LSD2003: International Workshop on Limit State Design in Geotechnical Engineering Practice
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Implementation of the AASHTO LRFD Bridge Design Specifications for Substructure Design

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ABSTRACT: Load factor design (LFD) codes have been used in the US since the 1970's, and in the highway industry since 1977, but their application was limited to the design of superstructure components. Introduction of the AASHTO LRFD Bridge Design Specifications (*LRFD Specifications*, 2002) in 1994 represents the first instance in the US where load and resistance factor design (LRFD) was codified for the geotechnical design of highway substructure features. While a few state departments of transportation (DOTs) proactively pursued implementation of the *LRFD Specifications*, the process has been slow. This implementation has been slowed, in part, by resistance to change, lack of applicable software, and inadequate staff training. More importantly, a number of problems have arisen that can be attributed to changes in the *LRFD Specifications* that resulted in noticeable differences with past design practice. This paper highlights the challenges that have been encountered and the measures taken to correct and implement the *LRFD Specifications* to gain the confidence of highway owners and designers.

1 LRFD IMPLEMENTATION

The *LRFD Specifications* were first promulgated nearly 10 years ago. When initially published, the American Association of State Highway and Transportation Officials (AASHTO) envisioned full implementation by all DOTs by the year 2000. The leverage applied to drive implementation was the intention to sunset (i.e., no longer publish) the long-standing AASHTO Standard Specifications (*Standard Specifications*, 2002) that have provided national requirements for highway bridge superstructure and substructure design since the 1930s. However, this did not happen until 2002, and DOTs now have until 2007 to either fully-implement the *LRFD Specifications* or develop their own alternatives to them. As such, implementation of the LRFD Specifications remains transitional and spotty.

Since the mid-1990's, the Federal Highway Administration (FHWA) has supported LRFD implementation by developing and promoting two LRFD training courses (one for superstructure and the other for substructure design) intended for state DOT staff and their consultant designers. To date, nearly 40 superstructure and 50 substructure courses have been presented. In the process of teaching these courses, some knowledge was gained regarding the level of LRFD implementation. Based on empirical information learned from the substructure design course (Withiam, et al. 1998), reasons cited for delayed implementation include:

- Unwillingness to change from past practices that have worked well, especially when preferred design methods were not included in the *LRFD Specifications*
- Some aspects of the new code were inconsistent with or resulted in more conservative designs than past practice
- Lack of training
- Lack of design software

Implementation of the *LRFD Specifications* for superstructure design is much further along than for substructure design. Using information compiled by LEAP Software, 22 state agencies have already implemented the code, 14 have set a date for implementation, 12 are reviewing but have yet to set a date for implementation, and two have no plans as yet for implementation. Figure 1 illustrates the level of implementation for each state agency.

Because the NHI substructure course was presented between 1998 and 2002, the empirical information obtained from the course regarding LRFD implementation substructure design is somewhat dated. Therefore, a survey was taken recently of state DOTs to obtain a snap shot of the current level of LRFD implementation for substructure design. The survey form was sent by email to each state agency, and is reproduced here in Table 1. The information requested in the survey was limited to minimize the time needed to complete the form to less than five minutes and hopefully generate a reasonable number of responses.

Survey responses were received from 22 states within the two week time limit requested when the survey was mailed. Of the agencies responding, 10 were from agencies that have already implemented LRFD for design superstructures, or about 45 percent of the agencies nationally. Of these, five agencies stated that they design substructures using the *LRFD Specifications* with exception, and the remaining five continue to design substructures using the *Standard Specifications*. Some of the exceptions noted include:

- Design deep foundations using provisions developed by agency
- Design foundations using *LRFD Specifications* but design walls using *Standard Specifications*
- Design all substructures using *LRFD Specifications* but design MSE walls using *Standard Specifications*

The results of this recent survey are consistent with empirical information gathered during the substructure design course. Reasons for these trends include 1) differences between LRFD design practice with past practices, 2) problems associated with calibrating the *LRFD Specifications,* especially for deep foundations and some wall types; and 3) perceived problems due to a lack of understanding. These issues are addressed in the following sections.

2 CHANGES IN DESIGN PRACTICE

As an entirely new specification, development of the *LRFD Specifications* provided an opportunity to make changes in design methodology to reflect changes in observed structure behavior relative to past design practice. Perhaps the most noteworthy change was the development of an entirely new model for vehicular traffic, reflecting changes in current and anticipated traffic loading. However, some less obvious changes were made in the substructure design provisions that had a significant effect when the *LRFD Specifications* were first published. These include 1) the location of the resultant earth pressure used for retaining wall design, 2) the live load surcharge applied on the retained soil mass behind walls to simulate the effects of vehicular traffic behind the wall, and 3) application of the new vehicle live load model and load distribution through the soil cover above culverts. The rationale for these changes and their impact on the *LRFD Specification*s will be described.

2.1 Location of Resultant Lateral Earth Pressure

The magnitude of lateral earth pressure loads on a on a retaining wall or abutment is a function of:

- Structure type
- Type, unit weight, and shear strength of the retained earth
- Anticipated or permissible magnitude and direction of lateral substructure movement
- Compaction effort used during placement of soil backfill
- Location of the ground water table within the retained soil
- Location, magnitude, and distribution of surcharge loads on the retained earth mass

The stiffness of the structure and the characteristics of the retained earth are the most significant factors in the development of lateral earth pressure distributions. Walls that can tilt, move laterally, or

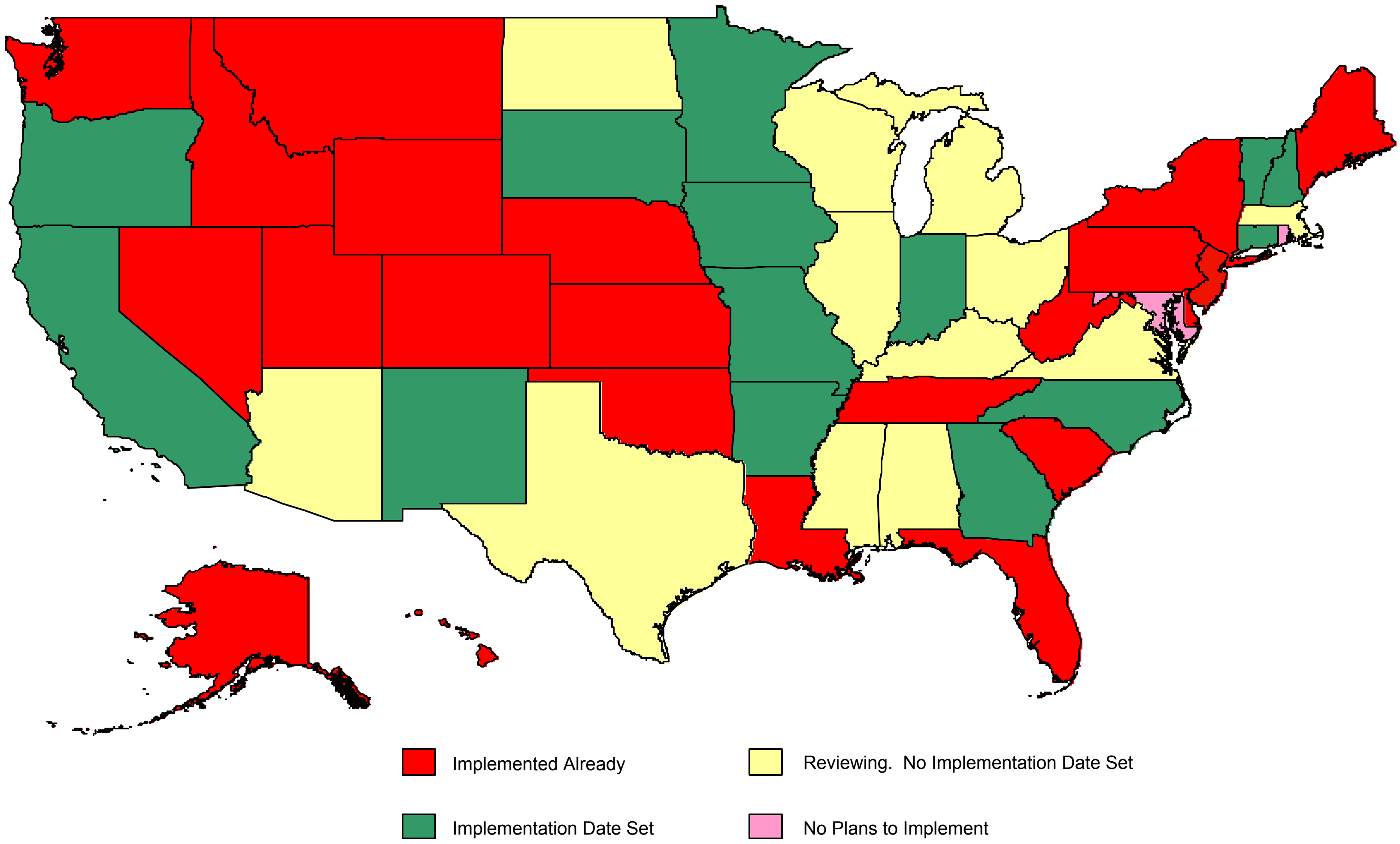


Figure 1. LRFD Implementation for Superstructure Design
(Modified after Leap Software, Inc., 2003)

Table 1.
Survey Form

I was responsible for developing training materials and was an instructor for the NHI course "LRFD for Highway Bridge Substructures." Before each course we surveyed the host agency to learn about their implementation of the AASHTO LRFD Specifications for Highway Bridges (*LRFD Specifications*). While the course was presented nearly 50 times to more than 30 agencies, nearly 5 years has passed since the first course was presented. Because much has changed we began, I ask you to take about 5 minutes to respond to the few questions below. I will use this information for part of a paper and presentation I will make in Boston next month at a workshop on limit states design in geotechnics. I will send a copy of the paper, "Implementation of the AASHTO LRFD Bridge Design Specifications for Substructure Design." Thank you.

1. What state agency do you represent? ______________________________

2. Your name: ______________________________

3. Your email address: ______________________________

If your agency ***HAS NOT*** yet implemented the AASHTO *LRFD Specifications*, please complete Question 4 and return the survey. If your agency has implemented the *LRFD Specifications*, please answer Questions No. 3 - 7 and return the survey.

4. If your agency ***HAS NOT*** implemented the *LRFD Specifications*, what year do you plan to start? ____________

5. If your agency ***HAS*** implemented the *LRFD Specifications*, what year did you start? ____________

6. Does your agency use the *LRFD Specifications* for substructure design? W/O exceptions; W/ exceptions; Highlight your response in **bold**.

7. If your agency takes exception to the *LRFD Specifications* for substructure design, highlight in bold those provisions you ***DO NOT*** use?

- Spread footing
- Driven pile
- Drilled shaft
- Conventional wall
- Prefabricated modular wall
- Flexible cantilever wall
- Anchored wall
- MSE wall
- Flexible culvert
- Rigid culvert

deflect structurally away from the retained soil (i.e., most retaining walls and abutments) can mobilize an active state of stress in the retained soil mass. These structures are typically designed using an active (i.e., minimum) lateral earth pressure distribution. Walls restrained against movement (e.g., integral abutments or walls for which lateral movement of the backfill could adversely affect nearby facilities) are typically designed to resist an at-rest earth pressure distribution. Walls forced to deflect laterally toward the retained soil are designed to resist the passive earth pressure. For practical purposes, the passive state of stress occurs most commonly as a result of lateral deflection of the embedded portions of retaining walls in the supporting soil. In the *LRFD Specifications*, passive earth pressure is treated as a resistance rather than a load.

The basic earth pressure, p, can be estimated using:

$$p = K_h \gamma_s' z$$

where:

p = Lateral earth pressure
K_h = Lateral earth pressure coefficient
γ_s' = Unit weight of soil
z = Depth from ground surface

The value of K_h used for design depends on the stress history of the soil (i.e., whether the soil is normally-consolidated [NC] or overconsolidated [OC]) and the displacement of the structure (i.e., whether the structure is flexible or stiff and whether soil loading is active or passive). The initial or at-rest value of K_h (i.e., K_o) ranges between about 0.4 and 0.6 for NC soils, and can exceed 1.0 for heavily OC soils. Structure movement will increase or decrease the value of K_h from K_o such that movement away from the soil will cause the value of K_h to decrease below K_o and movement toward the soil will cause the value of K_h to increase above K_o. Minimum and maximum values of K_h (i.e., K_a, and K_p) are mobilized when the shear strength of the soil is completely mobilized. For conventional walls (i.e., gravity, semi-gravity and inverted T-type cantilever), the lateral movement required to develop the minimum active earth pressure or maximum passive earth pressure is a function of the type of soil retained, as shown in Table 2.

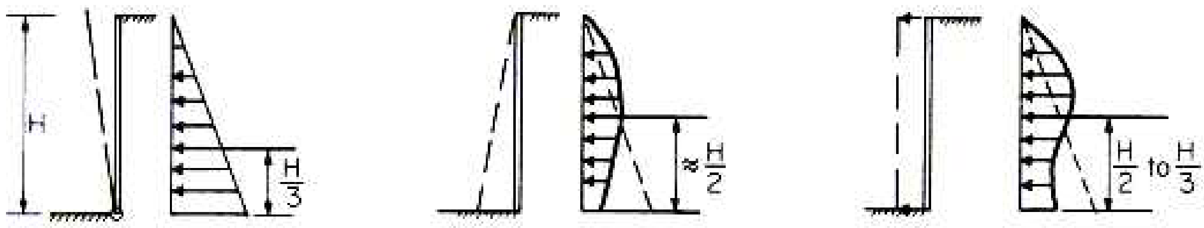
Table 2. Relative Movements Needed to Achieve Active or Passive Earth Pressure Conditions (Clough and O'Rourke, 1991)

Backfill Type	Values of Δ/H	
	Active	Passive
Dense Sand	0.001	0.01
Medium Dense Sand	0.002	0.02
Loose Sand	0.004	0.04

Δ = Lateral movement needed to mobilize active or passive earth pressure
H = Wall height

Nearly all cantilever retaining walls of typical proportions used for highway applications deflect sufficiently to permit mobilization of active earth pressures. Gravity and semi-gravity walls designed with a sufficient mass to support only active earth pressures will tilt and/or translate in response to more severe loading conditions (e.g., at-rest earth pressures) until stresses in the retained soil are relieved sufficiently to permit development of an active stress state in the retained soil. However, at-rest earth pressures could develop on the stem of cantilevered retaining walls where a rigid stem-to-base connection may prevent lateral deflection of the stem with respect to the base. For such a condition, excessive lateral earth pressures on the stem could conceivably cause structural failure of the stem or stem-to-base connection.

As shown in Figure 2a, the resultant force from a linearly increasing pressure distribution is located at the centroid of the pressure diagram at H/3 from the base of a wall which tilts about its base. However, if the wall tilts about its top or translates laterally as shown in Figures 2b and 2c, the location of the resultant force is higher than traditionally assumed for design. Location of the resultant force above the centroid of the pressure diagram occurs because as a wall deflects in response to lateral earth loading, the backfill must slide down along the back of the wall for the retained soil mass to achieve an active state of stress. This movement causes arching of the backfill against the upper portion of the wall which causes an upward shift in the location at which the resultant of the lateral earth load is transferred to the wall. Recognizing the possibility of these different responses to lateral earth loads, the *LRFD Specifications* prescribed that the resultant earth lateral pressure be located at 0.4H from the base of the unsupported wall section for conventional gravity and inverted T-type cantilever walls. For other wall types (i.e., nongravity cantilever or other types of flexible walls which tilt or deform laterally in response to lateral loading), significant arching of the backfill against the wall does not occur, so that the resultant lateral load due to active and other pressure distributions could be assumed to act at H/3 above the base of the wall.



(a) Tilting About Base (b) Tilting About Top (c) Translation

Figure 2. Location of Resultant Lateral Earth Pressure (Hunt, 1986)

The requirement that the resultant lateral earth pressure be located at 0.4H above the base of gravity and inverted T-type cantilever walls, (rather than H/3 as traditionally done), was controversial and led designers to question the validity of the *LRFD Specifications*. Designers soon realized that locating the resultant earth pressure at 0.4H resulted in an increase in the moment about the toe of 0.4H/0.33H or slightly more than 20 percent in lateral load effect compared to past design practice.

In response to numerous comments from DOTs and designers, AASHTO sponsored NCHRP 20-7, Task 88 (Withiam, et al., 1999) to reassess and update the provisions for wall design in the *LRFD Specifications*, and to conduct calibration analyses to incorporate performance data compiled since the code was originally calibrated. As part of that effort, the appropriateness of prescribing 0.4H as the location of the resultant lateral earth pressure was reevaluated. This study determined that while the resultant lateral load due to the earth pressure may act as high as 0.4H above the wall base for a conventional gravity retaining wall, such structures walls are not representative of the more flexible inverted T-type cantilever walls used in highway applications.

The flexibility of inverted T-type cantilever walls and the appropriateness of assuming K_a for the design were demonstrated in analyses reported in notes for NHI Course No. 13064 (1994). The results of a series of elastic structural analyses were presented for walls of typical proportions of stem thickness to wall height bearing on and supporting dense sand with a horizontal backslope. The walls analyzed varied in height from 1.5 to 9 m. The analyses also considered the effects of creep and cracking of the concrete stem. The results of these analyses were then compared with guidelines provided in the *LRFD Specifications*, where the lateral deflection required at the top of the wall stem to mobilize K_a for dense granular soil backfill is 0.001H. The results of the analyses are presented in Tables 3 and 4.

Table 3. Stem Deflections Using Full Section Modulus
(University of Maryland and NBE, Ltd., 1994)

Wall Height H (m)	Average Stem Thickness (m)	Estimated Stem Deflection, Δ		Deflection Required to Mobilize K_a Δ/H (dim)
		For K_a Δ/H (dim)	For K_o Δ/H (dim)	
1.5	0.12	0.00013	0.00021	0.001
4.5	0.30	0.00074	0.00119	0.001
7.5	0.50	0.00159	0.00255	0.001
9.0	0.60	0.00151	0.00241	0.001

Assumes $\phi_f = 37°$, c = 0, $K_a = 0.25$, and $K_o = 0.40$ for backfill soil; and $f'_c = 27.6$ MPa for concrete

Table 4. Stem Deflections Using Reduced Section Modulus Considering Cracking and Creep
(University of Maryland and NBE, Ltd., 1994)

Wall Height H (m)	Average Stem Thickness (m)	Estimated Stem Deflection, Δ		Deflection Required to Mobilize K_a Δ/H (dim)
		For K_a Δ/H (dim)	For K_o Δ/H (dim)	
1.5	0.12	0.00066	0.00106	0.001
4.5	0.30	0.00371	0.00594	0.001
7.5	0.50	0.00795	0.01273	0.001
9.0	0.60	0.00486	0.00779	0.001

Assumes $\phi_f = 37°$, c = 0, $K_a = 0.25$, and $K_o = 0.40$ for backfill soil; and $f'_c = 27.6$ MPa for concrete

Table 3 shows that lateral deflections are sufficient to mobilize K_a for walls greater than about 5.5-m high loaded by an active earth pressure distribution and for walls greater than about 4-m high loaded by an at-rest earth pressure distribution. Table 4 shows the effects of cracking and creep of concrete. The table shows that due to the reduced stiffness caused by cracking and creep of concrete in the wall stem, lateral deflections are sufficient to mobilize active earth pressures for walls greater than about 2.0-m high loaded by a K_a earth pressure distribution and for walls greater than 1-m high loaded by a K_o earth pressure distribution. Because the walls were analyzed as supported on a rigid base, the results ignore the effects of the lateral deflection along the base caused by differential pressure along the wall foundation which would also contribute to a reduction in the lateral pressure to active loading. So for this type of wall, designers can assume that the lateral deformations will be sufficient to mobilize active earth pressures.

The results of the evaluations and analyses described here resulted in a recommendation (Withiam, et al., 1999) to permit application of the resultant load from lateral earth pressure at H/3 for conventional retaining walls. AASHTO incorporated this recommendation in the 2002 Interims to the *LRFD Specifications*. However, additional commentary needs to be developed to clarify the application of compaction-induced lateral earth pressures and application of a load factor for this force effect.

2.2 Live Load Surcharge

When applicable, the force effects of surcharge and traffic loads on backfills must be considered for the design of walls and abutments. In the *LRFD Specifications*, if traffic is expected within one-half the height behind a wall, the live load traffic surcharge is assumed to act on the retained earth surface. To simplify consideration of traffic loads for wall design, AASHTO treats traffic loads as a uniform earth surcharge on the retained soil. The increase in lateral earth pressure due to live load surcharge is estimated as:

$$\Delta p = K_s \gamma'_s h_{eq}$$

where:

Δ_p = Constant lateral earth pressure due to uniform traffic surcharge
γ'_s = Effective unit weight of soil
K_s = Coefficient of earth pressure
h_{eq} = Equivalent height of soil for the design live load

For active earth pressure conditions, K_s is taken as K_a, and for at-rest conditions, K_s is taken as K_o.

The *Standard Specifications* prescribe that h_{eq} = 0.61 m. However this provision had remained unchanged for decades when vehicle loads were much lighter. As a result, development of the *LRFD Specifications* provided an opportunity to evaluate the appropriateness of this simplified approach. For example, because the weight of vehicle loads had increased over the years, was h_{eq} = 0.61 still a reasonable approximation? For another, because the lateral load effect from surcharges on retaining walls is greatest near the surface and diminishes nonlinearly with depth, was the simplified approach of a uniform lateral load effect regardless of wall height still reasonable? This approach was reevaluated as part of developing the new code and resulted in a provision in the *LRFD Specification* that h_{eq} be applied as a function of wall height, as shown in Table 5.

Table 5. Equivalent Height of Soil for Vehicular Loading (AASHTO, 1994)

Wall Height (m)	h_{eq} (m)
< 1.5	1.70
3.0	1.20
6.0	0.76
> 9.0	0.61

Values of h_{eq} in Table 5 were determined based on evaluation of horizontal pressure distributions produced on retaining walls from the updated vehicular live load model in the *LRFD Specifications* using a Boussinesq elastic half-space solution and a Poisson's Ratio of 0.5.

This new model for live load surcharge had a significant effect on the design of most walls. For a 6-m high wall, the new model resulted in a 25 percent increase in lateral load from vehicle loading compared to past practice, and for shorter walls, the effect was even greater. As a result, the base width of walls increased. Many wall designers believed the required increase in base width was due to the load and resistance factors in the new specification. But in fact the greater width was due to an updated treatment of live load surcharge using a new live load model to reflect the current weight and size of vehicle traffic. As a result, designers were suspicious about the new code.

In response to concerns from DOTs and designers, more refined analyses (Kim and Barker, 2002) were conducted to study the effect of vehicle loads on retaining structures. These subsequent analyses included several differences compared to the original analyses:

- Wheel loads were treated as uniformly loaded areas rather than point loads
- The loaded surface was analyzed as a two-layer (i.e., pavement and subgrade) rather than a one-layer system (i.e., subgrade only)
- Application of the AASHTO vehicle load models of truck or tandem plus lane load
- Separate treatment of loading parallel to structure (i.e., retaining wall) and perpendicular to structure (i.e., abutment)
- Values of Poisson's Ratio typical of soil backfill rather than ν = 0.5

The recommendations resulting from these analyses were introduced in the 1999 Interims to the *LRFD Specifications*. Table 6 shows that if vehicle wheel loads are applied at the back of a wall less than 6-m high, the equivalent load is greater than the value of h_{eq} = 0.61 used in the *Standard Specifications*. However, due to the presence of safety barriers near walls used for grade separations, it is unlikely that wheel loads could be applied at the back of a wall. Therefore, treatment of vehicle loads on retaining walls in the current *LRFD Specifications* is the same as in the *Standard Specifications*.

Table 6. Equivalent Height of Soil for Retaining Wall (AASHTO, 2002)

Wall Height (m)	h_{eq}; Δ from wall	
	0 m	1 m
1.5	1.52	0.61
3.0	1.07	0.61
> 6.1	0.61	0.61

Table 7 shows that the effect of vehicle wheel loads applied at the back of an abutment exceeds the value of h_{eq} = 0.61 used in the *Standard Specifications* for wall heights of 6 m or less. But because most DOTs use an approach slab that is partially supported on the abutment, the effects of vehicle loads are generally less than used before.

Table 7. Equivalent Height of Soil for Abutment (AASHTO, 2002)

Abutment Height (m)	h_{eq} (m)
1.5	1.22
3.0	0.91
6.1	0.61

So with the changes in the live load surcharge for vehicle loading promulgated with the 1999 Interims to the *LRFD Specifications*, the effect of vehicle loading on a retaining structure was then similar, if not identical, to past practice.

2.3 Live Load Distribution Through Earthfill

The design of buried structures (culverts) using the *LRFD Specifications* has resulted in more conservative designs (i.e., stronger cross section) for culverts with a shallow soil cover (≤ 3 m) compared to past practice using the *Standard Specifications*. The difference in designs using the two design codes is due to a difference in the way the effects of vehicle loading are transferred through the pavement/subgrade to the underlying culvert.

For culverts with a shallow soil cover (less than about 3 m), the effects of vehicle loading tend to control the structural design of the culvert cross section. For cover depths greater than 0.6 m, the *Standard Specifications* prescribe that surface point loads, such as from wheel loads, be spread through the soil cover over an area having sides equal to 1.75 times the depth of cover. The *LRFD Specifications* assume that for cover depths greater than 0.6 m, the contact pressure from a prescribed tire footprint is distributed through the soil backfill in a manner similar to the 60 degree (from horizontal) spreading rule found in many geotechnical textbooks. For depths less than 0.6 m, the area of the tire footprint itself is to be used to determine pressure below the surface due to live loads. At 1-m depth of cover, this modification has the effect of increasing design pressures for vehicle loading by 70% compared to the method used in the *Standard Specifications*.

As with other approximate methods for determining vertical earth pressures, the AASHTO procedures for spreading live loads through earth fills are intended to obtain force effects averaged across the culvert diameter. These procedures are used to calculate the average wall thrust due to vehicle live loads, but are not appropriate for determining concentrated force effects from live loads, such as bending stresses or localized deflections, because actual wheel loads do not distribute uniformly through the soil. Rather, wheel loads distribute more as predicted by elastic theory as shown in Figure 3. This figure shows that live loads produce much higher localized effects for shallow covers than predicted by the approximate average pressure models used in AASHTO. However, the peak pressures attenuate rapidly with depth. Thus, consideration of localized bending effects due to live loads is usually a concern only for culverts with shallow covers or those subjected to larger than typical concentrated live loads.

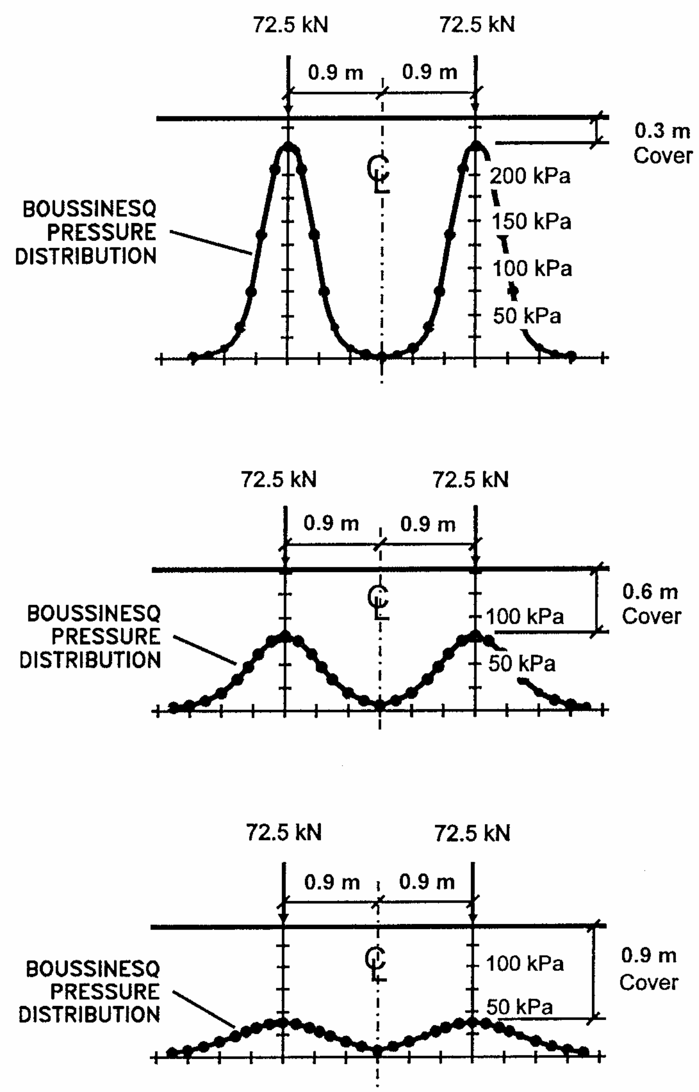


Figure 3. Design Truck Pressure Distribution From Elastic Theory

Other vehicle load effects which were modified in the *LRFD Specifications* include load factors, live load impact factors (now termed dynamic load allowance factors), multiple presence factors, and the design live load model itself. The *LRFD Specifications* have reduced the live load factor from 2.0 to 1.75, a reduction of 12 percent. However, both the magnitude and the effective depth of live load impacts have been increased. At 1 m of cover, the modification to the impact factor increases design pressures by over 125 percent. Also, the multiple lane presence factor has been increased by 20 percent for one lane contributions, although it was left unchanged for two lane contributions.

The net effect on design due to all of the factors is illustrated in Figure 4 for live load contribution from the design truck for a single lane contribution. As can be seen, the changes significantly increase the conservatism of the specification for live load design. For a 1-m cover, the required design pressure is about 100 percent greater for the *LRFD Specifications* than for the *Standard Specifications*.

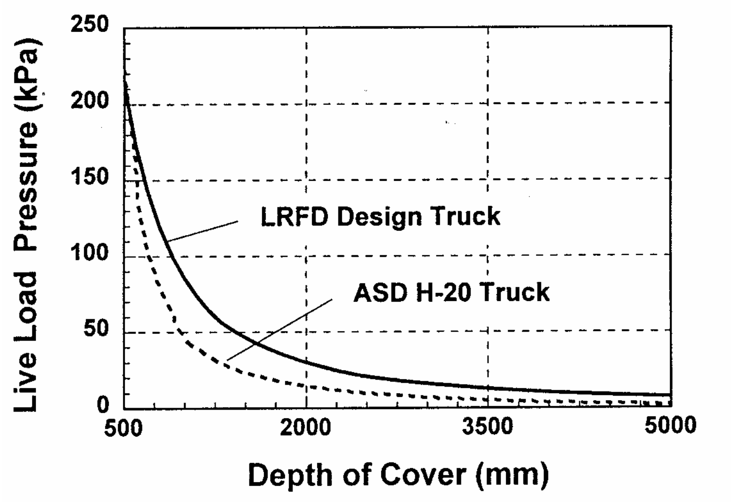


Figure 4. Comparison of AASHTO Live Load Pressures Through Earth Fills

While considerable effort was expended in developing the live load model in the *LRFD Specification*s, the appropriateness of these changes for below-ground structures was not evaluated. Further study is needed to refine the new LRFD live load model for the design of culverts.

3 CALIBRATION ISSUES

FHWA's Jerry DiMaggio has identified the following requirements for a design code:

- Must be complete
- All failure modes (limit states) must be addressed
- Guide a designer through the design process
- Must be unambiguous so that each competent user gets the same result
- Document development so it can be checked and modified in the future

While delayed implementation of the *LRFD Specifications* can be attributed to failure to meet some of these requirements, the *LRFD Specifications*, like other limit states codes, have had their growing pains for other reasons. Some can be attributed to difficulties encountered when the code was first calibrated during the early 1990's. For walls and abutments, difficulties were encountered by trying to attempt reliability-based calibrations with limited data. But the most problematic area has been the design of deep foundations, and in particular driven piles. Because such a large number of methods have been developed for the geotechnical design of driven pile foundations, calibration of the *LRFD Specifications* was limited to a few of the most commonly used methods. As a result, some methods used with success by some state agencies were not calibrated. Another impediment developed when designers learned that the calibrated resistance factors presented in the *LRFD Specifications* did not consider that driven pile design routinely integrates feedback from the monitoring of test piles during driving and load testing to confirm capacities estimated during design. In an attempt to correct this problem, a later change to the *LRFD Specifications* by AASHTO only exacerbated the situation which led to added confusion and mistrust of the code for substructure design. Similar, although far less troublesome concerns have been raised for other portions of the *LRFD Specifications* in the areas of drilled shaft, anchored, and MSE wall design.

3.1 Foundations

The *LRFD Specifications* include provisions for the design of spread footing, driven pile and drilled shaft foundations. For the most part the provisions for foundation design have not been change substantially since the code was introduced in 1994.

The design of spread footing foundations using the *LRFD Specifications* requires stability checks for bearing, sliding and eccentricity using resistance factors calibrated for each of these limit states. With the changes described previously in the location of the resultant lateral earth pressure and live load surcharge, footing designs using the *LRFD Specifications* are now comparable to designs developed in the past using the *Standard Specifications*. However one area of footing design that still needs to be addressed is the design of spread footings for serviceability. To be consistent with the provisions for the design of superstructure components, the *LRFD Specifications* prescribed that footings (and all other substructures as well) be designed for a load factor of 1 and a resistance factor of 1 for the Service I Limit State because these provisions in the code are not calibrated. This requirement usually is not critical for superstructure design where deformations can be reasonably estimated using elastic theory. But this is not the case for soil-structure interaction problems where deformations are often nonlinear and stress dependent. For footings in particular this is a problem because the footing designs are often controlled by serviceability rather than strength. As a result, footing designs can be controlled by uncalibrated design provisions in the *LRFD Specifications*. This problem will be addressed in an upcoming research project (see description of NCHRP 12-66 in Section 4) that should be underway in early 2004.

The portion of the *LRFD Specifications* that has been the most problematic is the design of driven pile foundations. The problems have been due in part to the large number of methods available for driven pile design, and the fact that pile design, at least for highway applications, entails input from both traditional geotechnical design methods and monitoring of piles during driving or load testing afterwards. When design provisions for driven piles were calibrated in the late 1980s and early 1990s, the data available for calibrations was much less than at present. Further, given the large number of methods available for static design, the methods calibrated for the *LRFD Specifications* were limited to those then most widely used based on a survey of state DOTs. As a result, methods used by some DOTs that had a lengthy experience base were not calibrated.

Another aspect of LRFD pile design that has caused some confusion is structural design. The structural design provisions for driven piles are found in Section 5 (Concrete), Section 6 (Steel) or Section 8 (Timber) depending on the material used for pile manufacture. This separation of geotechnical and structural resistance (capacity) differs from that used in the *Standard Specifications* where both were included in the provisions for pile design. As a result, the *LRFD Specifications* had separate resistance factors for axial loading, eccentric loading, and driving damage, but the code was offered no guidance on how these factors should be combined for design. The confusion caused by this presentation led one state agency to design a pile foundation bearing on rock solely for load eccentricity without considering the effects of axial loading and driving damage customarily considered in the *Standard Specifications*. This treatment has been clarified since the *LRFD Specifications* were first promulgated, but designers still express confusion with the process.

However the aspect of driven pile design that has caused the most confusion, and the aspect that probably has represented the greatest impediment to code implementation for substructure design, is the manner in which the calibrated resistance factors are used for design of the geotechnical axial capacity of driven piles. Beginning with the 1992 Interims to the *Standard Specifications*, AASHTO recognized the value of increasing levels of construction control in reducing the uncertainty in axial geotechnical pile capacity. As shown in Table 8, the *Standard Specifications* permit the minimum factor of safety to be reduced when more reliable methods are used in combination for design and construction control. However this feature of the *Standard Specifications* was not fully integrated in the *LRFD Specifications*.

Table 8. Factor of Safety on Ultimate Axial Geotechnical Capacity
Based on Level of Construction Control (AASHTO, 2002)

Basis for Design and Type of Construction Control	Increasing Design/Construction Control				
Subsurface Exploration	✓	✓	✓	✓	✓
Static Calculation	✓	✓	✓	✓	✓
Dynamic Formula	✓				
Wave Equation		✓	✓	✓	✓
CAPWAP Analysis			✓		✓
Static Load Test				✓	✓
Factor of Safety (FS)	3.50	2.75	2.25	2.00[1]	1.90

[1] For any combination of construction control that includes a static load test, FS = 2.0.

Table 9 presents the resistance factors in the *LRFD Specifications* for the ultimate axial geotechnical capacity of driven piles loaded in compression. The values of ϕ presented in Table 9 were calibrated assuming the axial geotechnical capacity is determined using just one method (e.g., capacity of a friction pile in clay based on static analysis for skin friction but without test monitoring during driving or load test after driving). Consequently, the added benefit of combining design methods with confirmatory testing during construction to reduce the uncertainty of the pile capacity used in the past was not carried forward into the *LRFD Specifications*.

Table 9. Resistance Factors for
Geotechnical Strength Limit State for Axially Loaded Piles (AASHTO, 2002)

Method/Soil/Condition		Resistance Factor
Ultimate Bearing Resistance of Single Piles	Skin Friction: Clay	
	α - method	$0.70\lambda_v$
	β - method	$0.50\lambda_v$
	γ - method	$0.55\lambda_v$
	End Bearing: Clay and Rock	
	Clay	$0.70\lambda_v$
	Rock	$0.50\lambda_v$
	Skin Friction and End Bearing: Sand	
	SPT-method	$0.45\lambda_v$
	CPT-method	$0.55\lambda_v$
	Skin Friction and End Bearing: All Soils	
	Load Test	$0.80\lambda_v$
	Pile Driving Analyzer	$0.70\lambda_v$

Recognizing this limitation, AASHTO prescribed a method for modifying ϕ by λ_v (see Table 9) beginning in 1997 to provide a method for designers to include combinations of design and construction control methods in the *LRFD Specifications*. Values of the modifier λ_v are shown in Table 10. But this adjustment caused greater confusion. Recalling that combining improved design and construction control test methods led to a reduction in factor of safety in the *Standard Specifications*, designers should have expected this corrective action in the *LRFD Specifications* would result in a higher resistance factor. In fact because values of λ_v are ≤ 1, the modified values of ϕ are either unchanged or less than the original prescribed resistance factors. So while this new approach was well intended, it caused designers to have even greater reservations about using the *LRFD Specifications* for substructure design.

Table 10. Values of λ_v (AASHTO, 2002)

Design/Construction Control Combination	λ_v
Driving formulas w/o stress wave	0.80
Wave equation analysis w/o stress wave	0.85
Stress wave on 2 - 5% w/ PDA	0.90
Stress wave on 2 - 5% w/ load test	1.00
Stress wave on 2 - 5% w/ PDA & CAPWAP	1.00
Stress wave on 10 - 70% w/ PDA	1.00

Drilled shafts were introduced in the 1992 Interims to the *Standard Specifications* and their use for highway applications has increased considerably since then. Design of drilled shafts using LRFD has been far less controversial than for driven piles, but some issues have caused concern. The major concern has been the lack of recommended resistance factors for the design of shafts in granular soils. As shown in Table 11, the LRFD Specifications provide resistance factors for the design of drilled shaft in cohesive soils and rock, but not in sand. At the time the *LRFD Specifications* were being developed in the late 1980s and early 1990s, the code developers had insufficient data to calibrate resistance factors for this case. Other concerns raised relate to a lack of guidance for the design of drilled shafts in intermediate geo-materials, consideration of the effects of construction procedures on the capacity of drilled shafts, and the influence of defects in the structural capacity of drilled shafts.

Table 11. Resistance Factors for Geotechnical Strength Limit States for Axially Loaded Drilled Shafts

Method/Soil/Condition			Resistance Factor
Ultimate Bearing Resistance of Single Drilled Shaft	Side Resistance - Clay	• α-method (Reese & O'Neill, 1988)	0.65
	Base Resistance - Clay	Total Stress (Reese & O'Neill, 1988)	0.55
	Side Resistance - Sand	Various	-(1)
	Base Resistance - Sand	Various	-(1)
	Side Resistance - Rock	• Carter & Kulhawy (1988) • Horvath & Kenney (1979)	0.55 0.65
	Base Resistance - Rock	• CGS (1992) • Pressuremeter Method (CGS, 1992)	0.50 0.50
	Side & Base Resistance	Load Test	0.80

(1) ϕ factors not been developed for shafts in cohesionless soils due to a lack of adequate field data.

Since the *LRFD Specifications* were promulgated, many of the problems identified here have been addressed as part of research efforts sponsored by NCHRP, FHWA and state DOTs. For example research has been completed to recalibrate the provisions for driven pile and drilled shaft foundations (NCHRP 24-17) and publication of FHWA Geotechnical Circular No. 6: Shallow Foundations (Kimmerling, 2003). However their implementation in the code is still some years away as provisions need to be developed and approved by AASHTO before they can be published and used by designers.

3.2 Walls

The *LRFD Specifications* include provisions for the design of conventional (i.e., gravity, semi-gravity, counterfort and inverted T-type) walls, prefabricated modular (i.e., bin) walls, flexible cantilever (i.e., soldier pile) walls, anchored walls and mechanically stabilized earth (MSE) walls. Unlike the provisions for foundation design, the provisions for wall design have undergone considerable change in recent years to address shortcomings in the initial version of the *LRFD Specifications*.

Soon after the *LRFD Specifications* were introduced, concerns were raised regarding the application of the resultant lateral load and equivalent live load surcharge. These issues and their resolution were described previously. But other issues that needed to be addressed included:

- Inconsistency in treatment of internal stability of MSE wall reinforcements between the *LRFD Specifications* and changes promulgated in 1997 in the *Standard Specifications* to reflect a change in FHWA recommended design methodology
- No design provisions for flexible cantilever walls
- Source and appropriateness of load factors for vertical and horizontal (lateral) earth loads and earth surcharge load effects
- Treatment of overall slope stability

Following implementation of recommendations from NCHRP 20-7, Task 88 in the 2002 Interims to the *LRFD Specifications*, the first two bulleted items have been addressed. The results of NCHRP 12-55 to calibrate load factors for vertical and horizontal earth loads and earth surcharges will address the third item. The last item may be addressed as part of NCHRP 24-21 to develop recommended LRFD design and construction specifications for soil nailed walls.

4 CORRECTIVE ACTIONS AND UPDATES

As with any well maintained design or construction code, the *LRFD Specifications* have undergone both minor and major changes to correct problem areas and have been supplemented with new technical content and recalibrated load and resistance factors as new performance data or design methods become available. For the most part, these efforts have been undertaken by the National Cooperative Highway Research Program (NCHRP), FHWA, state DOTs, most notably those of Florida, Pennsylvania and Washington, and industry sponsors. The AASHTO-sponsored efforts through NCHRP related to highway substructures exceed $4M in support since 1994 when the *LRFD Specifications* were first promulgated. (Note: Total funding through NCHRP for LRFD-related related efforts exceed $10M.) Activities have ranged from general efforts to maintain the code and targeted projects in the areas of structure foundations, retaining walls and abutments, seismic design and culverts. Completed, on-going and planned NCHRP projects relevant to substructure design include:

<u>General</u>

- *NCHRP 12-42, "LRFD Bridge Design Specifications Support"*: The objective of this project is to provide timely assistance to the AASHTO Subcommittee on Bridges and Structures in interpreting, implementing, revising, and refining the *LRFD Specifications*. (On-going)

- *NCHRP 20-5, Topic 28-02, "Survey of Limit State Design Practice"*: This project documents a survey of U.S., Canadian and international transportation agencies to review the development and implementation of geotechnical related LRFD specifications compared to the *LRFD Specifications*. Results published in NCHRP Synthesis 276. (Completed)

Foundations

- *FHWA DTFH61-00-C-00031, Geotechnical Engineering Circular No. 6*: FHWA's primary reference of recommended design and procurement procedures for shallow foundations that presents state-of-the-practice guidance on the design of shallow foundation support of highway bridges. Circular presents detailed design examples for shallow foundations using ASD and LRFD methods.

- *NCHRP 24-17 "LRFD Deep Foundation Design"*: The objective of this project was to provide recommended revisions to the driven pile and drilled shaft portions of Section 10 of the *LRFD Specifications* to reflect current best practice in geotechnical design and construction. (Completed)

- *NCHRP 12-47 "Redundancy in Highway Bridge Substructures"*: The objective of this project was to develop a methodology for considering substructure redundancy in the design and evaluation of highway bridges to extend the methodology developed in NCHRP Project 12-36, "Redundancy in Bridge Superstructures," to bridge substructures. Results published in NCHRP Report 458 (Completed)

- *NCHRP 12-66, "Calibration of AASHTO LRFD Specifications for Serviceability in the Design of Bridge Foundations and Substructures"*: The objective of this project will be to develop calibrated load and resistance factors for the serviceability design of foundations and substructures, and development of recommended revisions to the *LRFD Specifications*. (Planned)

- *Guide LRFD Design and Construction Specifications for Micropile Foundations*: Industry initiative sponsored by the Deep Foundations Institute (DFI) and International Association of Foundation Drilling (ADSC). The specifications were developed using the same framework as LRFD design and construction specifications, incorporate design and construction recommendations in FHWA-SA-97-070, and the resistance factors in the guide design provisions were calibrated by matching to current AASHTO LRFD and FHWA ASD/LFD design practice.

Walls and Abutments

- *NCHRP 20-7, Task 88, "Develop New AASHTO LRFD Specifications for Retaining Walls"*: This project resulted in recalibration of resistance factors for wall design and development of recommended revisions to Section 10 for walls in the LRFD Specifications that included design provisions for flexible cantilever walls (e.g., soldier pile walls) and updated methods for assessing internal stability for MSE walls. The recommended revisions were promulgated in the 2002 Interims to the *LRFD Specifications*. (Completed)

- *NCHRP 12-55, "Load and Resistance Factors for Earth Pressures on Bridge Substructures and Retaining Walls"*: The objective of this project is to develop recommended load factors for vertical earth loads, lateral earth loads and earth surcharge loads for foundations and retaining walls for possible inclusion in the *LRFD Specifications*. (On-going)

- *Project 24-22, "Selecting Backfill Materials for MSE Retaining Walls"*: The objective of this project will be to develop selection guidelines, soil parameters, testing methods, and construction specifications that will allow the use of a wider range of backfill materials within the reinforced zone of mechanically stabilized earth (MSE) retaining walls. (On-going)

- *NCHRP 24-21, "LRFD Soil-Nailing Design and Construction Specifications"*: The objective of this research is to develop recommended LRFD design and construction specifications for soil-nailed retaining structures. (On-going)

Seismic Loading

- *NCHRP 12-49 "Comprehensive Specification for the Seismic Design of Bridges"*: The objective of this project was to enhance safety and economy through the development of LRFD specifications and commentary for the seismic design of bridges. The research considered design philosophy and performance criteria, seismic loads and site effects, analysis and modeling, and design requirements. The specifications are nationally applicable with provisions for all seismic zones. The recommended provisions are an AASHTO guide specification. Results published in NCHRP Report 472. (Completed)

- *NCHRP 12-70, "Seismic Analysis of Retaining Walls, Buried Structures, and Embankments"*; The objective of this project will be to develop recommended specifications for the seismic design of retaining walls, buried structures and embankments consistent with the format of the *LRFD Specifications* and the seismic provisions for highway bridges. (Planned)

Culverts

- *NCHRP 12-45, "Recommended Specifications for Large-Span Culverts"*: The objective of this project was to develop recommended design and construction specifications for metal and concrete large-span (3 – 9 m) culverts. Results published in NCHRP Report 473 but have yet to be incorporated in the *LRFD Specifications*. (Completed)

- *NCHRP 20-7, Task 89, "LRFD Specifications for Plastic Pipe and Culverts"*: The objective of this research was to develop design provisions for inclusion in the *LRFD Specifications*, based on the state of the art for this material. Results published in NCHRP Report 438. (Completed)

- *NCHRP 4-26, "Thermoplastic Drainage Pipe, Design and Testing"*: The objectives of this research are to develop a recommended LRFD specification for thermoplastic pipe for culverts and drainage systems for consideration by AASHTO; and to develop a QA/QC procedure to test manufactured thermoplastic pipe. (On-going)

As highlighted in the bulleted items, most of these projects have a stated objective of developing recommended changes to the *LRFD Specifications*. While not all of these efforts have been fully adopted, the studies provide the technical committees of AASHTO with the rationale and technical bases for changes to the code.

5 CLOSING

The *LRFD Specifications* are a work in progress, and this is certainly true in the area of substructure design as summarized here. Possibly more important, implementation has been slowed by resistance to change, lack of applicable software, and inadequate staff training. In fact similar growing pains have been encountered by limit states design codes developed throughout the world during the past two decades. So while the *LRFD Specifications* have admittedly encountered implementation problems since 1994, the code continues to evolve, grow and hopefully improve.

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