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Implementation Status of Geotechnical Load and Resistance Factor Design in State Departments of Transportation

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Preface

The Bridge Design Specifications published by the American Association of State Highway and Transportation Officials (AASHTO 1994, 1998) has introduced the Load and Resistance Factor Design (LRFD) method to account for uncertainties associated with estimated loads and resistances in bridge design. Most state departments of transportation implemented the specification by designing the bridge superstructures using the LRFD method while still using the Allowable Stress Design (ASD) method for the bridge foundation design. The Federal Highway Administration and AASHTO set a transition date of October 1, 2007, after which all federally funded new bridges including substructures shall be designed using the LRFD method.

The issues faced by the states as they implement LRFD for geotechnical design are varied in type and consequence. LRFD is a more detailed framework for design than ASD or Load Factor Design (LFD), requiring greater attention to more items, which often puts greater demands on data gathering of the required input. More analysis also requires more staff time, as does the greater interaction and iteration between geotechnical and structural practitioners. The geotechnical resistance factors themselves, developed from readily available information and distributed nationwide, may not yet reflect the empirical accuracy of local practices. Many states have noted significant differences between LRFD-designed foundations and successful installations designed with earlier design guidelines. LRFD promises refinement of geotechnical design practice, but that refinement comes at a cost of greater initial effort, time to become familiar with a new practice, and the work required to hone LRFD factors to reflect the knowledge that many states currently possess in their existing design systems.

In this context, the Transportation Research Board's Foundations of Bridges and Other Structures Committee invited several state structural and geotechnical engineers to participate in a workshop at the 2008 TRB Annual Meeting. The objective of this workshop was to present various states' experiences in the practical implementation of LRFD in geotechnical and substructure design. Both geotechnical and structural perspectives were included. Presentations included case histories of actual implementation of LRFD for a bridge project and state experiences with implementation of LRFD into everyday design of substructures from a policy and procedure point of view. This circular presents summaries of four state department of transportation presentations given at that workshop. It is hoped that the experiences presented in this circular may be beneficial to agencies still working through their own LRFD implementation efforts. The contents of the papers presented are the opinions of the authors and not endorsed by the Transportation Research Board or AASHTO.

The TRB Foundations of Bridges and Other Structures Committee sincerely thanks the authors for their contributions to both the workshop and this circular. The committee also acknowledges the efforts of the fellow state highway agencies who peer reviewed the document and provided valuable suggestions. Finally, we thank TRB staff representative G. P. Jayaprakash for providing continued support and assistance during delivery of the workshop and development of this circular.

—Mark J. Morvant, Chair
Foundations of Bridges and Other Structures Committee

Contents

Louisiana Department of Transportation and Development's Experience with LRFD

ARTHUR D'ANDREA
CHING-NIEN TSAI
Louisiana Department of Transportation and Development

The Louisiana Department of Transportation and Development (LA DOTD) has adopted the AASHTO LRFD bridge design specifications as the method for designing new structures. Although LA DOTD began looking at this new method in 1995, it was not until the improved AASHTO Section 10 of these specifications was adopted in 2005 that our department moved to integrate both substructure and superstructure using the LRFD design code.

LRFD implementation is in process. Its success hinges on the department's ability to complete the calibration of resistance factors using local geotechnical practices. Success also depends on our ability to educate both state engineering staff and consultants on the concept and procedures of LRFD. Additionally, the LA DOTD is challenged with adopting a method and procedure that serves both the conventional contracting methods and the new design–build approach. The following sections provide a brief summary of the actions taken thus far by the LA DOTD in LRFD implementation.

LRFD IMPLEMENTATION PLAN AND INITIAL CHALLENGES

LA DOTD created an implementation committee that included members from its engineering staff, academia, and the Federal Highway Administration division bridge engineer. LA DOTD staff members included representatives from structures, geotechnical, rating, and document publishing sections. The committee created and acted on the following items**:**

- Defined implementation strategies for the design of substructure and superstructure;
- Established a plan for staff and consultant training;
- Evaluated available software;
- Defined needs for update of manuals standards and software;
- Established an implementation schedule and deadlines;
- Sought assistance from FHWA Resource Center; and
- Communicated plan to administration, FHWA, and other affected entities.

Many challenges were confronted as the plan was implemented. Most of the structural portions of the LRFD were defined first. Due to a lack of experience with the LRFD in the geotechnical area, initial LRFD projects used it for superstructures and the allowable stress design (ASD) for substructures. Initial challenges also included the following:

- Working through the implementation and phasing out the metric system;
- Making available LRFD software tools for project mass production;
- Comprehending the new versus the old methodology; and
- Understanding the process from design to construction.

The design engineers developed equivalent safety factors as part of the learning process. They needed to evaluate the LRFD equivalent factor of safety for various analysis and testing methods. As part of the learning process, equivalent safety factors were developed. Table 1 shows the LRFD equivalent factor of safety developed for driven piles.

LA DOTD phased in the new design method by first using ASD for the substructure and LRFD for the superstructure. Reviewing the equivalent safety factors, the structural designer's concern was that the factors of safety fell below the value of 2.0. Whether or not they were appropriate, comparisons were made between the ASD and LRFD resulting pile loads, factors of safety (FS), and resistance factors (ϕ) as shown below:

- Typical 70′ span substructure loads
  - ASD with FS = 2, pile load = 340 tons
  - LRFD with ϕ = 0.7, pile load = 390 Tons, 15% more, higher substructure resistance demand
- Long span bridges 470′ main span
  - ASD with FS = 2, pile load = 330 tons
  - Same pile layout LRFD ϕ = 0.7, pile load = 260 tons, 20% less than the ASD method

EXPERIENCE WITH LOCAL PROJECTS

LA DOTD learned valuable lessons through the implementation of LRFD for local projects. LA DOTD initial projects used the AASHTO national ϕ factors. Dynamic testing became more prevalent with pile bents and column bents using friction and bearing piles. High site variability was mitigated by geotechnical investigations that included deep borings, cone penetration tests (CPTs), pile driving analyzers (PDAs), statnamic load tests, and static load tests. In those cases, the ϕ values varied from 0.65 to 0.80.

The I-10 twin-span reconstruction project in New Orleans was one of the first major projects to use LRFD design in Louisiana. Figure 1 shows the I-10 twin-span project.

LRFD increased the early data exchange between the structural engineers and the geotechnical engineers. It also demanded that engineers rationalize the concept of site variability in the establishment of frequency for both the soil analysis and load testing. The project geotechnical engineers communicated with the designer on geotechnical parameters through tables is shown in Table 2.

LOCAL LRFD CALIBRATION

The LA DOTD in conjunction with Louisiana Transportation Research Center (LTRC) began the calibration effort in July 2006, and the driven pile calibration is now completed. The planned future calibration effort includes drilled shafts and possibly retaining walls. The calibration of the drilled-shaft design is in progress. Due to a very small database of completed drilled-shaft load tests in Louisiana, the Mississippi Department of Transportation has graciously provided its load test database. The drilled-shaft calibration is scheduled for completion in the summer of 2009.

The pile load test database used for the calibration was established by conducting an extensive search in the LA DOTD project library. Only the precast, prestressed concrete (PPC) piles that have been tested to failure and have adequate soil information were included in the

TABLE 1 LRFD Equivalent Factor of Safety for Driven Piles

LRFD Equivalent Factor of Safety for Driven Piles (Axial Compression)																		
R=DI/LL	Φ factor for Dynamic Analysis and Static Load Test Method (LRFD Table 10.5.5.2.3-1)											Φ factor for Static Analysis Methods (LRFD Table 10.5.5.2.2-1)						
	Static Load Test and quality control by dymanic testing (varies between 0.6 to 0.9 per No. of load test and site variability)							Dynamic Test	Wave equation anaylsis w/o pile dynamic measurement or load test	FHWA - modified Gates dynamic pile formula	ENR dynamic pile formula	α method	β method	λmethod	Nordlund/Thurman method	SPT method	CPT	End Bearing in rock
	0.9	0.85	0.8	0.75	0.7	0.65	0.55	0.65	0.4	0.4	0.1	0.35	0.25	0.4	0.45	0.3	0.5	0.45
0.5	1.8	1.9	2	2.1	2.3	2.4	2.9	2.4	4	4	15.8	4.5	6.3	4	3.5	5.3	3.2	3.5
1	1.7	1.8	1.9	2	2.1	2.3	2.7	2.3	3.8	3.8	15	4.3	6	3.8	3.3	5	3	3.3
2	1.6	1.7	1.8	1.9	2	2.2	2.6	2.2	3.5	3.5	14.2	4	5.7	3.5	3.1	4.7	2.8	3.1
3	1.5	1.6	1.7	1.8	2	2.1	2.5	2.1	3.4	3.4	13.8	3.9	5.5	3.4	3.1	4.6	2.8	3.1
4	1.5	1.6	1.7	1.8	1.9	2.1	2.5	2.1	3.4	3.4	13.5	3.9	5.4	3.4	3	4.5	2.7	3
5	1.5	1.6	1.7	1.8	1.9	2.1	2.4	2.1	3.3	3.3	13.3	3.8	5.3	3.3	3	4.4	2.7	3
6	1.5	1.6	1.7	1.8	1.9	2	2.4	2	3.3	3.3	13.2	3.8	5.3	3.3	2.9	4.4	2.6	2.9
7	1.5	1.5	1.6	1.8	1.9	2	2.4	2	3.3	3.3	13.1	3.8	5.3	3.3	2.9	4.4	2.6	2.9
8	1.5	1.5	1.6	1.7	1.9	2	2.4	2	3.3	3.3	13.1	3.7	5.2	3.3	2.9	4.4	2.6	2.9
9	1.4	1.5	1.6	1.7	1.9	2	2.4	2	3.3	3.3	13	3.7	5.2	3.3	2.9	4.3	2.6	2.9
10	1.4	1.5	1.6	1.7	1.9	2	2.4	2	3.2	3.2	13	3.7	5.2	3.2	2.9	4.3	2.6	2.9
11	1.4	1.5	1.6	1.7	1.8	2	2.3	2	3.2	3.2	12.9	3.7	5.2	3.2	2.9	4.3	2.6	2.9
12	1.4	1.5	1.6	1.7	1.8	2	2.3	2	3.2	3.2	12.9	3.7	5.2	3.2	2.9	4.3	2.6	2.9
13	1.4	1.5	1.6	1.7	1.8	2	2.3	2	3.2	3.2	12.9	3.7	5.1	3.2	2.9	4.3	2.6	2.9
14	1.4	1.5	1.6	1.7	1.8	2	2.3	2	3.2	3.2	12.8	3.7	5.1	3.2	2.9	4.3	2.6	2.9
15	1.4	1.5	1.6	1.7	1.8	2	2.3	2	3.2	3.2	12.8	3.7	5.1	3.2	2.8	4.3	2.6	2.8
16	1.4	1.5	1.6	1.7	1.8	2	2.3	2	3.2	3.2	12.8	3.7	5.1	3.2	2.8	4.3	2.6	2.8
17	1.4	1.5	1.6	1.7	1.8	2	2.3	2	3.2	3.2	12.8	3.7	5.1	3.2	2.8	4.3	2.6	2.8
18	1.4	1.5	1.6	1.7	1.8	2	2.3	2	3.2	3.2	12.8	3.6	5.1	3.2	2.8	4.3	2.6	2.8
19	1.4	1.5	1.6	1.7	1.8	2	2.3	2	3.2	3.2	12.8	3.6	5.1	3.2	2.8	4.3	2.6	2.8
20	1.4	1.5	1.6	1.7	1.8	2	2.3	2	3.2	3.2	12.7	3.6	5.1	3.2	2.8	4.2	2.5	2.8

LRFD Equivalent Factor of Safety = (1.25DL+1.75LL)/[Φ(DL+LL)] = (1.25R+1.75)/[Φ*(1+R)]
R = Ratio of DL/LL
DL = Dead Load
LL = Live Load
Based on the cleaned-up version of the balloted LRFD Section 10
Updated 10/10/05

FIGURE 1 I-10 twin-span reconstruction project in New Orleans.

study. A total of 42 PPC pile load tests met these criteria. In addition to the load test results, all other relevant information including soil borings, pile driving logs, dynamic testing and analysis, and CPT data were collected. It is important to note that more than 90 percent of the pile load test data is from south Louisiana, where the soil is weaker than that from north Louisiana. The majority of the soil profiles in the database consist of clays, although approximately 25 percent of them are in mixed soils of sand and clay. Statistical analyses were used to evaluate the different pile-design methods that include static design methods, direct CPT design methods, and dynamic measurement methods. In addition, reliability analyses based on various methods were conducted to calibrate the resistance factors (φ) for each design method. The results of the calibration are published in TRB (*11*). The resistance factors from the calibration using first order second moment (FOSM) are about 10% lower than those from the calibrations with the more sophisticated methods such as the first order reliability method (FORM) or Monte Carlo method. Due to the limited soil types represented in the collected pile load tests, the database contains only a few piles driven into sandy soils. Unlike the AASHTO factors that separate sands from clays, the LA DOTD's resistance factor for the static method combines both soil types.

Limited data are available for the stiffer soils. Therefore, care should be exercised if applying these resistance factors to those stiffer soils typically found in north Louisiana. This deficiency will necessitate more load tests on the stiffer soils in order to provide an adequate database to facilitate an adequate calibration. For this purpose, at least for the next few years, more load tests will be included in the larger projects located in north Louisiana soils. Comparing the resistance factor for the static method, the LA DOTD calibration resulted in resistance factors approximately 25 percent to 60 percent greater than the resistance factors from

TABLE 2 FB-Pier Soil Input Parameters: I-10 Twin-Span Project (B-25 – STA 349 + 11)

Soil Layer	Bottom Elevation (ft)	Soil Type	Soil Model [1]	Water Elevation [2] (ft)	Total Unit Weight (pcf)	N	ϕ (degrees)	Subgrade Modulus - K (pci)	Cohesion - c (psf)	ε_{50}	Poisson's Ratio - υ	Young's Modulus - E (ksi)	Shear Modulus - G (ksi)	Vertical Failure Shear (psf)	Torsional Shear Stress (psf)	Axial Bearing Failure [3] (kips)
0	-9.5															
1	-18.5	cohesive	Clay (Soft<Water)	0	120	--	--	20	150	0.028	0.40	0.3	0.1	75	75	5.4
2	-29.5	cohesive	Clay (Soft<Water)	0	114	--	--	133	1,000	0.010	0.45	1.2	0.4	950	950	81.0
3	-33.5	cohesive	Clay (Soft<Water)	0	116	--	--	57	430	0.016	0.40	0.4	0.1	439	439	34.8
4	-39.5	cohesive	Clay (Soft<Water)	0	120	--	--	91	680	0.012	0.40	0.6	0.2	665	665	55.1
5	-45.5	cohesive	Clay (Stiff<Water)	0	113	--	--	266	2,000	0.007	0.45	4.8	1.7	1,385	1,385	162.0
6	-57.5	cohesionless	Sand (Reese)	0	120	51	40	121	--	--	0.35	10.6	3.9	3,795	3,795	1,481.4
7	-71.5	cohesive	Clay (Soft<Water)	0	110	--	--	67	500	0.014	0.45	0.4	0.1	500	500	40.5
8	-75.5	cohesive	Clay (Soft<Water)	0	127	--	--	112	840	0.011	0.40	0.9	0.3	829	829	68.0
9	-81.5	cohesive	Clay (Soft<Water)	0	119	--	--	133	1,000	0.010	0.45	1.2	0.4	1,015	1,015	81.0
10	-93.5	cohesive	Clay (Soft<Water)	0	123	--	--	67	500	0.014	0.45	0.4	0.1	500	500	40.5
11	-98.5	cohesionless	Sand (Reese)	0	120	9	28	38	--	--	0.40	1.9	0.7	2,374	2,374	119.9
12	-126.5	cohesionless	Sand (O'Neill)	0	120	60	40	139	--	--	0.35	12.5	4.6	10,293	10,293	2,417.4
13	-144.5	cohesive	Clay (Stiff<Water)	0	110	--	--	240	1,800	0.007	0.45	3.9	1.3	1,696	1,696	145.8
14	-156.5	cohesive	Clay (Stiff<Water)	0	109	--	--	306	2,300	0.006	0.45	6.5	2.2	1,815	1,815	186.3
15	-161.5	cohesive	Clay (Soft<Water)	0	120	--	--	67	500	0.014	0.40	0.4	0.1	500	500	40.5
16	-167.5	cohesive	Clay (Stiff<Water)	0	100	--	--	492	3,700	0.005	0.45	17.6	6.1	1,360	1,360	299.7
17	-175.5	cohesive	Clay (Stiff<Water)	0	120	--	--	226	1,700	0.007	0.45	3.4	1.2	1,620	1,620	137.7
18	-181.5	cohesive	Clay (Soft<Water)	0	124	--	--	80	600	0.013	0.40	0.5	0.2	600	600	48.6
19	-189.5	cohesive	Clay (Stiff<Water)	0	120	--	--	200	1,500	0.008	0.45	2.7	0.9	1,500	1,500	121.5
20	-199.5	cohesionless	Sand (Reese)	0	120	65	38	149	--	--	0.35	13.5	5.0	18,435	18,435	2,417.4

[1] Soil model designations are from the FB Pier Input Screen. Please reference the FB Pier Manual for a discussion of the specific method referenced.
[2] Water elevation is assumed to be an average of 0 ft. at the site.
[3] Axial bearing failure is calculated for a 36-inch square concrete pile.

the AASHTO calibration. The equivalent safety factor for the static method is about 2.6— a value that is similar to the safety factors used in ASD practice that are between 2.5 and 3.0. This comparison should provide the geotechnical engineer with a level of comfort in adopting the AASHTO LRFD method, while it also allows the application of a risk level based on the importance of the project. The calibrated dynamic resistance factor is much lower than providedfor by the AASHTO calibration. This is probably due to the dynamic analysis being less effective in mostly clayey soils.

STATUS AND IMPACT FOR GEOTECHNICAL LRFD IMPLEMENTATION

The transition from the ASD method to the LRFD method will require changes in all facets of the geotechnical engineering practice, including exploration, testing, design, and construction. The changes are necessitated due to the concept of risk control, the desire to obtain better data, and the philosophical change to increase verification testing during construction. Compared with the traditional LA DOTD ASD field exploration practices, the AASHTO LRFD method increases the effort slightly because of the greater load demand coming from the increase of truck load from HS-20 to HL-93. Prior to LRFD, the standard practice for the shear strength tests for LA DOTD was the unconfined compressive strength tests, because the AASHTO 1996 specifications did not address the type of testing required. LA DOTD has long recognized the shortcomings of the uniaxial unconfined compression tests, but has not made the unconsolidated undrained triaxial (UU) test a standard practice prior to implementing LRFD. AASHTO LRFD's requirement of triaxial tests such as the UU test or the consolidated undrained triaxial (CU) tests has accelerated the transition into making UU test a standard. In addition to the need for more field exploration and a more stringent testing method, more careful data analysis is also required to analyze site variability and more quality-control testing will be needed.

As the results of these changes, LA DOTD is embarking on a major effort to write a geotechnical design manual and rewrite the *2006 Louisiana Standard Specifications for Roads and Bridges*. A draft of the geotechnical design manual is near completion. Modifications to section 804 (driven piles) of the standard specifications have been completed. The changes are primarily the inclusion of resistance factors for the static and dynamic load tests and a shift in the responsibility for data interpretation from the field project engineer to the geotechnical design engineer.

As stated previously, the bridge specifications require deeper soil borings and CPT soundings and the use of triaxial strength tests rather than the traditional uniaxial, unconfined compressive strength tests. The intent of these requirements is to improve the quality of the data collected and thereby improve the reliability of the capacity prediction otherwise provided by using the traditional methods. This change will lengthen the project duration to allow for geotechnical study and thereby impact project schedules and costs. Typically, triaxial tests cost twice that of uniaxial tests. The additional soil borings, or CPT soundings, and testing requirement increase the project costs for the geotechnical study by at least 50 percent, depending on the project size and structure types. Other cost increases include engineering analysis and construction testing. The demand on the resources will not be limited to exploration and laboratory testing but also the resulting engineering evaluations required. In addition to evaluating static soil properties, the site variability study will become a standard practice as part of the LRFD method. This will require more time for the design of foundation systems.

Training

The engineers in LA DOTD have taken at least one National Highway Institute (NHI) class training on the LRFD design of substructures and designed several projects using the LRFD method since its use went into effect in 2008. Even with this training and experience, there are many issues of uncertainty such as site selection and site variability determinations.

The geotechnical consultants may face a greater challenge and more obstacles without the training that provides a better understanding of LRFD principles. For example, a geotechnical consultant working on a LA DOTD project decided to arbitrarily use *high* site variability in the design of pile foundations without going through the site variability evaluation process as a more conservative approach. This is somewhat analogous to using the common practice in the ASD method of arbitrarily selecting a safety factor. However, it is not an acceptable practice because a basic principle of the LRFD method is to design for a pre-determined risk level. By arbitrarily selecting the site variability, the risk level of the design was unknown, thereby violating a basic principle of the LRFD method.

Resources

Geotechnical engineering resources are limited and will be strained due to the requirements for verification tests for either static or dynamic load tests. For example, the number of dynamic load tests required for a driven pile project increases significantly from the ASD to the LRFD practice. LA DOTD currently has the capability of operating two pile driving analyzers (PDAs) simultaneously. However, the additional PDA work will drain all the resources available for it. Compounding this problem is the requirement of additional geotechnical engineering hours to support the LRFD method. The additional engineering support services required for the LRFD method will make it increasingly difficult to meet project needs with the current staffing level.

The geotechnical consultant community has similar limitations. A limited number of qualified consultants have the experience with PDA testing and the associated Case Pile Wave Analysis Program (CAPWAP) analysis. It will be difficult to meet the demand and maintain the quality of work if workload is expanded beyond the capacity of the local geotechnical engineering community.

Complexity

Another task is to train the field personnel. Using the ASD method, field decisions can be made relatively easily. However, using the LRFD method, decisions to accept or reject a load test must be evaluated with the information from the previous load tests and consideration for the method of analysis on which the resistance factors are based. LRFD makes it much more difficult to make this kind of decision in the field.

Quality

The largest impact of the LRFD method implementation is the increased cost of conducting geotechnical engineering investigation. Almost all phases of geotechnical engineering are impacted by the conversion to the LRFD method. With the current trend of reducing government size, the existing LA DOTD staff has to cope with the increasing workload from surplus fund projects and expanded work scopes. The chance of increasing staff for the LA DOTD is minimal at best. More than likely, staffing levels will continue to be reduced. The outcome is clear. More outsourcing will result. Unfortunately, private consultants have only minimal exposure to the LRFD. They will certainly require more oversight for the next several years, resulting in a greater demand on LA DOTD engineering resources. The likely immediate outcome to this dilemma will be increased construction costs and a reduction in the quality of geotechnical engineering.

Future Goals

The primary LA DOTD Goals in use of the LRFD method are as follows:

- Continue to improve the LRFD design and calibration by taking advantage of detailed local data collected for the different methods and tests;
- Strive for consistent reliability for the entire structure in all possible failure modes;
- Maintain uncompromised the safety and integrity of the AASHTO code and bridge design life in the face of escalating construction costs;
- Improve the code to account for the appearance of complex partnerships and new methods of contracting, construction, and ownership;
- Develop construction quality control procedures and documents; and
- Maintain or increase staffing level to the increased demand resulting from LRFD implementation.

Benefits

The benefits from implementing the LRFD method include the following:

- Consistency in geotechnical design because of the uniform reliability design;
- Increased interaction between field inspectors and designers; and
- Improved quality of construction due to the increased field testing requirements.

At the early stages of LRFD implementation, there has been occasional confusion. However, these and other benefits have been observed from completed or ongoing LRFD projects. One of the greatest advantages is the increased communication among the different design disciplines of structure, geotechnical, and hydraulic engineering. For example, the geotechnical engineers simply provided pile capacities or plan order lengths when designing with ASD. Using LRFD, the geotechnical engineers have to understand the sources of loading components and design limit states in order to provide an adequate foundation design.

Finally, the role of the geotechnical engineers in bridge design and construction has come of age. With the intent of improving efficiency and safety, it is paramount that this specialized discipline within the civil engineering profession be allowed to contribute to bridge design. The geotechnical engineers will now share in the risks by taking responsibility for the design alongside the structural designer. Unlike past regional efforts, the LRFD code provides a national basis for all designers and builders to produce better designs as the aging bridges are replaced in the nation’s transportation system.

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Florida Department of Transportation's Experience with LRFD

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In 1987, AASHTO-initiated research on the application of LRFD to bridge foundation design. The research was administered by the National Cooperative Highway Research Program (NCHRP) to develop NCHRP 24-4 and was summarized in report NCHRP 343, titled, "Manuals for the Design of Bridge Foundations." Florida Department of Transportation (FDOT) first reviewed the research in 1992. However, the priority then was on converting the department's practices to metric units. When the LRFD specifications were available in 1994, the FDOT looked at this issue more seriously. In early 1995, when FHWA's LRFD training class was available via satellite broadcasting, FDOT had its engineers trained in some of those sessions. After the training, FDOT drafted an implementation plan in late 1995 and set up a timeline for the plan. The preliminary timeline was as follows:

- Oct 1995—Implementation Preparation
- July 1997—Training
- July 1998—Implementation

IMPLEMENTATION PREPARATION

The first step for the LRFD implementation was to (1) convert design documents to LRFD; (2) modify all software to the LRFD environments; and (3) calibrate geotechnical resistance factors for Florida foundations.

Document Conversion

The two main documents in the FDOT Structures Design Office to be converted to LRFD were the "Structures Design Guidelines" (SDG) and the "Soils and Foundation Handbook" (SFH). The SDG sets forth the basic FDOT design criteria that are exceptions to those included in AASHTO LRFD Bridge Design Specifications. These exceptions may be in the form of deletions from, additions to, or modifications of the AASHTO LRFD specifications. The SDG also sets forth the foundation design criteria for bridges as well as miscellaneous structures; e.g., mast arms, overhead signs, etc., and retaining walls design. The SFH is intended to define the tasks involved in performing subsurface investigations and geotechnical aspects of the design and construction of roadways and highway structures. There were also minor changes in the "Florida Standard Specifications for Roads and Bridges" and "CADD Standards."

Software Modification

To facilitate the LRFD implementation, all Florida DOT developed computer design software had to be modified. Among them were the geotechnical engineering software such as FL-Pier (now FB-MULTIPIER) and SPT94 (SPT97 or FB-DEEP). Florida DOT also requested its design software suppliers to modify or switch the design to LRFD.

Calibration of Geotechnical Resistance Factors for Florida Foundations

Resistance factors used for geotechnical engineering applications are related to

- Type of foundations and earth retaining structures;
- Soil type–geologic formation;
- Method of field exploration and laboratory testing; and
- Design methodology.

After reviewing the 1994 version of AASHTO LRFD "Specifications for Bridge Design," FDOT decided to contract with the University of Florida to calibrate the resistance factors for deep foundations for the following reasons:

1. AASHTO (NCHRP 24-4) Resistance factors were calibrated for design methods commonly used in many parts of the country and for geologic formations generally applicable to most of the country.
2. Florida has its unique geological formations; e.g., soft vuggy limestone, ball-bearing fine sands, etc.
3. Methods of analyses that FDOT used are not included in the AASHTO LRFD specifications; e.g., SPT94 (SPT97/FB-DEEP) for piles, McVay's method for drilled shafts socketed in soft vuggy limestones, etc.

Therefore, adopting the AASHTO LRFD specifications without considering items 2 and 3 would not lead to the intended level of structural reliability.

Methods Used for Calibration

There are two calibration methods: fitting with allowable stress (ASD) and reliability theory. FDOT's approaches were first to fit with ASD and then to calibrate using reliability theory.

Fitting with ASD

This method is usually used to determine or calibrate resistance factors when a database or case histories are not available. In ASD design, the factor of safety is applied to the sum of unfactored loads and can be expressed as

$$\frac{R_n}{FS} \geq Q_D + Q_L \rightarrow R_n \geq FS(Q_D + Q_L) \tag{1}$$

where R_n is the nominal resistance , Q_D and Q_L are nominal values of dead and live load, and FS is the factor of safety.

In the LRFD design, the concept of partial factors is applied to the loads. With only dead and live loads:

$$\phi R_n \geq \gamma_D Q_D + \gamma_L Q_L \rightarrow \phi \geq \frac{\gamma_D Q_D + \gamma_L Q_L}{R_n} \tag{2}$$

where R_n is the nominal resistance, ϕ is the resistance factor; Q_D and Q_L are dead and live load, respectively; γ_D and γ_L are respective dead and live load factors. Substitute the R_n with the R_n of equation 1 to get the resistance factor, ϕ, in terms of FS:

$$\phi \geq \frac{\gamma_D Q_D + \gamma_L Q_L}{FS(Q_D + Q_L)} \rightarrow \phi \geq \frac{\gamma_D (Q_D / Q_L) + \gamma_L}{FS(Q_D / Q_L + 1)} \tag{3}$$

Using the AASHTO respective dead and live load factors of 1.25 and 1.75 together with various safety factors and dead to live load ratios a number of resistance factors were obtained and are listed in Table 1.

CALIBRATION USING RELIABILITY THEORY

FDOT used reliability theory based on the first order second moment (FOSM) method to do the calibration of the resistance factors to a given set of load factors. The calibration consisted of the following steps:

1. Estimate the resistance factors in ASD design methods by fitting with the factors of safety to avoid drastic deviation from the past safe and satisfactory practices;
2. Consider loads, Q, and resistance, R, are random variables and $Q < R$. The probability of failure is

$$P_f = P(R < Q) = P[(R - Q) < 0] \tag{4}$$

TABLE 1 Resistance Factors Obtained Using AASHTO Dead and Live Load Factors of 1.25 and 1.75 with Various Safety Factors and Dead to Live Load Ratios

Q_D/Q_L	Resistance Factor, ϕ			
	FS = 1.5	FS = 2.0	FS = 2.5	FS = 3.0
1	1.00	0.75	0.60	0.50
2	0.94	0.71	0.57	0.47
3	0.92	0.69	0.55	0.46
4	0.90	0.68	0.54	0.45
5	0.89	0.67	0.53	0.44
6	0.88	0.66	0.53	0.44
7	0.88	0.66	0.53	0.44
8	0.87	0.65	0.52	0.44
9	0.87	0.65	0.52	0.43

3. Assuming R and Q to be log-normal distributions equation 4 can be rewritten as

$$P_f = P[\ln(R/Q) < 0)] = 1 - f\left[\frac{\ln(\bar{R}/\bar{Q})\sqrt{(1+V_Q^2)/(1+V_R^2)}}{\sqrt{\ln[(1+V_R^2)(1+V_Q^2)]}}\right] - \qquad (5)$$

where $\overline{R}$ and $\overline{Q}$ are the means of resistance and loads, respectively; V_R and V_Q are coefficients of variations of R and Q.

4. Instead of specifying a probability of failure, the reliability is expressed in terms of a reliability index, β

$$\beta = \frac{\ln(\bar{R}/\bar{Q})\sqrt{(1+V_Q^2)/(1+V_R^2)}}{\sqrt{\ln[(1+V_R^2)(1+V_Q^2)]}} \qquad (6)$$

The approximate probability of failure correspondent to the reliability index is listed as follows:

Reliability Index, β	2.0	2.5	3.0	3.5	4.0	4.5
Probability of Failure, P_f	8.5×10^{-2}	0.99×10^{-2}	1.15×10^{-3}	1.34×10^{-4}	1.56×10^{-5}	1.82×10^{-6}

1. Observe the resistance factor versus safety factors with different bridge span lengths, dead–live load ratios, type of foundations, and design methods (Table 1).
2. Select a target reliability index based on the level of safety or probability of failure used in the current design method.

$$\phi = \frac{\lambda_R(\gamma_D Q_D / Q_L + \gamma_L)\sqrt{(1+V_{QD}^2+V_{QL}^2)/(1+V_R^2)}}{(\lambda_{QD} Q_D / Q_L + \lambda_{QL})\exp[\beta_T\sqrt{\ln[(1+V_R^2)(1+V_{QD}^2+V_{QL}^2)]}]} \qquad (7)$$

Calibration for Driven Piles

Using the above methods together with the FDOT's and University of Florida's deep foundation databases, resistance factors were calibrated for the following pile foundation design and construction methods:

- SPT 94/SPT97 (FB-DEEP)—FDOT Pile Design Software
- Dynamic Load Testing using PDA
- Static Load Testing including Osterberg Cell Load Test and Statnamic Load Test

Results of the calibration are shown in Table 2. The uplift calibration was done by fitting with ASD design because of lack of data.

TABLE 2 Resistance Factors for Driven Piles

Design	Method	ASD		Target Reliability Index, β_T	Resistance Factor, ϕ
		FS	β		
Compression	SPT94/SPT97	2.0	2.45–2.57	2.5	0.65
	PDA (EOD)	2.5	3.04–3.14	2.5	0.65
	WEAP	3.0	2.28–3.11	2.5	0.35
	Static Load Test	2.0	–	2.5	0.75
Uplift	SPT94/SPT97	2.0	–	–	0.55
	Static Load Test	2.0	–	–	0.65
Lateral	Structural Stability	–	–	–	1.0

Calibration for Drilled Shaft

Using the same procedure and databases, FDOT calibrated drilled shafts for bridge foundation that are

- Founded in both soils and rocks (FHWA for soils, and McVay for rock);
- Socketed in rock only (McVay); and
- Subjected to Static Load Test (including Osterberg Cell and Statnamic Load Tests).

The results of the calibrations are shown in Table 3. The uplift calibration was done by fitting with ASD design because of insufficient load test data.

Drilled shafts are used as foundations for high mast light poles, sign structure, and signal structure. These miscellaneous structures are relatively low dead load but subjected to high wind load, lateral, and torsional loads. However, the databases do not contain load tests for these loading conditions; therefore, a calibration by ASD fitting was used to determine the resistance factors for lateral load and torsional resistance of drilled shafts for these types of structures (see Table 4).

Tables 2, 3, and 4 were included in the FDOT's structures design guidelines for all bridge foundation as well as miscellaneous structures design. Resistance factors for shallow foundations and retaining wall systems were not calibrated, and AASHTO's resistance factors were used.

Training

Starting in July 1997, FDOT provided six two-day training sessions at various cities in Florida. The target audience for the training was bridge designers, FDOT engineers, and

TABLE 3 Drilled Shafts for Bridge Foundations

Design	Method	ASD		Target Reliability Index, β_T	Resistance Factor, ϕ
		FS	β		
Compression (in rock)	Side friction including soil	2.5	4.45–4.60	4.0	0.6
	Side friction	2.5	3.73–3.83	3.5	0.65
	Side friction and 1/3 end bearing	2.5	3.49–3.59	2.5	0.55
	Static Load Test	2.0	–	2.5	0.75
Uplift	Same as side friction only	2.5	–	–	0.5
Lateral	Structural Stability	–	–	–	1.0

consultant engineers who do business with FDOT. Therefore, the topics of the training included loads, concrete, steel, and geotechnical design. However, the trainings were in metric units. Sample design problems were also provided in the training for various topics of bridge design.

Implementation

The timeline date for LRFD implementation was July 1998. The plan was to implement it in metric units but in March 1998 the department decided to convert the newly implemented metric units back to English units for all structures designed in Florida. So the implementation was postponed until all the design documents converted back to English units. Furthermore, FDOT realized that LRFD designs were not available for curved girder and movable bridges and did not have software for mechanically stabilized earth (MSE) and cast-in-place walls. Therefore, the decision was made to use load factor design (LFD) to design the curved girder bridge and movable bridges, and ASD for designing MSE retaining walls.

TABLE 4 Drilled Shafts for Miscellaneous Structures

Design	Method	ASD		Target Reliability Index, β_T	Resistance Factor, ϕ
		FS	β		
Compression*	Same as for bridge foundations	2.5	–	–	0.55
Uplift*	Same as for bridge foundations	2.5	–	–	0.5
Lateral Load	Brom	1.5	–	–	0.9
Torsion	FDOT Structures Design Office	1.0	–	–	1.0

*Use McVay's side friction method.

TABLE 5 Resistance Factors for Driven Piles:
FDOT Structures Design Guidelines, 2007

Loading	Design Method	Construction QC Method	Resistance Factor, ϕ
Compression	FB-Deep Davisson Capacity	PDA and CAPWAP	0.65
		Static Load Testing	0.75
		Statnamic Load Testing	0.70
Uplift	FB-Deep Skin Friction	PDA	0.55
		Static Load Testing	0.65
Lateral (Extreme Event)	FBPier[1]	Standard Specifications	1.00
		Lateral Load Test[2]	1.00

[1] Or comparable lateral analysis program.
[2] When uncertain soil conditions are encountered.

In 2002, with the exception of the steel curved girder bridge, still using LFD and MSE walls and the ASD method, FDOT completed LRFD implementation.

In 2003, FDOT completed the resistance factor calibration for the statnamic load test, and in 2005 a resistance factor for nonredundant drilled-shaft foundation was added to the structures design guidelines. The latest resistance factors, 2007 version, for piles and drilled shafts since then are shown in Tables 5 and 6, respectively.

TABLE 6 Resistance Factors for Drilled Shafts:
FDOT Structures Design Guidelines, 2007

Loading	Design Method	Construction QC Method	Resistance Factor, ϕ[6] Redundant	Resistance Factor, ϕ[6] Non-redundant
Compression	For soil: FHWA alpha or beta method[1]	Std Specifications	0.60	0.50
	For rock socket: McVay's method[2] neglecting end bearing	Standard Specifications	0.60	0.50
	For rock socket: McVay's method[2] including 1/3 end bearing	Standard Specifications	0.55	0.45
	For rock socket: McVay's method[2]	Statnamic Load Testing	0.70	0.60
	For rock socket: McVay's method[2]	Static Load Testing	0.75	0.65
Uplift	For soil: FHWA alpha or beta method[1]	Standard Specifications	Varies[1]	Varies[1]
	For rock socket: McVay's method[2]	Standard Specifications	0.50	0.40
Lateral[3]	FBPier[4]	Standard Specifications or Lateral Load Test[5]	1.00	0.90

[1] Refer to FHWA-IF-99-025, soils with N<15 correction suggested by O'Neill.
[2] Refer to FDOT "Soils and Foundation Handbook."
[3] Extreme event.
[4] Or comparable lateral analysis program.
[5] When uncertain conditions are encountered.
[6] These resistance factors can be found at http://www.dot.state.fl.us/structures/StructuresManual/CurrentRelease/StructuresManual.htm.

Finally, FDOT mandated the use of LRFD design for all MSE and cantilever walls on July 1, 2007. To facilitate the implementation, FDOT provided two EXCEL spreadsheets for the retaining walls design. The spreadsheets analyzed for AASHTO limit states Strength 1-a, 1-b, and IV. However, the bearing resistance factors that FDOT used are slightly deviated from AASHTO 2007 specifications. The differences are as follows:

These retaining walls programs were written with the modification of bearing resistance factor:

- AASHTO All retaining walls $\phi b = 0.45$ (SPT)
- FDOT MSE walls $\phi b = 0.55$
- Cantilever walls MSE $\phi b = 0.50$

These factors were obtained by using the ASD fitting method.

RECENT WORKS ON RESISTANCE FACTORS

Refinement of resistance factors is an ongoing work. The work focuses not only on the existing factors but also on new foundation types and new calibration methods, e.g., spatial variability, etc. Recently FDOT finished a research study on cylinder pile design considering plugged–unplugged conditions. The resistance factor calibration was part of the research outcome and has been implemented by coding in the FB-DEEP program.

There are several other research projects related to resistance factors currently in progress: (1) Spatial Variability, which uses geostatistic analyses such as the variogram [$\gamma(h)$] and covariant function [$Cv(h)$] to calibrate resistance factors (ϕ) with the consideration of 3-D (vector h) distances, expects to have results in 2009; (2) New Instrumentation to evaluate a new system of embedded instrumentation for wireless monitoring of pile foundation during construction is planned to instrument every pile. When enough data are collected, FDOT will re-calibrate the resistance factor to see if this will bring up the confidence level; thus raising the ϕ value for design. This work may be available in late 2009. (3) New Design Concept, a neural network application will someday use FDOT's new web-based geotechnical database and may be the future of its foundation design. The idea is to use the database as a training platform and geostatistic analysis to develop the neural network. When a new project is planned, the neural network program will search for sites with similar foundation and geologic conditions, which will include all design and construction data. Design options will be provided by the program. Each option will include recommendations such as field investigation, field tests and load tests amount requirements, and a corresponding foundation system design and its reliability. Resistance factors will be recalibrated when enough data are collected using this approach—after more learning data, e.g., design method, construction, and performance, etc., is available.

SUMMARY

FDOT started the implementation process in early 1995 by first sending most of its design engineers to some FHWA satellite LRFD training classes. The implementation started with documents and software conversion, geotechnical resistance factors calibrations, and in-house trainings. Preliminary implementation was in mid-1998 but wasn't completed until January

2002. The implementation of LRFD MSE wall design did not start until mid-2007. Fine tuning and recalibrating resistance factors for any items is an ongoing project, especially when a new method of design or construction is used.

CONCLUSION

Through a long implementation process, FDOT found that in order to have a successful LRFD implementation, it needed to realize and do the following:

- AASHTO resistance factors should be used with caution because they may not be proper for every design method, particularly design methods specific to one state;
- A geotechnical database should be developed by each state;
- Resistance factors should be calibrated based on a locally developed database and local geologic conditions as well as experience; and
- While the knowledge of AASHTO's covariance (COV) is important, uncertainty associated with mean properties (i.e., function of number of borings–lab tests), variance of in situ laboratory data both laterally and vertically, as well as spatial distance from the borings to pile–shaft are equally as important and should be incorporated into standard design. Lacking is a nationwide standard procedure for assessing the pile–shaft COV relative to boring COV (i.e., vertical and horizontal) as well as any reductions (i.e., covariance between nearby boring and pile–shaft) from which LRFD ϕ may be found.

North Carolina Department of Transportation's Experience with LRFD

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North Carolina Department of Transportation (NCDOT) has been wrestling with the implementation of AASHTO LRFD Bridge Design Specifications since the late 1990s. NCDOT's concerns include the following:

- North Carolina's local geology was not accounted for in the code;
- NCDOT's Vesic driven pile design method was not included in the AASHTO code; and
- The proposed resistance factors in the AASHTO specifications appeared to be too low.

As a consequence, NCDOT decided to award a research contract to North Carolina State University (NCSU) to develop local calibrations based on NCDOT's practice. This research was conducted by professors M. S. Rahman and M. A. Gabr, who were assisted by K. J. Kim of NCDOT, and R. Z. Sarica and M. S. Hossain of NCSU. Some of the recommendations of this research will be incorporated with the AASHTO LRFD code to become the department's LRFD practice.

NCDOT's ALLOWABLE STRESS DESIGN (ASD) PRACTICE

The North Carolina geology is broadly divided into three provinces: coastal plain, piedmont, and mountains. Each comprises approximately 35%, 50%, and 15% of the state's area, respectively. Figure 1 is a map showing North Carolina's physiographic provinces. The Coastal Plain consists of marine deposits of Quaternary, Tertiary and Cretaceous periods. The northeast outer coastal plain consists primarily of Quaternary deposits; the northern inner coastal plain is comprised primarily of Tertiary deposits; and both the southern inner and outer coastal plain areas consist of primarily Cretaceous deposits. The piedmont geology generally consists of residual soils on top of weathered rock, which transitions into rock. The mountains consist of shallow layers of saprolite or weathered rock on top of rock.

As expected, NCDOT's ASD driven pile design practice evolved around the local geology. The practice consists of using prestressed concrete piles in the outer coastal plain, and using steel H-piles, steel pipe piles, and prestressed concrete piles in the inner coastal plain. Steel H-piles, steel pipe piles, and spread footings are used in the piedmont and mountains. Drilled piers are used throughout the state. In the coastal plain, Vesic's static analysis method, supplemented by the Nordlund method, is used for driven pile design. In the piedmont and mountains, pile length determinations are primarily by inspection. In these regions, piles are typically driven to weathered rock or rock. Generally, NCDOT uses a factor of safety of 2 for pile design.

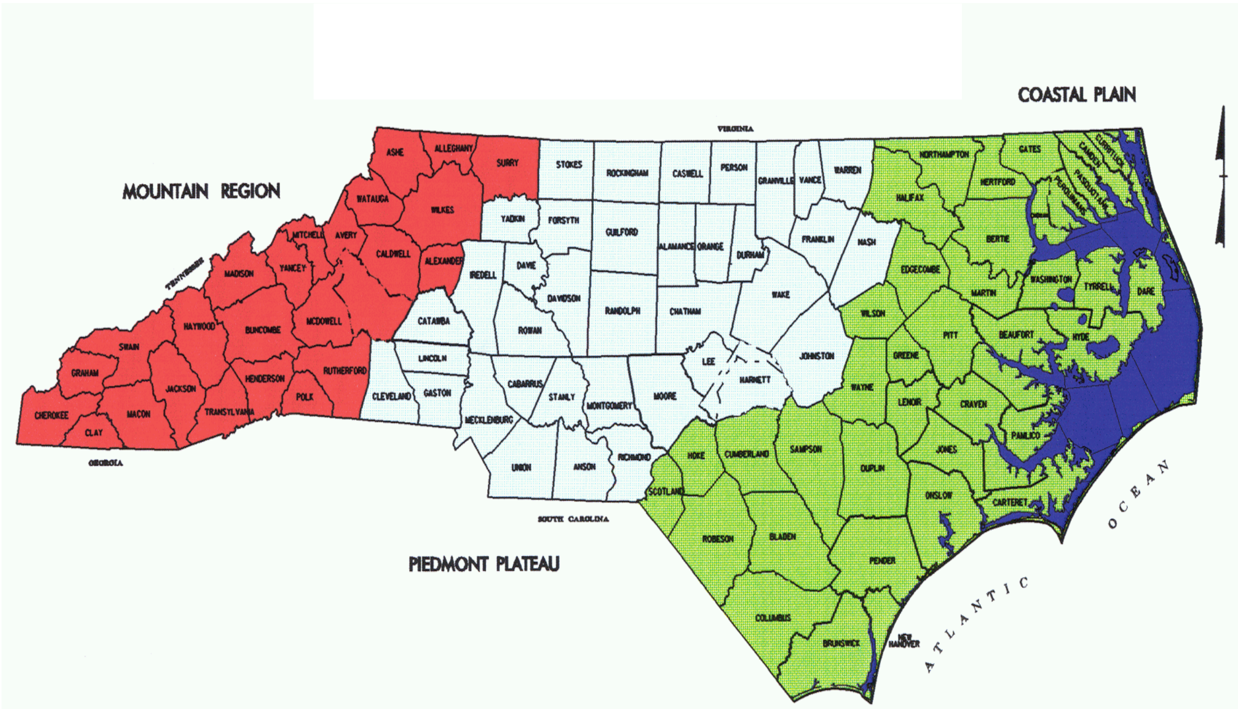

FIGURE 1 Geology map of North Carolina.

LOCAL CALIBRATION RESULTS

NCSU calibrated the resistance factors in the framework of reliability theory for the load and resistance factor design (LRFD) of driven pile's axial capacity using pile load test data available from the NCDOT highway construction projects. A total of 140 pile driving analyzer (PDA) data and 35 static load test data were compiled and grouped into different design categories based on four pile types and two geologic regions. Resistance statistics were evaluated for each design category in terms of bias factors. The bias factor is defined as the ratio of the measured pile capacity from a load test over the predicted pile capacity by a static analysis method. Bayesian updating was employed to improve the statistics of the resistance bias factors, which were derived from a limited number of pile load test data. Load statistics presented in the AASHTO LRFD bridge design specifications were used in the reliability analysis and the calibration of the resistance factors.

Reliability analysis of the current NCDOT practice of pile foundation design was performed to evaluate the level of safety and to select the target reliability indices. Resistance factor calibration was performed for the three methods of static pile capacity analysis commonly used in the NCDOT: the Vesic, the Nordlund, and the Meyerhof methods. Two types of first order reliability methods (mean value first order second moment method and advanced first order second moment method) were employed for the reliability analysis and the calibration of the resistance factors. The NCSU research recommended resistance factors for the three methods of static pile capacity analysis and for seven different design categories of pile type and geologic region. The resistance factors developed and recommended from this research are specific for the

TABLE 1 Data Available for Calibration (Coastal Plain)

Pile Type	PDA (EOD)	PDA (BOR)	PDA (Both)	Static Load Test
P/S Conc.	85	26	20	22
HP Steel	17	3	2	2
Steel Pipe	7	15	10	0
Conc. Cylinder	3	0	0	3

pile foundation design by the three static capacity analysis methods and for the distinct soil type of the geologic regions of North Carolina.

Table 1 is a summary of pile driving analyzer (PDA) and static load test data for different pile types in the coastal plain available for the calibration study. Sufficient PDA and static load test data were available for prestressed concrete piles. However, most of the data for steel H-piles were "end of drive" (EOD) PDA data. Therefore, in general, it does not represent the true ultimate resistance. NCDOT had insufficient data for steel pipe piles and concrete cylinder piles. Table 2 is a summary of recommended resistance factors for coastal plain steel H-piles for the three static methods. There is an insignificant difference between the resistance factors for the Vesic and Nordlund methods. However, the resistance factors for the Meyerhof method are approximately 20% lower than those for the Nordlund method. Table 3 is a summary of recommended resistance factors for coastal plain prestressed concrete piles. Again there is no significant difference between the Vesic and Nordlund resistance factors, but the resistance factors for the Meyerhof method are higher by a factor of 1.4–2.

COMPARISON BETWEEN AASHTO AND NCDOT RESISTANCE FACTORS

The AASHTO resistance factor for the Nordlund method is 0.45 for a reliability index of 2.3. The NCDOT Nordlund resistance factor for prestressed concrete piles is 0.45, for a reliability index of 2.5. Therefore, there is close agreement between the AASHTO and NCDOT resistance factors for the Nordlund method of prestressed concrete piles. On the other hand, the NCDOT Meyerhof resistance factors are considerably higher than those of AASHTO. The NCDOT resistance factors derived for all three static analysis methods of steel H-piles are significantly higher than the AASHTO resistance factors. This observation leads to the conclusion that there is a need to differentiate between high and low displacement piles. However, current AASHTO resistance factors do not differentiate between different pile types.

TABLE 2 Recommended ϕ for Coastal Plain H-Piles

Target Reliability Index	Vesic	Nordlund	Meyerhof
2.0	0.75	0.80	0.65
2.5	0.65	0.70	0.55

TABLE 3 Recommended ϕ for Coastal Plain Prestressed Concrete Piles

β	Vesic	Vesic	Nordlund	Nordlund	Meyerhof	Meyerhof
	N TOE ≤ 40	N TOE > 40	N TOE ≤ 40	N TOE > 40	N TOE ≤ 40	N TOE > 40
2.0	0.60	0.50	0.55	0.40	0.90	0.80
2.5	0.50	0.40	0.45	0.35	0.70	0.60

NCDOT's LRFD IMPLEMENTATION

Based on the results from the calibration research

- The Nordlund static analysis method will be adopted for NCDOT's LRFD practice;
- The AASHTO resistance factors for the Nordlund method will be adopted for all driven pile foundations in the piedmont and mountain areas and for prestressed concrete piles, steel pipe piles, and concrete cylinder piles in the coastal plain;
- The NCDOT resistance factor for the Nordlund method will be adopted for steel H-piles in the coastal plain as an interim;
- NCDOT will adopt the AASHTO resistance factors for drilled piers and shallow foundations.
- For micro-piles and continuous flight auger piles, NCDOT will use the resistance factors calibrated by fitting with factor of safety until AASHTO develops appropriate factors (see Table 4);
- For driven pile dynamic analysis, NCDOT will continue to use Goble Rausche Likin Wave Equation Pile Driving (GRL WEAP) to verify nominal resistance during driving and use resistance factors of 0.60 and 0.75 for GRL WEAP without and with PDA, respectively. These factors are based on fitting with factor of safety; and
- NCDOT plans to conduct at least 20 PDA beginning of restrike tests in the coastal plain in order to recalibrate the resistance factors for steel H-piles.

TABLE 4 Resistance Factor Fitting with Factor of Safety

γ_D	1.25	1.25	1.25	1.25	1.25	1.25	1.25
γ_L	1.75	1.75	1.75	1.75	1.75	1.75	1.75
Q_D/Q_L	1.5	1.5	1.5	1.5	1.5	1.5	1.5
FS	1.50	2.00	2.50	2.75	3.00	3.50	4.00
ϕ	0.97	0.73	0.58	0.53	0.48	0.41	0.36

PROPOSED DRIVEN PILE DESIGN PROCESS BETWEEN THE STRUCTURE DESIGN UNIT AND THE GEOTECHNICAL ENGINEERING UNIT

End Bents

1. Structures design unit (SDU) will provide nominal compressive structural resistance for common pile types. This will be in the form of nomographs as presented in Figure 2 for steel H-piles and steel pipe piles. Nominal compressive structural resistance will be the starting point for foundation analysis.
2. Geotechnical engineering unit (GEU) will adjust nominal compressive structural resistance to obtain maximum factored structural resistance based on driving conditions (good driving vs. severe driving) and drivability analysis (total resistance, stresses, and blow counts) and further reduce maximum factored structural resistance to account for factored downdrag load, if applicable.
3. GEU will determine whether pile driving analyzer (PDA) tests and CAPWAP will be used to develop pile driving criteria based on technical and economic considerations.
4. There are three options from which to choose to verify dynamic resistance of a pile: (a) Use PDA tests and CAPWAP as specified in the AASHTO specifications and incorporate the results in developing pile driving criteria using GRL WEAP. A dynamic resistance factor of 0.75 will be used in this case; (b) Use a limited number of PDA tests as needed to verify pile bearing and drivability and incorporate the results in developing pile driving criteria using GRL WEAP. A dynamic resistance factor of 0.6 will be used in this case; (c) Use GRL WEAP only to develop pile driving criteria. A dynamic resistance factor of 0.6 will be used in this case.

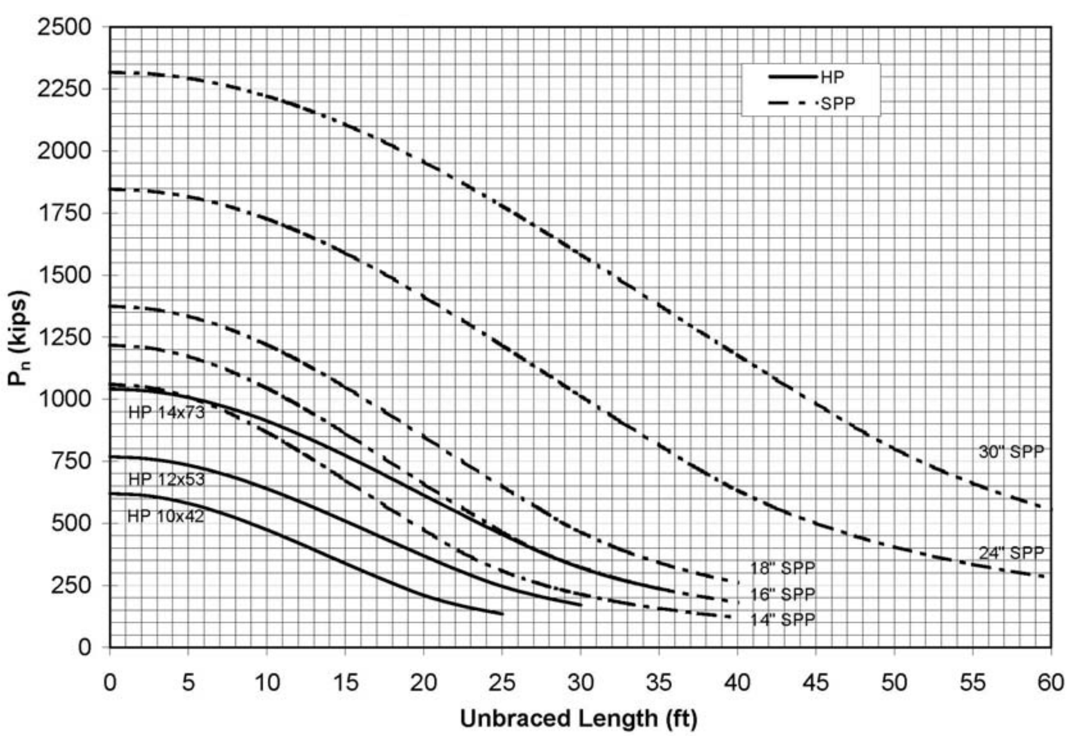

FIGURE 2 Nominal compressive resistance for steel piles.

5. GEU will determine required nominal static geotechnical resistance to support maximum factored structural resistance using different static analysis methods such as the Nordlund and the Tomlinson (with corresponding static resistance factors) and establish a depth vs. resistance graph to estimate required pile length.

6. Based on both the static and the dynamic analyses described above, GEU will provide preliminary foundation recommendations to SDU consisting of pile type and size and maximum factored resistance. At the same time, GEU will request factored loads on piles, total number of piles per bent, and corresponding pile spacing and pile installation configuration (single or double row and battered or plumb).

7. Upon receiving the requested information from SDU, GEU will do the following: (a) equate factored loads to factored resistances, (b) estimate pile lengths for nominal resistances, which are equal to factored resistances divided by static resistance factors, and (c) determine required driving resistances, which are equal to factored resistance divided by dynamic resistance factors.

8. GEU will provide final foundation design recommendations consisting of pile type and size, factored resistances, estimated pile lengths, number of piles per bent, required driving resistances to be verified during pile driving, and appropriate notes for inclusion in the contract plans.

Interior Bents

The interior bent design process is very similar to the end bent design process described above. Steps 1 through 4 can be repeated here. In Step 5, loss of geotechnical resistance due to scour needs to be taken into account in determining nominal static geotechnical resistance.

1. GEU will compute a preliminary point of fixity (POF) using assumed loading conditions such as axial loads in the range of 300–400 kips per pile, and lateral loads in the range of 2 to 3 kips per pile. The analysis will be conducted using LPILE program.

2. Based on both the static and the dynamic analyses described above, GEU will provide preliminary foundation recommendations to SDU consisting of pile type and size, preliminary POF, and maximum factored resistance. At the same time, GEU will request factored loads on piles, total number of piles per bent, and corresponding pile spacing and pile installation configuration such as single or double row and battered or plumb from SDU.

3. GEU will compute a new POF using factored structural loads received from SDU; provide SDU with a second iteration of POF, pile type and size, and maximum factored resistance; and request the same information as in Step 7.

4. Final POF will be determined when the new POF is less than three feet higher or two feet lower than the previously computed POF. Otherwise, at least one more iteration will be required.

5. Once a final POF is set with the corresponding structural loads, perform the same tasks as stated in Step 7 of the end bent design process. If there is any loss of geotechnical resistance due to scour, it must be included in determining required driving resistance.

6. GEU will provide final foundation design recommendations consisting of pile type and size, factored resistances, POF, estimated pile lengths, number of piles per bent, required driving resistances to be verified during pile driving, and appropriate notes for inclusion in the contract plans.

COMPARATIVE CASE STUDIES

End Bents

The following three bridge projects were selected for study: B-3467 in Halifax County, B-3692 in Robeson County, and B-3871 in Martin County. These three bridges are in different geologic formations. B-3467 is in a Cretaceous formation, B-3692 is in a region where a Tertiary formation overlay a Cretaceous formation, and B-3871 is primarily in a Tertiary formation. Steel H-piles were used on all three bridges. The results are summarized in Table 5. It should be noted that the ASD pile lengths were based on the current NCDOT practice of using an allowable load of 50 tons per pile for H-pile 12x53 steel piles. LRFD pile lengths were based on the AASHTO LRFD specifications that permit use of factored structural loads of up to the factored compressive structural resistance of the pile. The comparison shows that adopting the AASHTO LRFD code may result in reduced pile lengths. Actual cost savings are unknown because pile driving has not been completed.

Interior Bents

Project B-3692 in Robeson County was selected for interior bent comparisons. The results are summarized in Table 6. It should be noted again that the ASD pile lengths were based on the current NCDOT practice of using an allowable load of 60 ton per pile for steel H-pile 14x73 piles, while LRFD pile lengths were based on the AASHTO specifications that permit the use of factored structural loads of up to the factored compressive structural resistance of the pile. As for end bents, adopting the AASHTO LRFD code may result in reduced pile lengths for interior bents.

TABLE 5 Design Comparison (End Bent)

Project	ASD		LRFD with PDA		LRFD with GRLWEAP (No PDA)	
TIP No. County	Total Pile Length (ft)	Number of Piles	Total Pile Length (ft)	Number of Piles	Total Pile Length (ft)	Number of Piles
B-3467 Halifax	840	8	840	4*	725	5
B-3692 Robeson	960	8	825	5	840	6
B-3871 Martin	1,395	9	1,100	5	1,225	7

*Four piles were needed in this example, which required a 20% reduction in resistance factor per AASHTO spec. Current NCDOT LRFD practice requires a minimum of 5 piles per bent.

NOTE: AASHTO recommended dynamic resistance factors (0.40 for GRL WEAP only and 0.65 for GRL WEAP with PDA) were used at the time this table was prepared. Current NCDOT LRFD practice uses the resistance factors of 0.60 for GRL WEAP only and 0.75 for GRL WEAP with PDA based on fitting with factor of safety.

TABLE 6 Estimated Pile Lengths (Interior Bent No. 1 Total)

Project	ASD (FS = 2)	LRFD No PDA (ϕ = 0.7)	Pile Type
B-3692	400 ft	325 ft	HP 14 x 73

Future Calibration Plans

NCDOT is planning to conduct a minimum of 20 PDA tests on steel H-piles in the Coastal Plain. Both end-of-drive data and beginning-of-restrike data will be collected. The restrike will be performed a minimum period of 96 hours after completion of the initial drive. Once data are obtained, a calibration process will be undertaken to provide resistance factors consistent with NCDOT's practice and based upon ultimate resistance. After this work is accomplished, additional calibration work will be conducted to develop dynamic resistance factors for GRL WEAP and PDA. This will be followed by calibrations for the other foundation types used by NCDOT.

SUMMARY AND CONCLUSIONS

- NCDOT had sufficient pile load test data to calibrate the resistance factors for prestressed concrete piles.
- Most of the test data available for steel H-piles were from the PDA end of drive, which does not represent the pile's ultimate resistance.
- There was a good match between nominal static resistance from the Nordlund method and PDA beginning of restrike data for coastal plain prestressed concrete piles
- The mean bias factor of coastal plain concrete piles for the Nordlund method and static load test results was close to unity. However, the coefficient of variation was high. On the other hand, the Vesic method predicted higher capacity than static load tests, but the coefficient of variation was reasonable.
- The AASHTO resistance factor for the Nordlund method is very close to the NCDOT resistance factor for prestressed concrete piles in coastal plain.
- Different resistance factors are required for low and high displacement piles.
- There is a significant difference between the resistance factors obtained from the NCDOT research and the AASHTO resistance factors for steel H-piles.
- Based on the case studies completed to date, it appears that by adopting the AASHTO LRFD specifications estimated total pile length will be generally less than that obtained by the current ASD practice.

Pennsylvania Department of Transportation's Experience with LRFD

BEVERLY MILLER
Pennsylvania Department of Transportation

PEnnsylvania Department of Transportation (PennDOT) presented its experience with LRFD implementation using a case history concerning abutment design followed by a description and status of PennDOT's implementation of the LRFD specification. The case history, presented from a structural point of view, demonstrates the potential practical effects and potential adjustments that may be required to state specifications as a result of implementation of the LRFD specification.

CASE HISTORY

The case history concerned the unexpected effects of the AASHTO LRFD braking force on the width of spread footings. Note that henceforth when "braking force" is mentioned, it refers to the AASHTO braking force. A complaint was received from a user of the department's abutment LRFD computer program that an unusually large footing size was designed. For footings designed in accordance with the LRFD specifications, complaints of large footing sizes are not uncommon. However, the abutment design was examined to determine the rationale for the increase in footing width. The increase was determined to be a result of the LRFD braking force.

While it was understood that the LRFD specifications had increased the braking force on short spans and decreased the braking force on longer spans, the effects of braking force on the width of the footing were not anticipated. As a result of this experience, PennDOT now includes verbiage in its "Design Manual Part 4, Structures" to warn of the increase in braking effects for short spans; i.e., that braking forces may be greater than previously experienced under load factor forces for structures less than 500.′ In addition, for long span structures, "braking force factors" are specified to bring the practice more in line with the braking forces experienced using load factor design.

A parametric study was conducted including five loading conditions: original dead load and live load, dead load and live load reduced by one-half, doubling dead load and live load, one-half of dead load and full live load, live load reduced by one-half original dead load. The braking force was varied, as well as the toe, stem, and heel widths; and the toe, heel, and stem reinforcement. Abutments founded on spread footings on soil and spread footings on rock were examined. Note that PennDOT's abutment and retaining wall program contains input fields to limit the settlement and bearing capacity, and that the program iterates on the design to meet the requirements. For each LRFD design data point, the program iterates 500 to 1,000 trial designs to arrive at optimum footing width.

For the case of an abutment placed on a spread footing on soil, despite the variability of braking force, using the LRFD specification, the footing width had increased uniformly, depending on the combination of dead load and live load selected, 6′ to 10′ beyond the design performed using allowable stress design (Figures 1 and 2). Bearing was controlling the design of the foundation, and PennDOT's abutment and retaining wall program would respond to the increased bearing pressure–bearing requirements by reconfiguring the footing width to meet the specified–input bearing capacity.

For the case of an abutment on a spread footing founded on rock, the LRFD braking force and combinations of live load and dead load were varied similar to the spread footing on soil. However, for the spread footing on rock, the footing width using the LRFD specifications and braking force was almost identical to the footing width that resulted from allowable stress

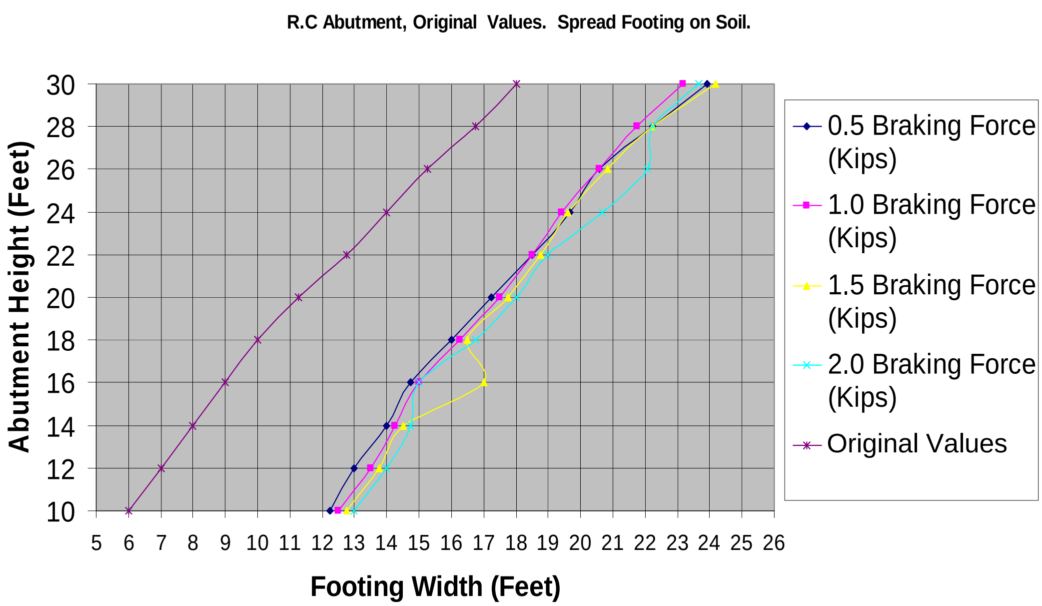

FIGURE 1 Spread footing on soil and braking force—original loads.

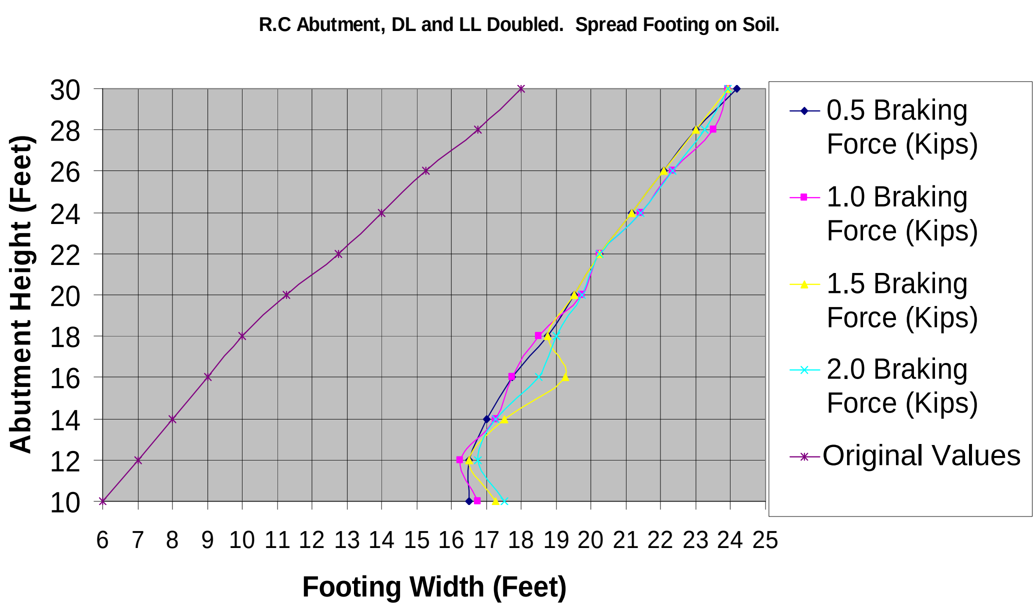

FIGURE 2 Spread footing on soil and braking force—dead load and live load doubled.

design. That is, the same loading configurations were considered as for the spread footing on soil for the spread footing on rock, but the result was a negligible increase in the footing width for the abutment on spread footing on rock (Figure 3). The rationale for the disproportional increase in the bearing pressure associated with the LRFD breaking force and the resulting increase in footing width necessary to meet the specified–input bearing capacity have yet to be determined.

LRFD IMPLEMENTATION

PennDOT's strategy for implementation of the LRFD specification was to examine the proposed specification changes and thoroughly understand the implications of the specification by performing parametric studies and sample designs or to modify the LRFD specification for PennDOT practice to be in agreement with former allowable stress practices. To facilitate the use of the LRFD specifications, LRFD computer programs were developed in preparation for acceptance of LRFD specification. Training sessions were held to inform designers of the LRFD specification changes and to train design personnel in the use PennDOT's LRFD programs, including programs for abutments and retaining walls and piers.

The timeline for PennDOT's implementation of LRFD began in 1993 with the formation of the task force work to examine the LRFD specifications and concluded in 2001 with the adoption of the 1996 LRFD AASHTO specification for both superstructures and substructures. Prior to 2001, the LRFD specifications were implemented for superstructures while using LFD design for substructures. Currently, LRFD has been completely implemented for bridges and for retaining walls as well as for piles, caisson, micro-piles, spread footings, and MSE walls.

The implementation of the LRFD specification for PennDOT presented some challenges. One of the challenges with LRFD design was the identification of the rationale for practical differences from the LFD specification, such as the cause of increased footing size, as indicated above. While some of the criteria that could contribute to the increase in footing size were

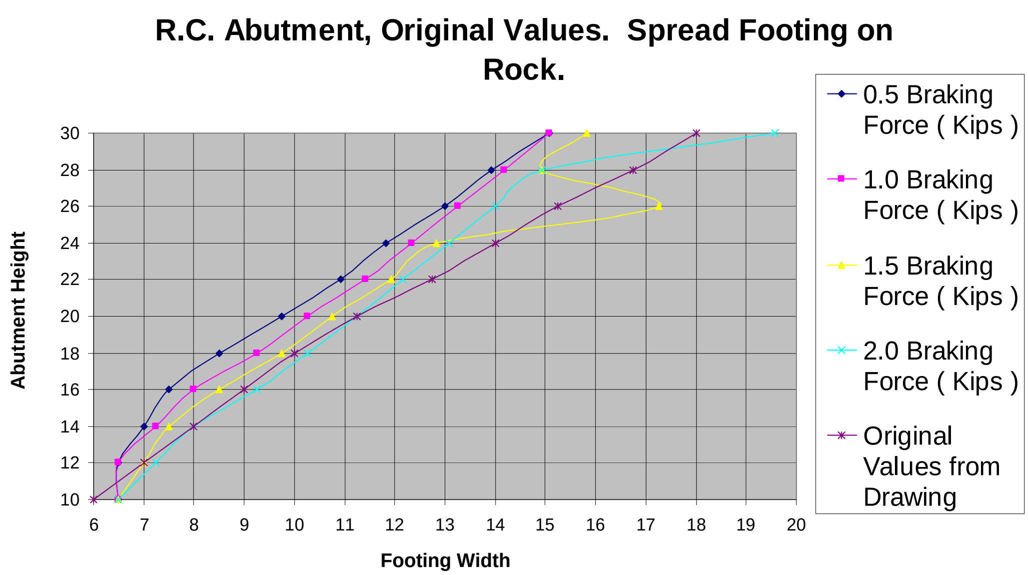

FIGURE 3 Spread footing on rock and braking force—original dead load and live load.

established, the investigation remains ongoing. Another challenge that PennDOT experienced with the LRFD specification was related to various items regarding the interpretation of the specification. For example, in the calculation of inclination factors, factored loads were initially incorrectly used. The specifications state that unfactored loads should be used to calculate the inclination factors. The use of the factored vertical and horizontal loads in the calculation of the inclination factors resulted in a decreased bearing capacity and, in turn, a larger foundation to meet the decreased bearing capacity. The implementation of LRFD specification for proprietary structures and introducing construction personnel to the concepts of LRFD design continue to be areas of concern.

LRFD has been implemented for PennDOT through the use of parametric studies, specification comparison, and advance planning and training. Implementation of the specification presents an ongoing learning curve with the opportunity to adjust practices.

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