°C, most thermoset adhesives may never cure (Schnerch, 2005). On the other hand, at very high temperatures epoxies may not remain workable long enough to allow proper installation of the composite strengthening. The development of adhesives and resins with higher glass transition temperatures and which are less sensitive to variations in cure temperature is a challenge which should be addressed.

Fire Resistant Composites

The development of fire resistant composites and resins is a vision shared by many researchers. When externally bonded FRP materials are exposed to open flames the organic resins and adhesives burn readily which can result in increased flame spread and the production of toxic smoke. Combustion of externally bonded FRP materials can also compromise the integrity of a structure potentially leading to collapse (Bisby et al., 2005). In the case of internal composite reinforcements the lack of oxygen prevents burning of the composites; however, the increase of the ambient temperature can cause softening of the matrix, which can lead to debonding and eventually collapse (Saafi, 2002). In a gap analysis conducted by Karbhari et al. (2003), the need to obtain a larger database of information on the behavior of composites exposed to fire was identified as a primary research objective. The development of resins and gel coats that can be used to enhance the fire resistance of composite systems was also identified as an important focus of future research. This vision could lead to substantial life savings and economic savings.

CONCLUSIONS

The use of composites in new construction and for strengthening of existing structures has become common practice worldwide. As engineers have gained confidence in the performance of composite materials, new advancements have emerged which more effectively utilize the properties of these promising materials. However, these technologies are still currently under development. While initial test results demonstrate the promise of these new technologies, they have not yet gained acceptance for widespread use in civil engineering infrastructure applications. Ongoing research and development is being conducted with the future vision of utilizing composite materials to their full potential. These new and novel applications indicate that there is a bright future for composites in civil engineering. However, there are still visions which need to be achieved to realize the full potential of these promising materials.

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