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IMPROVEMENT OF THE GEOTECHNICAL AXIAL DESIGN METHODOLOGY FOR COLORADO'S DRILLED SHAFTS SOCKETED IN WEAK ROCKS

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DOT
DEPARTMENT OF TRANSPORTATION
RESEARCH

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COLORADO DEPARTMENT OF TRANSPORTATION
RESEARCH BRANCH

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16. Abstract: Drilled shaft foundations embedded in weak rock formations (e.g., Denver blue claystone and sandstone) support a significant portion of bridges in Colorado. Since the 1960s, empirical methods and "rules of thumb" have been used to design drilled shafts in Colorado that entirely deviate from the AASHTO design methods. The margin of safety and expected shaft settlement are unknown in these methods, however, both are needed for the implementation of the new and more accurate AASHTO Load and Resistance Factor Design (LRFD) method in CDOT design guidelines. Load tests on drilled shafts provide the most accurate design information and research data for improvement of the design methods for drilled shafts.

As a part of the construction requirements for the T-REX and I-25/Broadway projects along I-25 in Denver, Colorado, four Osterberg (O-Cell) load tests on drilled shafts were performed in 2002. The bedrock at the load test sites represents the range of typical claystone and sandstone (soft to very hard) encountered in the Denver metro area. To maximize the benefits of this work, the O-Cell load test results and information on the construction and materials of the test shafts were documented, and an extensive program of simple geotechnical tests was performed on the weak rock at the load test sites. This includes standard penetration tests, strength tests, and pressuremeter tests. The analysis of the all test data and information and experience gained in this study were employed to provide: 1) best correlation equations between results of various simple geotechnical tests, 2) best-fit design equations to predict the shaft ultimate unit base and side resistance values, and the load-settlement curve as a function of the results of simple geotechnical tests, and 3) assessment of the CDOT and AASHTO design methods.

Implementation: Three implementation products are recommended in this study. First, preliminary design methods as required in the LRFD method for drilled shafts with conditions close to those encountered at the four load test sites. All the qualifications and limitations for using these design methods are presented (e.g., construction procedure). Incorporation of these methods will lead to savings in CDOT future construction projects. Second, preliminary recommendations to improve and standardize geotechnical subsurface investigation for drilled shafts in Colorado. Third, a detailed strategic plan for CDOT to identify the most appropriate design methods per LRFD for Colorado's drilled shafts embedded in various weak rock formations. This plan of six tasks requires: (1) acquiring the testing and site specific resistance parameters of the LRFD, and (2) the assembly and analysis of a database that contains the results of old and new load tests, and results of simple geotechnical tests at the load test sites. Detailed guidelines for planning and performing new load tests on drilled test shafts in CDOT future construction projects and data analysis are provided.

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CONVERSION TABLE

U. S. Customary System to SI to U. S. Customary System

(multipliers are approximate)

Multiply (symbol)	by	To Get (symbol)	Multiply	by	To Get
LENGTH					
Inches (in)	25.4	millimeters (mm)	mm	0.039	in
Feet (ft)	0.305	meters (m)	m	3.28	ft
yards (yd)	10.914	meters (m)	m	1.09	yd
miles (mi)	1.61	kilometers (km)	m	0.621	mi
AREA					
square inches (in^2)	645.2	square millimeters (mm^2)	mm^2	0.0016	in^2
square feet (ft^2)	0.093	square meters (m^2)	m^2	10.764	ft^2
square yards (yd^2)	0.836	square meters (m^2)	m^2	1.195	yd^2
acres (ac)	0.405	hectares (ha)	ha	2.47	ac
square miles (mi^2)	2.59	square kilometers (km^2)	km^2	0.386	mi^2
VOLUME					
fluid ounces (fl oz)	29.57	milliliters (ml)	ml	0.034	fl oz
gallons (gal)	3.785	liters (l)	l	0.264	gal
cubic feet (ft^3)	0.028	cubic meters (m^3)	m^3	35.71	ft^3
cubic yards (yd^3)	0.765	cubic meters (m^3)	m^3	1.307	yd^3
MASS					
ounces (oz)	28.35	grams (g)	g	0.035	oz
pounds (lb)	0.454	kilograms (kg)	kg	2.202	lb
short tons (T)	0.907	megagrams (Mg)	Mg	1.103	T
TEMPERATURE (EXACT)					
Farenheit (°F)	5(F-32)/9 (F-32)/1.8	Celcius (° C)	° C	1.8C+32	° F
ILLUMINATION					
foot candles (fc)	10.76	lux (lx)	lx	0.0929	fc
foot-Lamberts (fl)	3.426	candela/m (cd/m)	cd/m	0.2919	fl
FORCE AND PRESSURE OR STRESS					
poundforce (lbf)	4.45	newtons (N)	N	.225	lbf
poundforce (psi)	6.89	kilopascals (kPa)	kPa	.0145	psi

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In Memory of Michael O'Neill, a Co-Author of this Report

The CDOT Research Branch would like to acknowledge the death of Michael W. O'Neill, Ph.D., P.E., Cullen Distinguished Professor of Civil and Environmental Engineering at the University of Houston, Texas. Mr. O'Neill passed away on August 2, 2003, shortly before the publication of this report. We would like to dedicate this report to his memory. His friendship and his dedication to this CDOT research project will be missed by the staff. He was a true partner in research.

ACKNOWLEDGEMENTS

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EXECUTIVE SUMMARY

Drilled shaft foundations embedded in weak rock formations (e.g., Denver blue claystone and sandstone) support a significant portion of bridges in Colorado. Since January 1, 2000, it has been the policy of CDOT to incorporate the new and more accurate AASHTO Load and Resistance Factor Design (LRFD) method for the design of its structures (including drilled shafts). On the resistance side, the LRFD method requires information on the shaft ultimate capacity load, shaft head load-settlement curve, and a resistance factor (ϕ) to ensure adequate level of margin of safety. Since the 1960s, empirical methods and "rules of thumb" have been used to design drilled shafts in Colorado that are based on the blow counts of the simple Standard Penetration Test (SPT). The margin of safety (or ϕ) and expected shaft settlement are unknown in these methods, which entirely deviate from the AASHTO design methods. AASHTO offers simple design methods with ϕ that are based on the results of simple geotechnical tests (strength tests for rocks, not SPT as in CDOT method). However, these methods are developed for conditions different from those encountered in Colorado (e.g., not for the weak rocks often encountered in Colorado). The most accurate design method for drilled shafts is to conduct a load test on test shafts constructed as planned in the construction project. The load tests are expensive and therefore are only considered for large projects. However, the very accurate information derived from the load test could be used: 1) to design production shafts with more confidence, resulting in large cost savings to the project, and 2) as research data to improve accuracy of simple design methods for drilled shafts, as presented next.

CDOT strategy is to correlate the measured results of a large number of load tests on drilled shafts embedded in various types of Colorado's weak rocks with the predictions of simple design methods that use data of simple geotechnical tests in order to identify the most appropriate: (1) geotechnical design methods to predict the ultimate axial resistance and settlements of Colorado's drilled shafts; and (2) resistance factors needed in the LRFD method.

As part of the construction requirements for the T-REX and I-25/Broadway projects along I-25 in Denver, Colorado, four Osterberg (O-Cell) load tests on drilled shafts were performed in 2002. The bedrock at the load test sites represents the range of typical claystone and sandstone

(soft to very hard) encountered in Denver. To maximize the benefits of this work, the O-Cell load test results, information on the construction and materials of the test shafts, and geology of bedrock were documented, and an extensive program of simple and diversified geotechnical tests was performed on the weak rock at the load test sites. This includes the SPT, strength tests, and pressuremeter tests. The analysis of all test data and information and experience gained in this study were employed to provide: 1) best correlation equations between results of various simple geotechnical tests, 2) best-fit design equations to predict the shaft ultimate unit base and side resistance values, and the load-settlement curve as a function of the results of simple geotechnical tests, and 3) assessment of the CDOT and AASHTO design methods.

The Implementation Products:

Product 1: Preliminary design methods as required in the LRFD for drilled shafts with conditions close to those encountered at the four load test sites. All the qualifications and limitations for using these design methods are presented (e.g., construction procedure). CDOT current design method is very conservative when drilled pier sockets are constructed in very hard claystone/sandstone rocks, leading to costly construction of Colorado high-capacity drilled shafts embedded in these rocks. The use of new design methods will lead to significant savings in CDOT future construction projects and will offset the additional construction and testing costs needed for implementation.

Product 2: Preliminary recommendations for improvement and standardization of the geotechnical subsurface investigation procedures for drilled shafts embedded in weak rock formations (definition for adequate subsurface geotechnical investigation, SPT, sampling procedure, UC test, and PM test).

Product 3: A plan to fulfill the CDOT strategic objectives for improvement of the geotechnical design methodology for drilled shafts. This plan is described in very specific details in Chapter 7 and has three parts:

Part I. A new research study to improve and standardize the geotechnical subsurface investigation procedures for drilled shafts in Colorado and acquire the testing and site specific resistance parameters of the LRFD.

Part II. The assembly of a database that contains the results of old and new load tests on typical Colorado drilled shafts, information on the materials, layout, construction of test shafts, and geological and geotechnical descriptions of the foundation bedrock at the load test sites (i.e., from results of SPT, UC, and PM tests). Three tasks are needed in this phase literature review, acquisition of new geotechnical data at sites of existing load tests, acquisition of new geotechnical data at sites of new axial load tests. At minimum, CDOT needs to perform a total of 24 loads tests on drilled shafts embedded in four types of Colorado's weak rocks. Complete guidelines for planning and performing new load tests on drilled test shafts in CDOT future construction projects described.

Part III. Analysis of all the information collected in the previous two parts to fulfill CDOT strategic objectives and publication and promotion of the findings. Complete details of the data analysis are described.

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1. INTRODUCTION AND OVERVIEW OF THE REPORT

1.1 Introduction

Drilled shaft (pier or caisson) foundations are extensively used in Colorado to reduce foundation movements in lightly loaded structures founded on expansive soil and bedrock, and provide high capacity for the most heavily loaded bridge structures and tallest buildings. In Colorado, Wyoming, and South Dakota, most drilled shafts are underlain by Late Cretaceous age sedimentary rock formations that in many locations have engineering properties of "weak rock" (Turner et al., 1993). Drilled shafts derive support by embedment in these weak rocks, typically found at relatively shallow depth. The contribution of overburden to the drilled shaft axial capacity is often ignored in the Colorado and AASHTO design methods. Two prevalent geologic formations for the weak rocks are the Pierre and Denver Formations (Turner et al., 1993). These formations consist of weakly cemented claystone, siltstone, sandstone, and interbedded sandstone/claystone, with composition consisting of varying amounts of fine-grained to very coarse-grained sediments. According to Jubenville and Hepworth (1981), the range of unconfined compressive strength for the Denver Formation is from 6 ksf (very stiff clay soils) to more than 60 ksf (very low strength rock), and that shear strengths are higher in the "blue" claystone which underlies downtown Denver. Drilled shafts in these weak rocks are attractive, when compared to driven piles, since the boreholes in these materials are relatively stable and the geomaterials are not usually difficult to excavate.

1.1.1 Geotechnical Axial Design Methods for Drilled Shafts

The geotechnical design of drilled shafts requires the consideration of both the Serviceability and Strength Limits. The Serviceability Limit ensures that the function of the structure under normal service conditions performs satisfactorily. This limit requires the determination of the top load-settlement curve in order to ensure that the settlement (w_{all}) developed under the design allowable load (Q_{all}) is less than the tolerable settlement. The Strength Limit ensures that the design provides adequate margin of safety against geotechnical failure. This limit requires determination of the ultimate unit base resistance (q_{max}) of the rock layer beneath the shaft, ultimate unit side resistance of the rock around the shaft (f_{max}), the average load factor and resistance factor (ϕ) in the load and resistance factor design (LRFD) method, and the factor of safely (FS) in the allowable stress design (ASD) method. The shaft ultimate resistance load (Q_{max}) is estimated as $A_b q_{max} + A_s f_{max}$, where A_b is the base area of the shaft in the rock socket and A_s is the side area of the shaft in the bedrock socket. The allowable base resistance (q_{all}), side resistance (f_{all}), and design load (Q_{all}) are defined by the Strength Limit as q_{max}/FS, f_{max}/FS, and Q_{max}/FS. As opposed to the ASD, where all uncertainty is embedded within a factor of safety (FS), the LRFD approach applies separate factors (load factor and resistance factor) to account for uncertainty in load and resistance. This will provide a more reliable approach for the design of highway structures and achieves a more consistent level of safety in both structure and substructure design. Additionally, LRFD allows for developments of resistance factors that depend on the level of confidence (errors and variability) of the measured geotechnical properties in the subsurface geotechnical investigation and the accuracy of the design method (see Chapter 7).

The most accurate and rational design method is to conduct a load test on shafts constructed in a manner and of dimensions and material identical to those planned for the construction shafts. The load test can be designed to obtain the load transfer f-w curve (where f is the unit side resistance and w is the shaft displacement relative to the surrounding rock) and f_{max} for rock layers within the test socket; and q-w curve (where q is the unit base resistance), and q_{max} for the rock layers beneath the shaft. The load test data can then be used: 1) to design the production shafts with more confidence (smaller FS and higher ϕ) that could result in potentially large cost savings to the project, and 2) as research data to improve the design methodology in all future applications. Because of the relatively high costs associated with performing load tests, it is difficult to perform these tests on every project.

AASHTO standard (2002) and AASHTO LRFD (1998), and the Canadian Foundation Engineering Manual (CFEM, 1992) offer simple geotechnical design methods for drilled shafts embedded in either soils (cohesive or granular) or rocks. These semi-empirical design methods were developed using correlations between the results of field loading tests in other geographical areas (with specific geology, construction, and materials) and the following geotechnical engineering properties:

- Unconfined compressive strength of intact cores (q_{ui}). The unconfined strength (UC) test is employed to determine both the UC strength (q_{ui}) and Young's modulus (E_i) of intact rock cores.
- Standard Penetration Test (SPT) blow counts per foot (bpf). These data and the vertical effective stress are considered only in cohesionless soil and cohesionless intermediate materials (defined later).
- RQD (Rock Quality Designation). Due to the presence of discontinuities (soft seams and/ or joints) in the rock mass, intact core strength (q_{ui}) and stiffness (E_i) as measured in the UC test could be larger than the rock mass strength and rock mass stiffness (E_m). Information on the RQD and conditions and structures of joints are utilized to develop reduction factors for strength and stiffness values obtained from laboratory testing on intact cores.

The in-situ properties of larger volumes of rock mass that cannot be tested in the laboratory can be measured with the Menard pressuremeter. This is advisable when good quality samples cannot be obtained for testing in the laboratory (very soft or fractured rock). Compared to the measured laboratory values, the use of in-situ pressuremeter (PM) tests will reduce to a large extent the influence of sample disturbance encountered with lab tests, account for the confining pressure not accounted for in the UC test, and could account for the presence of discontinuities in the rock mass. From the pressuremeter tests on weak rocks, the stress-strain curve and E_m can be measured directly (O'Neill et al., 1996) and the unconfined compressive strength can be estimated indirectly (as will be described in this study). However, the PM test does not stress a large enough volume of geomaterial to be truly representative of the entire rock mass as in the load test.

Differences in the selection of factor of safety or resistance factor and in the definitions of ultimate base resistance, ultimate side resistance, and tolerable settlements among the design methods lead to different output design results from these methods, as will be discussed in this report. The selected FS or ϕ depends on the accuracy and reliability of the predicted resistance values from these design methods compared to the true values as measured in the load tests. The FS for the simple design methods is often larger than 2, leading to more costly design when compared to design based on load tests. Ultimate resistance can be defined to correspond to full mobilization of the plastic resistance (true resistance), to initiation of fracturing in the rock, or to a specified displacement. Settlement analysis of the drilled shafts is often ignored in many design methods. Settlements in these design methods are controlled by selecting a large factor of safety or very conservative definition of ultimate resistance (not to correspond to large displacement) so that the allowable design loads calculated from the strength limit design are smaller than the design loads that correspond to the tolerable settlement. In rocks, most of the resistance to working loads is achieved by means of side resistance, as opposed to base resistance, and, in this case, the settlements within the socket will be small (AASHTO, 1998). If the rock socket capacity is derived from the base resistance, the settlements in the range of working load become important and could be obtained from an analytical solution (Eq. C10.8.3.5-1 and 2 in AASHTO 1998) that employs modulus of elasticity of the in-situ rock (E_m).

None of the design methods described in AASHTO and CFEM provides procedures very specific to weak rocks or considers all the factors that actually control side and base resistance. O'Neill et al. (1996) and FHWA (1999) presented design methods for a new category of geomaterials at the boundary between soil and rock called "Intermediate Geomaterials, IGM" (Chapter 2). This method considers the following additional parameters:

1. Geomaterial information in addition to q_{ui} and RQD, including: cohesion and internal friction characteristics of the weak rock and interface; size and frequency of discontinuities and soil seams, if any, in the geomaterial; mass elastic modulus of the rock mass (E_m), the ratio (E_m/q_{ui}), and Poisson's ratio of the weak rock.
2. Materials information of the drilled shaft: composite stiffness modulus of the shaft (concrete and steel, E_c), slump of the concrete at time of placement, the unconfined compressive strength of the concrete f_c'. In this investigation, it is assumed that $f_c' > q_{ui}$.
3. Geometrical information of the drilled shaft: socket length (L), socket diameter (D), and ratio L/D.
4. Construction information on the drilled shafts: interface roughness; cleanness of the interface and bottom of the shaft hole; and time/method for excavation and placement of concrete, and if water entered the socket borehole during drilling.

1.1.2 CDOT Geotechnical Design Method, Needs, and Strategic Objectives

Since the 1960s, empirical methods and "rules of thumb" have been used to design drilled shafts in the Denver Metropolitan/Colorado Front Range area. This empirical formula is geared toward the allowable stress design method (ASD) because it predicts only the allowable geomaterial resistance from the results of the standard penetration test (N-value). In the Colorado SPT-Based (CSB) design method, the allowable end bearing pressure in kips per square foot (ksf) is taken as N/2, and the allowable side shear as ten percent of the allowable unit end bearing pressure. The main deficiencies of this method are:

- There are no theoretical bases to justify using side resistance values equal to 10% of the base resistance.
- Because the N-value-based design methods are rather crude, most practitioners limit the allowable end bearing to about 50 ksf and allowable side resistance to 5 ksf. For hard claystone and sandstone bedrock, this limitation leads to overconservative design as will be demonstrated in this study.
- AASHTO, FHWA, and CFEM recommend the use of vertical effective stress in addition to N-values in design methods for weak cohesionless rock formations (50<N<100). For cohesive intermediate geomaterials (IGM) and rocks (including cohesionless GM with SPT larger 100), AASHTO, FHWA, and CFEM recommend the use of strength data. However, the same CSB design method is uniformly applied for both the cohesive and cohesionless weak rocks and rocks that is based on SPT-N. O'Neill et al. (1996) indicated that any SPT-based design method for cohesive IGM is unreliable and should be used as a last resort.
- The margin of safety (factor of safety or resistance factor) and expected settlement embedded in the CSB design method are unknown. This restricts the ability of the design engineer to exercise judgment and account for risk by applying the proper margin of safety.
- This method deviates from FHWA/AASHTO design approach and mainstream practices. There is no resistance factor or factor of safety for this design method in AASHTO.

Turner et al. (1993) and Attwooll (2002) concluded based on the results of load tests that the CSB design method can on occasion result in unconservative factors of safety in side shear and suggested modifications to the CSB design method (presented in Chapter 2). Turner et al. (1993) and Jubenville and Hepworth (1981) recommended using the CSB design method with caution and understanding. However, the CSB design method has endured simply because it has worked "magically" well. Jubenville and Hepworth (1981) reported "The writers are not aware of instances where excess pier loading has resulted in unacceptable settlement of properly constructed piers." All failures that Attwooll (Senior Geotechnical Engineer) is aware of were caused by construction defects. He asked a number of senior geotechnical practitioners in the Denver area, and none was aware of any instance where the Denver design method (CSB) did not work in over 40 years. However, the lack of reported failures could be an indication of CSB procedures being excessively conservative. Many Colorado geotechnical engineers have recommended investigating the CSB method by performing load tests or instrumentation of drilled piers beneath existing structures.

Since January 1, 2000, it has been the policy of CDOT to incorporate the new and more accurate AASHTO Load and Resistance Factor Design (LRFD) method for the design of all structures (including drilled shafts). The AASHTO LRFD Bridge Design Specification manual (1998) provides resistance factors for normal soils or rocks, not the intermediate weak rocks often encountered in Colorado. Bedrock is treated as a special condition in AASHTO LRFD (1998). The AASHTO design methods are based on load tests performed in other geographical areas (with geologic, material, construction, and geometrical details different from those encountered in Colorado) and for design methods not often utilized by Colorado geotechnical engineers.

To address all issues listed above, CDOT strategic objectives for drilled shafts are to:

- Identify the most appropriate (feasible, accurate, and cost-effective) geotechnical design methods to predict the ultimate axial resistance and settlements of Colorado drilled shafts socketed in the weak rock formation that are based on simple and routine geotechnical tests (e.g., SPT, UC test, and the Menard pressuremeter tests).
- Identify, for the recommended design methods, the most appropriate resistance factors (ϕ) needed for the LRFD method.

1.2 New Load Tests in Denver

The $1.6 billion T-REX (Transportation Expansion) project involves design and construction of significant roadway improvements and a new light rail transit (LRT) system for a 19-mile long corridor along interstates I-25 and I-225 in south and southeastern metropolitan Denver. Dozens of reconstructed and new bridges are included with hundreds of drilled shaft foundations. The I-25/Broadway project, close to the north edge of the T-Rex project, involves replacement of the I-25 Viaduct over Broadway and railroad tracks. As part of the construction requirements for T-REX and I-25/Broadway projects, four Osterberg ("O-Cell") load tests on drilled shafts were performed in early January, 2002. A location map for the test shafts is shown in Figure 1.1. The first two shafts were sacrificial (I-225 and County Line) and the others at Franklin and Broadway were production shafts. The referenced name and locations of these test shafts are:

- I-225. The center of the I-225 test shaft is at Northing 654316.813, Easting 986153.558, and Elevation 5643.844. It is located in the general area of the I-25/I-225 interchange. Several new structures supported on drilled shafts are planned for the general area of this major intersection. Terracon (Attwooll, 2002) performed the geotechnical subsurface investigation and provided the geotechnical design recommendations for this and the County Line bridge structures.
- County Line. The County Line test shaft was 10 feet from Boring CL-2. CL-2 is at Northing 630111.259, Easting 995892.360 and Elevation 5885.859. It is located in the area between the outside shoulder of southbound I-25 and the exit from I-25 to County Line Road. This test shaft is near the production shafts for the new I-25 bridge over County Line Road.
- Franklin. The production test shaft is located beneath the west column of the 2nd pier column of the Franklin Bridge over I-25 (2nd from west abutment). Shanon and Wilson (2002) performed the geotechnical subsurface investigation and provided the geotechnical design recommendations for this structure.

- Broadway. Production shafts will support the new bridge structure (F-16-JK) that will carry I-25 over Broadway. Ground Engineering Consultants (2002) performed the geotechnical subsurface investigation and provided the geotechnical design recommendations for this structure. The Broadway test shaft is located along bent 6 (6^{th} bent from west abutment) beneath the north side of the center column (center column is supported by two shafts).

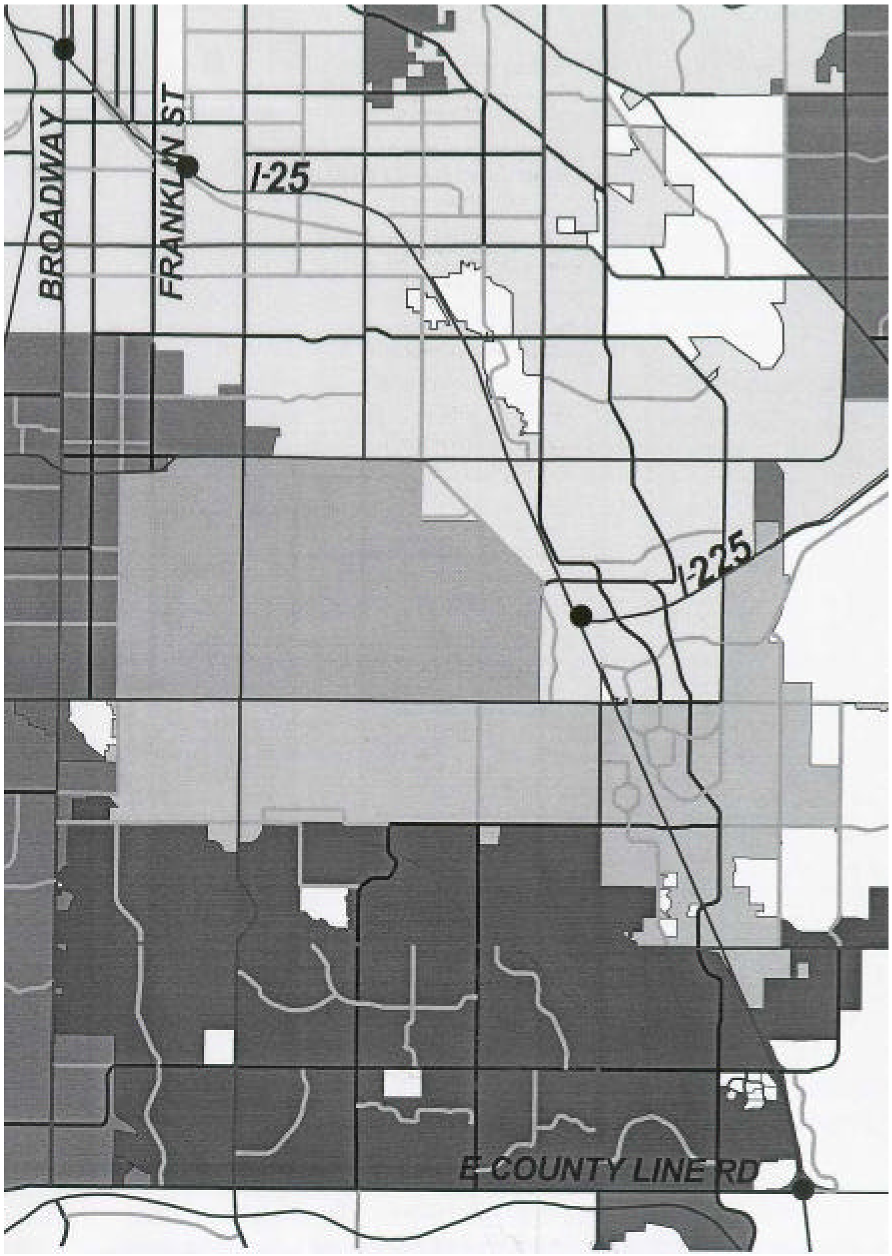


Figure 1.1. A Location Map for the Test Shafts Constructed in Denver, Colorado

Bedrock characteristics at the load test sites represent the range of typical claystone, sandstone or interbedded sandstone/claystone bedrock encountered in the Denver metropolitan area (see Chapter 4 for more details). The weathered claystone bedrocks at I-225 and County Line behave more like very stiff to hard clay than “rock,” because they are sedimentary clays that have been heavily overconsolidated by the weight of several hundred feet of overburden that was subsequently removed by geologic processes. The Franklin bedrock consists mostly of thinly bedded, bluish gray and sandy claystone (called locally Denver Blue shale). The Broadway bedrock consists predominately of well-cemented, bluish gray, and clayey sandstone (Denver Blue). CDOT standards describe the bedrock for the I-225 and County Line test shafts according to their standard penetration resistance (N in units of bpf) as firm (20<N<30), or medium hard (30<N<50), or hard (50<N<80). CDOT standards describe the bedrock for the Franklin and Broadway test shafts as very hard (N>80). The unconfined compressive (UC) strength ranges from 8 to 16 ksf for the claystone of the I-225 and County Line shafts, 40 to 80 ksf for the Franklin claystone, and 85 to 300 ksf for the Broadway sandstone. The County Line and I-225 very weak claystone is classified as very stiff clays according to the AASHTO, CFEM, and FHWA manuals. The Franklin and Broadway bedrock is classified as rock according to AASHTO and CFEM.

In this study, the claystone at the County Line and I-225 site, is referred to as “soil-like claystone,” as “very hard sandy claystone bedrock” for the Franklin bedrock, and “very hard clayey sandstone bedrock” for the Broadway bedrock. The word “shale” was avoided in this study to be as correct geologically as possible. The difference between claystone and shale is mainly that claystone is more massive with fewer bedding planes. Shale is typically fissile meaning that it breaks into thin sheets (laminated) along bedding planes. The “shale” definition is appropriate for the Franklin weak rock.

To maximize the benefits of the four O-Cell load tests, this study documented all the details of these test shafts and performed an extensive program of simple and diversified geotechnical tests at the load test sites. The analysis of the all test data and information and experience gained in this study were employed to provide preliminary recommendations to improve the axial design methodology for Colorado drilled shafts socketed in weak rocks, and to develop a comprehensive plan to fulfill CDOT strategic objectives listed above. These are described in more detail in the following Section.

1.3 Overview and Organization of the Report

Appendix A presents the definitions of all terms and units used in this report. All units presented in this report are in ksf for strength and resistance values and number of blows per foot (bpf) for SPT-N value.

In this report, a weighted average load factor of 1.5 was assumed and used to estimate the factor of safety (FS) from the recommended resistance factor ϕ as FS=1.5/ϕ. The structural engineer should calculate the actual design load factors, based on the actual loading conditions experienced by the bridge structure, and use it to estimate FS.

Appendix B and Chapters 2, 3, 4, and 5 provide the supporting material for the study findings and recommendations presented in Chapters 6 and 7.

1.3.1 Overview of the Study Research Approach

Appendix B provides a complete description of bedrock formations likely to be encountered in the Denver metropolitan area and other populated areas along the Front Range Urban Corridor. The general distribution, general geologic descriptions of specific formations, typical depths to bedrock, and "typical" bedrock hardness are discussed in Appendix B.

Chapter 2 presents a summary of the FHWA/AASHTO design methods for different geomaterials, the CSB design method, and other Colorado design methods that are based on strength data and load tests. It also describes the CDOT, AASHTO, FHWA, and CFEM standards for description and classification of geomaterials, including weak rocks, and the FHWA definitions of ultimate base and side resistance.

The layout, construction, and materials of the test shafts are fully described and documented in this report (Chapter 3). In addition, the CDOT Research Office administrated extensive geotechnical investigations (relative to normal practice) at each of the four load test sites (described in Chapter 3). Within 10 ft of the edge of each test shaft, three test holes were drilled to enable conducting the SPT and PM tests, and to recover rock core specimens for conducting the UC strength test on intact core specimens in the lab. Chapter 3 also describes the O-Cell load test and the new procedure developed by the CDOT Research Office for the analysis of the raw data of the O-Cell load test, including the construction of an approximate load-settlement curve.

Chapter 4 provides and discusses the following testing results at each load test site:

- Logs of the auger and core drilling that shows the extent, type of different soil/rock layer, and results of SPT-N values (bpf) every 5 ft.
- Laboratory test results including q_{ui} and E_i for the weak rock layers.
- The PM test results on the stiffness (E_m) and strength for the weak rock layers.
- Results of O-Cell load tests, including the load transfer curves (f-w and q-w curves), normalized versions of these curves, q_{max} and f_{max} of different weak rock layers, and the load settlement curve of each test shaft.

The results of all tests are thoroughly analyzed in Chapter 5 to provide:

- The best correlation expressions to predict the unconfined compressive strength of weak rocks (q_{ui}) from the results of insitu PM and SPT test data.
- The best PM modulus to represent the Young's modulus of the rock (E_m) and correlation expressions between q_{ui} and E_m.
- The best-fit design equations to predict f_{max} and q_{max} and an approximate load-settlement curve based on measured SPT-N-values, UC-q_{ui} data, and PM-E_m data.
- An assessment of the Colorado SPT-Based (CSB) design method, including the range of factor of safety measured with this method.
- An assessment of the AASHTO and FHWA design methods by comparing the prediction for f_{max} and q_{max} from these methods, using the measured strength and stiffness data, with those measured from the O-Cell load tests.

1.3.2 Overview of the Study Findings and Recommendations

Chapter 6 summarizes all the study findings of Chapter 5. It also provides specific details for two implementation products. The third implementation product is presented in Chapter 7. The reader who wishes to obtain more specific details of these products should turn to Chapters 6 and 7.

Product 1: Preliminary design methods for drilled shafts with conditions (geotechnical and geological description of the weak rocks, layout, construction, and materials of test shafts) close to those encountered at the four load test sites. The design methods include design equations and resistance factors needed in the LRFD method to predict the shaft's design base and side resistance value and settlement at service loads as a function of the results of simple geotechnical tests. The qualifications for using the design recommendations are presented (e.g., adequate subsurface geotechnical investigation, rapid drilling and concreting, no water in the socket from a perched water table or contractor practice, no use of slurry in the rock socket, and long-term performance). It is also expected that the rock sockets of all production shafts in Colorado will be roughened with simple side cutters. Conservative scaling recommendations for using the design methods for shafts with diameter larger than 5 ft are provided. Recommendations for the minimum embedment length of drilled shafts in rocks are also provided.

Product 2: Preliminary recommendations for improvement and standardization of the geotechnical subsurface investigation procedures for drilled shafts embedded in weak rock formations (definition for adequate subsurface geotechnical investigation, SPT, sampling procedure, UC test, and PM test).

Product 3: A detailed and comprehensive plan to fulfill the CDOT strategic objectives for drilled shafts that include three parts (covered in six tasks).

Part I. A new research study to 1) improve and standardize the geotechnical subsurface investigation procedures in Colorado, and 2) acquire resistance parameters for the LRFD method to account for measurement errors due to equipment or testing procedures, and to account for the spatial variability of subsurface materials at the site. This is covered in Task 1.

Part II: The assembly of a database that contains the results of old and new load tests on typical Colorado drilled shafts; information on the materials, layout, construction of test shafts; and geological and geotechnical descriptions of the foundation bedrock at the load test sites (i.e., from results of SPT, UC, and PM tests). Three tasks are needed in this phase:

Task 2: Literature review. To acquire old load test data and other information in weak rock formations as those in Colorado. Possible sources of these data and the criteria for accepting these data are outlined. This task will review the literature for other information relevant to this study (e.g., explore the use of new innovative load tests, other than the O-Cell, in Colorado).

Task 3: Acquisition of new geotechnical data at sites of existing load tests. For those sites in Colorado identified in Task 1 as having acceptable load tests, but in which adequate geotechnical data do not exist, additional geotechnical data on the weak rocks will be acquired. Complete details of the geotechnical subsurface investigation at these sites (SPT, PM, and UC tests) are outlined.

Task 4: Acquisition of new geotechnical data at sites of new axial load tests. At minimum, a total of 24 loads tests on four types of weak rocks are needed. Based on the findings of Tasks 1 and 2, the preliminary number and location of new load tests will be determined. New load tests performed under this task can be considered as proof test, and as means for justifying potential savings to the construction project.

This task thoroughly addresses all the issues that the design and geotechnical engineer need to consider in planning and performing a load test in a given construction project, including: 1) purpose and promotion of load tests 2) location and number of load tests, 3) type of test shafts (production or sacrificial), 4) features, limitations, and cost of conventional and O-Cell load tests, 5) design of O-Cell load test, and 6) data collection at the site of the load test.

Part III: Analysis of all the information collected in the previous two steps and publication and promotion of the findings (covered in Tasks 5 and 6). As a starting point, four categories of weak rock are suggested, but the analysis should finalize the proper number of these categories and their description. For each category of weak rock, the analysis will identify the most appropriate geotechnical axial method (design equations and resistance factors) for drilled shafts embedded in different categories of weak rocks encountered in Colorado. The analysis will provide the conditions, qualifications and limitations for using these design methods. The typical range of strength, stiffness, and ultimate base and side resistance for each category of weak rock layer will be obtained. Finally, the analysis will refine/validate the correlation expressions suggested in this study between results of various simple geotechnical tests.

2.0 GEOTECHNICAL DESIGN METHODS FOR DRILLED SHAFTS

Because different types of geomaterials behave differently, different design methods are recommended for each category of geomaterial in the literature. Initially, this chapter presents descriptions of these categories of geomaterial and the definitions for their ultimate resistance. Then, this chapter presents a brief summary of a number of current geotechnical design methods for predicting the ultimate base resistance (q_{max}) and ultimate side resistance (f_{max}) of weak rocks and rocks. The main sources for these design methods are:

- The 2002 17th Edition of Standard AASHTO and the 1998 LRFD AASHTO with 2002 interim revisions.
- FHWA (1999) Design Manual for drilled shafts.
- The design methods employed in CDOT and Colorado consulting geotechnical firms.

Chapter 3 will provide the test data needed in the design methods described in this chapter. Default values for strength, stiffness, and Poisson's ratio of weak sedimentary rock formations (siltstone, sandstone, and claystone) are available in AASHTO (2002 and 1998) specifications.

2.1 Descriptions of Geomaterials and the Definitions for their Ultimate Resistance

CDOT procedures for description of the bedrock are based on the penetration resistance (bpf) or N. It is called weathered claystone or clay if N<20, firm if 20<N<30, medium hard if 30<N<50, hard if 50<N<80, and very hard if N>80. In the Canadian Foundation Engineering Manual (1992) and AASHTO (1998, 2002), design methods are given for either soils or rocks. According to CFEM, soft or weakly cemented rocks with unconfined strength less than 20 ksf should be treated as soils, although geologically they may be referred to as rocks. For rocks with unconfined strength between 20 ksf and 100 ksf, the rock is identified as very weak rock, and between 100 ksf and 500 ksf it is identified as weak rock. The boundaries between soils and rocks are not clear in AASHTO. A statement was found in AASHTO (1998) that implies that cohesive soil can have unconfined compressive strength up to 37.5 ksf (1.8 MPA), and cohesive geomaterial with greater strength values should be treated as rocks. Cohesionless geomaterials with SPT N less than 100 are considered sandy soils.

The most complete classification for geomaterials is described in the 1999 FHWA design manual:

- Cohesive soil (clay or plastic silt with unconfined compressive strength less than 10 ksf).
- Granular soil (sand, gravel, or non-plastic silt with average SPT blow counts less than 50 blows per foot (bpf)).
- Rock (cohesive, cemented geomaterial with unconfined compressive strength larger than 100 ksf).
- Intermediate geomaterials (IGM's) are transition material from soils and rock, divided into three categories:

  I. <u>Argillaceous geomaterial:</u> Heavily overconsolidated clays, clay shales that are prone to smear when drilled. The compressive strength is between 10 ksf and 100 ksf. These materials exhibit excessive strength loss due to drilling or upon exposure to free water present in the formation, water used to assist in the drilling process, or from water migrating from the fluid concrete. Appropriate identification tests are described by FHWA (1996). This type of rock is just a hard soil, with no real cohesion.

  II. <u>Calcareous rocks</u>: Limestone, limerock and argillaceous geomaterials that are not prone to smearing when drilled. The compressive strength is between 10 ksf and 100 ksf. These materials are distinguished from the previous category in that the potential for strength loss at the face of the borehole due to drilling or presence of water is less. This is a true rock with cohesion. In Texas, these kinds of rocks contain calcium carbonate, so they are called calcareous rocks.

  III. <u>Very dense granular geomaterials</u>: Residual, completely decomposed rock and glacial till with SPT N values between 50 and 100 blows per foot (bpf).

Colorado Testing Procedure 26-90 (Slake-Durability Test) is a testing procedure to classify the shales as soil-like (nondurable) or rock-like (durable) shale. This procedure can be utilized to differentiate Category I from Category II of the IGM's.

Ultimate resistance can be defined to correspond to full mobilization of the plastic resistance (true resistance), to initiation of fracturing in the rock, or to a specified displacement. Differences in these criteria among the design methods lead to different design output results. The FHWA design manual (1999) establishes the ultimate resistance for compression loads as (1) the plunging load for cohesive soils; (2) the load corresponding to an arbitrary settlement of 5% of the shaft diameter for granular soil and IGM's, and (3) 1 inch for rocks and cohesive IGM. All design methods presented in the FHWA manual yield resistances corresponding to these definitions of failure. The true bearing capacity of the IGM's and rocks beneath the base will likely be higher than these values and will be achieved at a displacement of 5 to 10 percent of the base diameter. However, most of the load tests for which data can be obtained will not have been carried to a deflection sufficient to produce complete base or side failures. Most tests, however, have been conducted to a displacement of at least 1 inch (25 mm). In addition, displacement criteria are chosen for some materials to control the settlement that will be induced under service loads. Therefore, in order to produce a meaningful, albeit conservative, design method, the 1-inch (25-mm) deflection criterion was suggested in the FHWA design manual.

2.2. AASHTO and FHWA Design Methods

2.2.1 Design Methods for Cohesionless Intermediate Geomaterials (50<SPT-N<100)

For very dense sands, AAHTO (1998) recommends several design methods. Using SPT blow count, N (bpf), the base resistance (ksf) can be estimated as:

q_{max} (ksf) = 1.2 N, with an upper limit of 90 ksf for $N>75$..2.1

Assuming a factor of safety of 2.4, q_{all} (ksf) = 0.5 N.

For the side resistance (ksf) in very dense sand, AASHTO (1998) recommends, among different methods,

f_{max}= q_{max}/20 for N< 53, ...2.2a

and

f_{max} (ksf) = 0.044 (N -53)+ 3.1 for N>53..2.2b

Better design methods for cohesionless IGM's are described in the FHWA design manual (1999) with units in ksf as:

q_{max} (ksf) = $1.07\sigma_v^{0.2}(N)^{0.8}$, ...2.3a

and

$f_{max} = K_o\sigma_v \tan\varphi$,..2.3b

where N is the average SPT blow count in which 60% of the potential energy of the hammer is transferred to the top of the driving string; σ_v (ksf) is the vertical effective stress (at the base for base resistance, middle of the layer for side resistance); K_o is the at rest coefficient of earth pressure, and φ is the angle of internal friction. Expressions for K_o and φ as a function of N and σ_v are described in the FHWA design manual.

AASHTO (1998) and FHWA (1999) did not recommend factors of safety and resistance factors for the design methods listed above, because of shortage of load test data. It is recommended to utilize factor of safety and resistance factors based on judgment and experience.

2.2.2 Side Resistance Design Methods for Cohesive Stiff Clays

In cohesive soils, the side resistance, in ksf units, can be estimated as:

f_{max} (ksf) = α q_{ui} (ksf)..2.4

α is zero between the ground surface and a depth of 5 ft or to the depth of seasonal moisture changes, whichever is deeper. α is also zero for a distance of the diameter of the shaft above the base. Elsewhere, different approaches are adopted to estimate α

- FHWA 1999: α= 0.28 for q_{ui} less than 6 ksf, and from there change linearly to 0.23 for q_{ui} = 10 ksf. A resistance factor of 0.65 is recommended.
- AASHTO Standard 2002: α is taken as 0.28 for cohesive soils (boundaries between cohesive soils and rocks are not defined). A resistance factor of 0.55 is recommended.
- AASHTO LRFD 1998: α= 0.28 for q_{ui} less than 8 ksf, and from there decrease with the increase of q_{ui} until q_{ui}=36 ksf, for which α= 0.16. Geomaterial with q_{ui}>36 ksf should be treated as rock. A resistance factor of 0.65 is recommended.

For the Denver weathered claystone shale with unconfined compressive strength less than 12 ksf Turner et al. (1993) recommended to estimate α as:

$$\alpha = 0.21+4.2/q_{ui}\ (\text{ksf}) \qquad 2.5$$

2.2.3 Side Resistance Design Methods for Cohesive Intermediate Geomaterials

The side resistance design methods presented in this and the following section are intended for use only in intact rocks. When the rock is highly jointed, use Tables B.5 and B.6 in the FHWA design manual (1999) to develop a reduction factor to arrive at a final value of f_{max} and E_m (as related to estimated f_{max} and measured E_i) for design. This reduction factor is a function of RQD and condition of joints.

O'Neill et. al. (1996) presented an advanced design method to predict the f-w and q-w relations in cohesive IGM (Category I and II of IGM material, Section 2.1) and construct the load-settlement curve. This method is considered an advanced design method because it takes into account many important parameters that influence the shaft side and base resistance, including q_{ui}, roughness of the shaft hole, the normal effective stress between the concrete and borehole wall when compression loading is initiated (σ_n), (L/D), L, E_m, (E_c/E_m), and other parameters presented below. This procedure was developed based on results of parametric finite element analyses and load tests, for uniform rock sockets, 2<L/D<20, 1.6 ft<D<5 ft, 10<E_c/E_m< 500, 115<E_m/q_ui<500, and for roughness patterns observed in auger-cut clay-shales in Texas. The mathematical equations that describes the f-w and q-w relations were programmed for the purpose of this study in an Excel Worksheet program called CDOTSHAFT. Construction of the f-w relation requires the determination of $f_{max,}$ determined as described in the following.

It is necessary to describe first the roughness classification of the interface (rough or smooth) and the smear classification of the interface (smeared or non-smeared). IGM and rocks behave very differently if the borehole is smooth after drilling than if it is rough (as will be discussed in the next section). If the borehole is rough, its behavior depends upon whether the sides of the borehole have any highly degraded cuttings, or "smear", remaining on the borehole wall. If the designer wishes to assure rough conditions, he or she should:

(1) Require the contractor to cut circular grooves (keys) approximately 3" high into the sides of the borehole that will penetrate at least 2" into the borehole wall over the full 360 degrees around the hole at spacing no greater than 1.5 ft, or
(2) Be convinced that the normal drilling process will produce a roughness pattern equivalent to (1) above, without leaving soft soil-like material on the borehole walls.

Otherwise, the borehole should be assumed to be smooth for design purposes. A smooth socket should have some degree of natural roughness if the construction specifications call for the contractor to avoid or remove any smeared material on the sides of the borehole. Roughness conditions could be verified with results of load tests as will be discussed in Chapter 5. As discussed before, Category I IGM may tend to remold and soften at the concrete-IGM interface during shaft construction, so smooth conditions may be appropriate for this category. It is unlikely for smear zone to develop in Category II IGM. Stress relief during the time the borehole is open also permits the exposed soil to swell and lose strength further. This effect is proportional to the time the borehole is allowed to remain open prior to concreting. This time should be reduced as much as possible.

For a smooth socket,

$$f_{max} = \alpha \, q_{ui} \quad \text{.......... 2.6}$$

where α (maximum of 0.5) is obtained from a figure available in the FHWA design manual that is dependent on σ_n and the effective angle of friction between the concrete and the IGM material (assumed 30 degrees). If the rate of concrete placement is 12 m/hour or greater, $\sigma_n = M\gamma_c z_c$, where γ_c is the unit weight of concrete; M is an empirical factor that depends on the fluidity of the concrete as indexed by the concrete slump, z_c is the depth to the point at which σ_n is computed but not to exceed 40 ft.

The expression for side resistance in rough sockets that corresponds to fully mobilized plastic conditions is:

$f_{max}=q_{ui}/2$ 2.7

The fully mobilized plastic conditions are not achieved until the displacement is very large, and the computed value from Eq. 2.7 will be unconservatively large. Therefore, for a rough socket, the f-w relation is needed to estimate the side resistance at an assumed settlement corresponding to geotechnical failure. This was achieved with the program CDOTSHAFT developed in this study.

2.2.4 Side Resistance Design Methods for Rocks

The following are the methods adopted by AASHTO and the FHWA Design Manuals for estimating side resistance in rocks that could also be considered for cohesive IGM materials. The recommended AASHTO (1998) resistance factors are 0.55 for the Carter and Kulhawy method and 0.65 for the Horvath and Kenny method.

Based on the analysis of 202 data points from laboratory and field load tests on drilled shafts excavated without artificial roughening of the borehole wall, Horvath and Kenny (1979, from FHWA 1999) recommended the following expression for f_{max} (units in ksf for resistance and strength values).

$f_{max} = 0.95\, q_{ui}^{0.5}$ 2.8

If the borehole is artificially roughened, Horvath et al. (1983, from FHWA 1999) recommended the following expression:

$f_{max} = 0.8\,(RF)^{0.45}\, q_{ui}$ 2.9

where RF is a roughness factor that is a function of groove spacing (s), socket radius (r), and depth of grooves (h), expressed as RF=(h/r)*(s+2h)/s. For the roughness pattern described in the FHWA design manual (grooves depth is 2 inches ("in.") with a spacing of 1.5 ft), and for typical socket radius of 2 ft, RF can then be estimated as 0.1, and the design equation can be written as $f_{max} = 0.286\, q_{ui}$.

Rowe and Armitage (1988, from FHWA 1999) analyzed a database of about 25 drilled shaft sockets in a wide variety of soft rock formations, including sandstone, limestone, mudstone, and shale. Based on this work, Carter and Kulhawy (1988, from FHWA 1999) suggested the following formula (in units of ksf):

$f_{max} = \mu\, q_{ui}^{\ 0.5}$ 2.10

For rock that drills very smooth or when the rock is drilled under slurry, μ as low as 0.92 was found. This is very close to the value recommended by Horvath and Kenney, and it represent a lower bound. The socket could be rough, either through normal drilling or drilling with the use of artificial grooving. The factor μ= 2.05 is the average of all cases and is recommended for regular clean sockets with grooves between 0.04 in. and 0.4 in deep. Rowe and Armitage define a rough socket as having grooves deeper and wider than 0.4"with spacing between 2" and 8". The RF for this roughness pattern is 0.018, almost one fifth of the roughness pattern specified in the FHWA design manual ($f_{max} = 0.13\, q_{ui}$ according to Horvath and Kenny method). For a rough socket, the recommended μ = 2.75. The ratio 2.75/0.92, almost 3, again points out the importance of borehole roughness in hard geomaterial.

Other methods were described in the literature with either a power function of q_{ui} or a linear relation of q_{ui}. The linear relations are often developed for a small range of strength data while the power functions apply to a larger range of strength data.

2.2.5 Base Resistance Design Methods

In cohesive soils with UC strength larger than 2 ksf, the base resistance can be approximated as:

q_{max} = 4.5 q_{ui} 2.11

The recommended resistance factor for this method is 0.55 based on the work reported in NCHRP Report 343 (1991).

Eq. 2.11 generates the theoretical base resistance in massive rock that is loaded undrained and that has absolutely no joints. However, the q_{max} on rock and IGM's material is highly dependent upon the structure and spacing of the joints in the geomaterial within 2 shaft diameters (D) of the base of the footing. The rock mass quality can be related to RQD as described in Table 4.4.8.1.2A of Standard AASHTO 2002. In the following, it is assumed that the cohesive IGM or rock is intact and the joints (or discontinuities) are closed and spaced more than 10 ft apart. For other descriptions of the rock mass, Table 4.4.8.1.2A of AASHTO 2002 can be utilized to develop a reduction factor as a function of RQD to arrive at a final value of q_{max}.

If the length of the socket (L) is larger than 1.5 times that the socket diameter (D), the FHWA design manual recommends

q_{max} = 2.5q_{ui},...2.12

where q_{ui} is the UC strength of intact cores collected within 2D below the base. This criterion was developed to correspond to the occurrence of fracturing in the rocks. Therefore, this criterion does not correspond to complete plastic failure or theoretical base resistance as in cohesive soils or soil-like claystone. The reason for the selection of this fracture-based criteria is because if the rock fractures (even though it will develop a higher resistance than 2.5 q_{ui}) it will eventually weaken due to stress relief along the fractures, and perhaps water infiltration through fractures. Thus, long-term capacity, which load tests such as those reported in this study do not address, may be less than short-term capacity, which they do address. If the rock is sensitive to water (has high slake loss as determined from Colorado Testing Procedure 26-90), the 2.5 q_{ui} criterion should be considered the ultimate bearing capacity, even in massive claystone and sandstone bedrock.

In the O'Neill et. al. design method (1996) described in the previous chapter, the q-w curve is a function of E_m, L/D, L, E_c/E_m , but it is not a direct function of the unconfined compressive strength of the IGM. With this method, q_{max} can be estimated at an assumed settlement but should not exceed 2.5 q_{ui}. In addition, the design base resistance at working loads should be limited to q_{ui} (FS=2.5).

AASHTO (2002) recommends this design method to estimate q_{max}:

$$q_{max} = \Psi (q_{ui}) \quad \text{.......} 2.13$$

Where Ψ is 4.3 for siltstone and claystone, and 5.0 for sandstone.

AASHTO LRFD (1998) recommends utilizing the Canadian Foundation Engineering Manual (1992) method with resistance factor of 0.5:

$$q_{max} = 1.2 \Lambda(q_{ui}) \quad \text{.......} 2.14$$

Where Λ is a depth factor = $(1+0.4 L/D) \leq 3.4$. This is a proven design method for cohesive IGM with horizontal layering in which the joints are closed.

Zhang and Einstein (1998, from FHWA 1999) have analyzed a database of drilled shaft load tests that were cast on or in generally soft rock with some degree of jointing (RQD between 70 and 100 percent, all joints are approximately horizontal and closed). For definition of failure, they used a combination of settlement of 4% of the base diameter where the shafts were pushed that far in the load test and judgment elsewhere. They have recommended the following best-fit expression for the data:

q_{max} (ksf) = 21.4 $(q_{ui})^{0.51}$ 2.15

The ratio (q_{max}/q_{ui}) ranges from 6.9 for q_{ui}= 10 ksf to 2.24 for q_{ui}= 100 ksf to 1.56 for q_{ui} = 200 ksf. There is no associated resistance factor for this method, therefore, a small resistance factor of 0.5 is recommended.

If settlement estimates are not to be performed, it is prudent to limit the base resistance for large-diameter drilled shafts in cohesive and cohesionless soils and intermediate geomaterials (FHWA, 1999). This is because the service loads that will be calculated from the design equations presented above and the factors of safety and resistance factors normally used in drilled shaft design may result in excessive settlements for large-diameter drilled shafts. Recommended values of reduced q_{max} values in terms of the computed values, when the shaft diameter is larger than 6 ft for cohesive soil and IGM and larger than 4 ft for cohesionless soils and IGM, are given in the FHWA design manual.

2.3. Colorado and CDOT Design Methods

2.3.1 Colorado SPT-Based Design (CSB) Method

Since the 1960s, empirical methods and “rules of thumb” have been used to design drilled shafts in the Denver Metropolitan/Colorado Front Range area. This empirical formula is geared toward allowable stress design method (ASD) because it predicts only the allowable geomaterial resistance from the results of the standard penetration test (N-value). The same CSB design method is uniformly applied by many geotechnical engineers to both cohesive and cohesionless weak rocks and to stronger rocks. That is, unit resistance is based only on SPT-N values. Rock classifications are secondary and are used to anticipate construction problems. On the other hand, different design methods are employed for different geomaterials (soils and rocks) in the

AASHTO/FHWA design manuals (discussed in Section 2.2). Other deficiencies of the CSB design method were discussed in the first chapter (Section 1.2).

In the CSB design method, the allowable base resistance in kips per square foot (ksf) is taken as:

q_{all} (ksf) = q_{max}/FS = 0.5 N...2.16

and the allowable side resistance is taken as:

f_{all} (ksf) = f_{max}/FS = N/20...2.17

Determination of an appropriate N value (SPT blow counts for penetration of 1 ft) for design purposes varies based on uniformity of subsurface conditions (or lack of), type of structure, and the engineer's experience. N is sometimes taken as the average value for the geomaterial layer under consideration, the lowest average N of a layer in all test holes, or perhaps even the absolute lowest N value obtained for a layer. If there is a clear variation in blow count data along the shaft length (e.g., blow counts increase with depth), the recommended allowable side shear varies along the shaft length. With the lack of information on the proper factor of safety embedded in the CSB design method, a factor of safety of 3 is often assumed and used to recommend ultimate base and side resistance values as:

q_{max}= 3q_{all},...2.18a

and

f_{max}= 3 f_{all} 2.18b

Because the CSB design method is rather crude, most practitioners limit the allowable base resistance, for geomaterials with N>100, to about 50 ksf (ultimate to 150 ksf) and allowable side resistance to 5 ksf (ultimate 15 ksf). The new football stadium in Denver was designed for allowable base and side resistance values of, respectively, 50 and 5 ksf. The geotechnical engineer for that project recommended load testing to justify higher values, but the owner declined due to scheduling issues.

The expression for q_{all} provided in Equation 2.16 matches the expression recommended in AASHTO LRFD (1998) for dense granular soils (Eq. 2.1). However, Equation 2.2 suggests that the side shear for dense granular soils is approximately 5% of the base resistance, not 10% as suggested in Eq. 2.17. Jubenville and Hepworth (1981) attempted to justify the use of the CSB design method only in the design of low capacity drilled piers which carry less than approximately 1000 kips. They concluded that the CSB method may be approximately valid for claystone bedrock with strength close to that of very stiff clays, and beyond this range, the utility of this method is questionable. They also indicated that the CSB method should be used with caution and understanding.

2.3.2 Colorado Strength-Based Design Methods

Few engineers in Colorado have used strength and SPT data combined with experience and knowledge of load tests to justify higher allowable bearing values. Most geotechnical engineers in Colorado do not elaborate on their analysis methods in their reports. They just provide an allowable or ultimate resistance value with possibly some general statements justifying a high value. Therefore, specific details of their recommendations and approaches could not be obtained from reviews of consulting reports or even through personnel communications.

Jubenville and Hepworth (1981) and Turner et al. (1993) reported the use of strength-based design methods for the design of high capacity piers used to support high-rise buildings in downtown Denver, built in the 1970's. The range of unconfined compressive strength for the "blue" claystone that underlies downtown Denver varies from 8 ksf to more than 60 ksf. Twenty-four PM tests conducted in the blue claystone underlying downtown Denver resulted in E_m ranging from 1400 ksf to 13000 ksf with an average of 5300 ksf, and limit pressure ranging from 73 ksf to 344 ksf, with average of 200 ksf. The piers, which carry loads to more than 6000 kips, are 4 to 9 feet in diameter, with penetration up to 30 ft into the blue claystone-siltstone. Generally, the piers are designed for q_{all} of 40 to 100 ksf, and f_{all} equal to 10% the q_{all}. The design equations utilized were

$q_{all} = 4.5\ q_{ui}/2.5,\ f_{all} = 0.1\ q_{all}$ 2.19

Jubenville and Hepworth (1981) described a procedure to estimate q_{all} based on the pressuremeter (PM) test data. They concluded that the predictions of q_{all} from the PM test data are more than three times those predicted by Equation 2.19, and therefore they give less weight to the PM test results. AASHTO LRFD (1998) describes a method to estimate q_{max} based on the PM test data, but this method was not pursued in this study because it leads to large resistance values as reported by Jubenville and Hepworth (1981). Instead, the PM test data were analyzed to extract E_m and indirectly to estimate q_{ui}.

In the Broadway viaduct project, the measured unconfined compression strength of the California sampler (this local method of sampling is discussed in more details in Chapters 3 and 6) specimens ranged from 3.1 ksf to 29.2 ksf with an average of 17.4 ksf. Much higher unconfined compressive strengths were obtained on two specimens obtained through coring: 183 ksf and 245 ksf. Based on these strength data, Table 2.1 lists the values of q_{max}, f_{max}, and resistance factors recommended for the design of the drilled shafts in the Foundation report prepared by the geotechnical engineering consultant (Ground Inc., 2002). The base resistance values are valid for a minimum caisson diameter of 1 ft and a maximum diameter of 75 in. to limit caisson movement to tolerable limits. Should caisson diameters exceed 75 in., the allowable base resistance should be reduced accordingly. The side resistance values assume that the upper 10 ft of the bedrock will get wet during caisson drilling, and shear rings (3 in. high, 2 in. deep, and 1.5 ft spacing) would be installed in the lower 10 ft of the caisson holes. In the event that the requirements for caisson roughening would be eliminated for any reasons, the side resistance should be reduced by 20 % if the caisson holes are dry at the time of installation, or by 40% if water is present in the caisson holes. The basis for all recommendations presented above was the results of the unconfined compressive strength tests and experience/knowledge with past load tests in Denver. More specific details of these recommendations, like the design methods and testing data used to obtain q_{max} and f_{max} or justification for the influence of hole roughening and presence of water, could not be obtained.

Artificial roughening was not required in the final construction plans of the Broadway project and, to be conservative, the design assumed the "wet condition" for the caisson holes. Therefore, the structural engineer applied a 20% reduction to the recommended resistance values in the design calculations (Table 2.1). The factor of safety corresponding to a resistance factor of 0.55 is 2.72. Thus, the equivalent allowable side resistance values recommended by the geotechnical engineer range from 2.2 ksf to 4.8 ksf, and those used by the structural engineer range from 1.77 ksf to 3.9 ksf. These values are substantially below those that would be recommended with the CSB design method. Therefore, the strength-based design method used in the Broadway project was more conservative than the CSB design method (assuming an SPT-N value of 100).

Table 2.1. Design Recommendations for the Broadway Shafts before and after the O-Cell Load Test

Loading Type	Resistance Type	Geotechnical Engineer Recommendation		Structural Engineer, Ultimate Resistance
		Ultimate Resistance	Resistance Factor	Ultimate Resistance
Before O-Cell Load tests				
Vertical Capacity	Side Resistance in Bedrock 1 to 10 ft	6.0 ksf	0.55	4.8 ksf
	Side Resistance in Bedrock below 10 ft	13.0 ksf	0.55	10.4 ksf
	Base Resistance	200 ksf	0.5	200
After O-Cell Load Test				
Vertical Capacity	Side Resistance in Bedrock	15 ksf	0.8	15 ksf
	Base Resistance	200 ksf	0.8	200 ksf

2.3.3 Colorado Load Test-Based Design Methods

Because of the expense of full-scale load tests on drilled shafts, there have been relatively few performed in the Denver area. A brief summary of design modifications based on load tests performed in Colorado are presented in this section. A future CDOT research study will document all the past and suitable load tests performed in Colorado (see Chapter 7).

Load tests reported by Turner et al. (1993)

Turner et al. (1993) reported the results of three load tests on shafts in the Denver Formation, in Denver, Colorado, and ten load tests were conducted on shafts in Pierre Shale, at sites in South Dakota and Colorado. Only ultimate side resistance data were obtained from these load tests. The measured ultimate side resistance values are compared to predictions from some of the design methods presented in the previous section. Conclusions were:

- Use Eq. 2.6 for the Denver weathered claystone with unconfined compressive strength less than 12 ksf.
- Use Horvath and Kenny method (Eq. 2.8) for claystone with unconfined compressive strength larger than 12 ksf.
- Undertake efforts to replace the use of N-values for the determination of design side resistance by methods based on strength data.
- Apply the CSB design method with caution. Analysis of load tests indicate factors of safety in the range of 0.8 to 1.6 when this method is used.

Load Tests Performed in the T-REX and Broadway Viaduct Projects

The results and analysis of these load tests are discussed in more detail later in this report.

Based on results of O-Cell load testing at the I-225 and County Line sites (soil-like claystone), Attwooll (2002) concluded that the factor of safety in side shear is generally low (unconservative). Attwooll (2002) recommended keeping the CSB design method as it is with the following modifications:

- For the side resistance, disregard the upper 5 ft of the shaft in the geomaterial. This is as recommended in the FHWA design equation for cohesive soils. It is also recommended in the FHWA design method to disregard the lower zone of the shaft for a distance equal to the diameter of the base.
- For the base resistance, use lowest N-value measured beneath the shaft tips. For layers with varying strength within 2D below the base, AASHTO recommends considering the layer with the lower strength value.

The results of O-Cell load tests at the Franklin and Broadway sites indicated that the traditional CSB design method is conservative when shafts derive their support in the hard Denver blue shale. No design modifications were recommended at the Franklin site because the construction project was progressing on a fast track schedule. The geotechnical engineer for the Franklin Bridge indicated the bedrock at the site varies in strength and the bedrock at the test shaft did not represent the weaker rock within the entire Franklin site.

Based on the results of the O-Cell load test at the Broadway site (presented later) performed during Phase I of the construction project, the geotechnical engineer for the Broadway project recommended changes to q_{max}, f_{max}, and resistance factors as listed in Table 2.1. The measured side resistance for the Broadway shaft between the O-Cell and Level 1 strain gages (Chapter 4) was not considered due to nonlinear distribution of the side resistance in the bedrock penetration zone and non-uniform stress distribution immediately above the O-Cell. The measured side resistance between Level 1 and Level 2 strain gages at the end of the O-Cell load test (15.9 ksf) was recommended for the design. The strength limit state of the base resistance was not reached in the O-Cell load tests. At settlement of 5% of the shaft diameter, the estimated base resistance was 218 ksf (close to the recommended value of 200 ksf). These were the lessons learned from the O-Cell load tests performed at the Broadway project:

- There was no appreciable savings in Phase 1 of the construction project. This was attributed to the following: 1) materials had already been delivered and assembled (reinforcing cages) for the remaining production shafts; 2) the depth of overburden was larger than stated in the construction plans; and 3) minimum socket length of 3D governed the design of shafts supporting the ramp.
- The computed potential savings in future phases of the Broadway project is approximately \$89 K. This is computed using an estimated reduction of penetration length in the bedrock of 356 ft and a unit price of \$250 /ft for future shafts. More savings are expected in the future for construction of bridges close to the Broadway site (Santa Fe and Alameda Interchanges) and in bedrock formations like those encountered at the Broadway site.
- The bidding cost of the two O-Cell load tests was \$140 K (\$70 K for each test).

3.0 DESCRIPTION OF TEST SHAFTS, LOAD TESTS, AND GEOTECHNICAL INVESTIGATIONS

3.1 Introduction

This chapter provides a description of the materials and construction of the four test shafts (listed in Chapter 1), geotechnical field and laboratory tests performed at or near the test shafts (O-Cell load test, SPT, UCT, and PMT), and a description of the analysis employed to extract results from the testing data. Three groups of tests are described in this chapter:

- Osterberg Cell (O-Cell) load tests.
- Laboratory and field tests performed by the geotechnical engineering consultants for the construction projects**.** When test results reported by the geotechnical consultants vary significantly from those obtained by CDOT, they are briefly presented and discussed.
- Extensive laboratory and field tests administrated by the CDOT Research Office.

Recommendations to improve the geotechnical subsurface investigations are presented in Chapter 6 (Section 6.5). These recommendations are based on the lessons presented in this chapter with new and existing geotechnical testing techniques and comparison of the subsurface geotechnical investigation (procedure and results) performed by CDOT and Colorado geotechnical firms at the four load test sites.

Results of O-Cell load tests and of the geotechnical laboratory and field testing program administrated by CDOT are presented in Chapter 4. These results will be referred to in this chapter.

3.2 Osterberg Cell (O-Cell) Load Tests

A schematic of the O-Cell test and a photograph of the Osterberg Cell are shown in Figures 3.1 and 3.2.

3.2.1 Test Shaft Construction

Drilling for all test shafts was performed by Anderson Drilling, Inc. The construction of these shafts is representative of the typical construction procedure for production shafts employed in the Denver area and in the T-REX and Broadway construction projects.

Construction information and layout information for each test shaft are listed in Table 3.1. Construction information includes: date of construction, time required for excavation of the shaft hole, amount of ground water accumulated at the base of shaft hole at end of the drilling operations, information on the smoothness of the shaft side walls, slump and placement time of the fresh concrete, and the concrete compressive strength (f'_c) at time of load test. Included in Table 3.1 are also the composite Young modulus of the shaft, E_c, defined as the sum of the concrete area multiplied by concrete modulus and steel area multiplied by the steel modulus divided by the entire test shaft area. The values of E_c were taken from the LOADTEST, Inc. test reports (2002). The concrete modulus (ksf) was estimated using the ACI formula as 8208 $(f'_c)^{0.5}$, where the units of f'_c are in psi. Layout information of each test shaft includes diameter of the shaft (D), length of shaft in the overburden (L_o), length of the bedrock socket (L), and depths to: groundwater level (GWL), competent bedrock, top and base of the shafts, the O-cell, and the 1st and 2nd levels of strain gages (SGs, see Figure 4.1a).

<u>Excavation Method:</u>

Drilling was performed with a flight auger placed at the end of a Kelly bar powered by the drill rig. Cutting teeth were attached to the base of the auger that extended approximately 0.5 in. beyond the edge of the auger to provide sufficient clearance to facilitate getting the auger in and out of the shaft hole. The drillers did not add any water during drilling to aid in picking up of the cuttings.

The test shafts at I-225 and County Line, embedded in the soil-like claystone, were drilled, respectively, with 42- and 48-inch diameter augers. When the shafts reached their intended depths, the lower 8 to 10 ft of the shafts were roughened by replacing the outer cutting teeth with a “roughening” tooth that extended about 1.7” beyond the edge of the auger. The roughening consisted of spinning the auger and cutting shallow grooves in the sides of the holes at about 6-inch vertical spacing. The primary purpose of the roughening is to somewhat remove the polished skin of the remolded material that can sometimes form in the softer claystone bedrock (i.e., remove smear zone). Expected depth of roughening in the intact rock is 0.5 in. to 1 in., which is less rigorous than roughening with shear rings. After roughening was completed, the outer tooth was removed. The base and side of shaft holes were then cleaned by spinning and removing the auger several times until little, if any, loose soil spoils were obtained. It was observed that the bases of the shafts were clean and very little water was present at the base of the shafts before concrete placement.

The GWL at the Franklin site had to be lowered using a side pump because the GWL was located in the overburden very close to the ground level. The GWL at the Broadway site was located at almost the level of the competent rock. The test shafts at the Franklin and Broadway sites were initially drilled with, respectively, 48-inch and 60-inch diameter augers to the top of the very hard rock. The hole sides were stabilized with natural slurry made of the on-site soil. Casing was then installed and screwed into the top 1 to 2 ft of the rock. The slurry inside the casing was then removed with a mud bucket. Casings were specified to keep the hole dry in the socket and to keep the overburden stable.

At the Broadway site, a 4-ft diameter auger was used for pre-drilling the bedrock socket and a 4.5-ft auger was then used to obtain the nominal socket diameter and to complete the excavation of the bedrock socket. For the Franklin test shaft, a 3.5 ft auger was used for drilling the nominal bedrock socket diameter. No artificial roughening efforts were employed for the Franklin and Broadway test shafts, as for the County Line and I-225 sites, because of the expectation (based on observations) that normal drilling and cleaning in the very hard rocks creates clean, intact shaft walls with no smear zones. However, the drillers believe that normal drilling in the very hard bedrock at the Franklin and Broadway sites creates naturally rough sockets as reported in the literature (Section 2.2). During drilling, the shaft sides were dry all the way to the bottom of the Broadway shaft. No similar information is available for the Franklin test shaft during the drilling operations. The base and side of shafts were cleaned with a mud bucket and/or auger.

Prior to concrete placement, the base of the Broadway shaft was dry and 18 inches of water was left at the base of the Franklin shaft.

Concrete Placement:

Immediately after the hole cleaning operations were completed, placement of the concrete started. Concrete was placed relatively slowly with a tremie pipe to keep the concrete under water and to avoid mixing the concrete with this water. The concrete slump, required by CDOT specifications to be 5 to 8 inches, was kept on the high side or slightly above the upper CDOT limit (Table 3.1). A seating layer of plain concrete (called "a reaction socket") was pumped in the base of the shafts, and then the reinforcing cage, with O-Cell attached at the bottom of the steel cage, was inserted in the wet concrete at the top of the reaction socket. The remainder of the concrete was slowly pumped by tremie pipe until the top of the concrete reached the targeted elevation. The temporary casing was pulled out and additional concrete was added to maintain the targeted elevation for top of concrete. The length of the reaction socket for each test shaft can be determined from Table 3.1 as the difference between the depth to the O-Cell and the depth to the tip of the shaft. This length was small for the I-225, County Line and Franklin test shafts (0.5 ft to 1.5 ft), but was large for the Broadway shaft (6.3 ft). The large length of reaction socket at Broadway caused difficulties and uncertainties in analysis of O-Cell test data for base resistance, as will be discussed later. The large length of the Broadway reaction socket was attributed the following reasons that should be alleviated in future O-Cell load tests on production or test shafts:

- During drilling, the depth to top of the rock was greater than expected. However, the length of the reinforcing cage was fixed and prepared in advance based on expected depth to top of competent rock as provided in the construction plans. Test boreholes should be performed in the future at the exact location of load test to determine the proper depth to top of competent rock.
- The designer decided at the last minute to increase depth of penetration in the competent rock by two feet because of concerns of inserting O-Cell in the production shafts. Such additional depth should be accounted for in the construction plans.

Table 3.1. Construction, Materials, and Layout Data for the Test Shafts

Test Shaft Name	I-225	County Line	Franklin	Broadway
Ground Elevation (ft)	5644	5886	5296	5255
Construction Date	1/8/2002	1/8/2002	1/11/2002	1/12/2002
Excavation Time (hours)	~3	~3	~ 5	~7 hours
Amount of ground water accumulated at the base of the shaft at end of the drilling	Dry	Dry	At least 18" (wet)	Dry
Smoothness of the shaft wall sides	Roughened to some extent with outer tooth in the lower 8 feet	Roughened to some extent with outer tooth in the lower 8 feet	Not artificially roughened, but suspected of being roughened with normal drilling procedure.	
Concrete Slump (inches)	9	7-9	7-9	7.5
Concrete Placement Time (hours)	2	2	3	4
Concrete Unconfined Compressive Strength (psi)	3423	3193	3410	3936
E_c or composite stiffness of the shaft (ksf)	0.53 x10^6	0.50 x10^6	0.53 x10^6	0.58 x10^6
Diameter of the shaft (D) in the bedrock socket (ft)	3.5	4	3.5	4.5
Depths to (in feet):				
GWL	15.5	Below Shaft tip	4	17.1
Competent Rock	12.5	8	4. 5	17
Top of the shaft	6	6	0	6.5
Level 2 SGs	15.75	11.5	11.7	20.75
Level 1 SGs	21.75	16.5	17.7	30.75
Base of O-Cell	27.75	21.5	23.7	40.75
Tip of the shaft	28.6	22	25.25	47.1
Length of shaft in the overburden, L_o (ft)	6.5	2	4.5	10.5
Length of rock socket, L, (ft)	16.1	14	20.8	30.1

3.2.2 Testing, Instrumentation, and Analysis

LOADTEST, Inc. performed all of the O-Cell load tests and prepared the test report for each test shaft (LOADTEST, Inc., 2002). See the referenced test reports for more details. The test assembly included a single O-Cell (Figures 3.1 and 3.2) which is a jack-like hydraulic device placed at the bottom of the steel reinforcing cage (Figure 3.1). When a fluid pressure is applied to the device, an equal upward and downward force is applied at the base of the O-Cell. The break in the test shaft occurs at the base of the O-Cell (i.e., the shaft moves upward above base of the O-Cell and downward below base of the O-Cell). A 21 inches-diameter O-Cell was used in the County Line and I-225 sites with a capacity of 2000 kips in each direction. A 34 inch diameter O-Cell was used in the Broadway and Franklin sites with a capacity of 6000 kips in each direction. The O-Cell is surrounded by top and bottom steel plates having 2 inches in thickness. The load increments were applied using the Quick Load Test Method (ASTM D1143), holding each successive load increment constant for four minutes by manually adjusting the O-Cell hydraulic pressure. The tests were continued until the ultimate side shear, the ultimate end bearing, or the capacity of the O-Cell was reached. The I-225 and County Line shaft dimensions and O-Cell depths were selected so that the shaft total base resistance load and side resistance load would be roughly equal and reach the ultimate conditions at the same stage. Unfortunately, this was not the case for the production test shafts at Franklin and Broadway. In the production shafts (Franklin and Broadway), the maximum applied load was limited by LOADTEST, Inc. (2002), to maintain the functionality of the shafts after test completion. After the O-Cell load tests were completed on the production shafts, the O-Cell and voids were filled with grout.

Instrumentation of the test shafts was employed to measure, collect, and store automatically, at 30 second intervals, the following data: 1) O-Cell gross load, 2) upward and downward movements of the O-Cell, 3) compression movement of the shaft, and 4) shaft strains at two levels: Level 1 SGs and Level 2 SGs (see Figure 4.1 and Table 3.1). Two strain gages are placed at each level.

The gross loads applied by the O-Cell act in two opposing directions, resisted by side shear above the O-cell and by both base resistance and side shear in the reaction socket below the O-Cell. Typical measured results from O-Cell testing showing gross O-Cell load versus upward and downward movement of the O-Cell are shown in Chapter 4. The net load at the base of the shaft is defined as the O-Cell load minus the buoyant weight of the shaft above the O-Cell. This load represents the total side resistance of the test shaft. The measured shaft strains, E_c, and cross-sectional area of the shaft (Table 3.1) were employed to estimate the axial load carried by the shaft at the two levels of strain gages. The changes in the net shaft axial loads from the O-Cell to Level 1, from Level 1 to Level 2, and from Level 2 to the top of the shaft (0 load), were employed to estimate the average net unit side resistance (or just side resistance in future references) in these zones. Examples of measured side resistance at different zones versus upward side movement are shown in Chapter 4. For top-loaded shafts, information for side resistance versus downward side movement (not upward as occurred inside the O-Cell) is needed. It is assumed in this study that the measured relations for side resistance versus upward movement (as in an O-Cell test) in any zone are equivalent to side resistance versus downward movement in that zone. Some have argued that a factor (generally assumed to be between 0.9 and 1.0) should be applied to side resistance values to account for the difference between a "push up" test vs. a "push down" test. A factor of 1 is employed in this study as recommended by LOADTEST, Inc (2002).

Figure 3.1. Photo of the O-Cell Placed in the Broadway Test Shaft

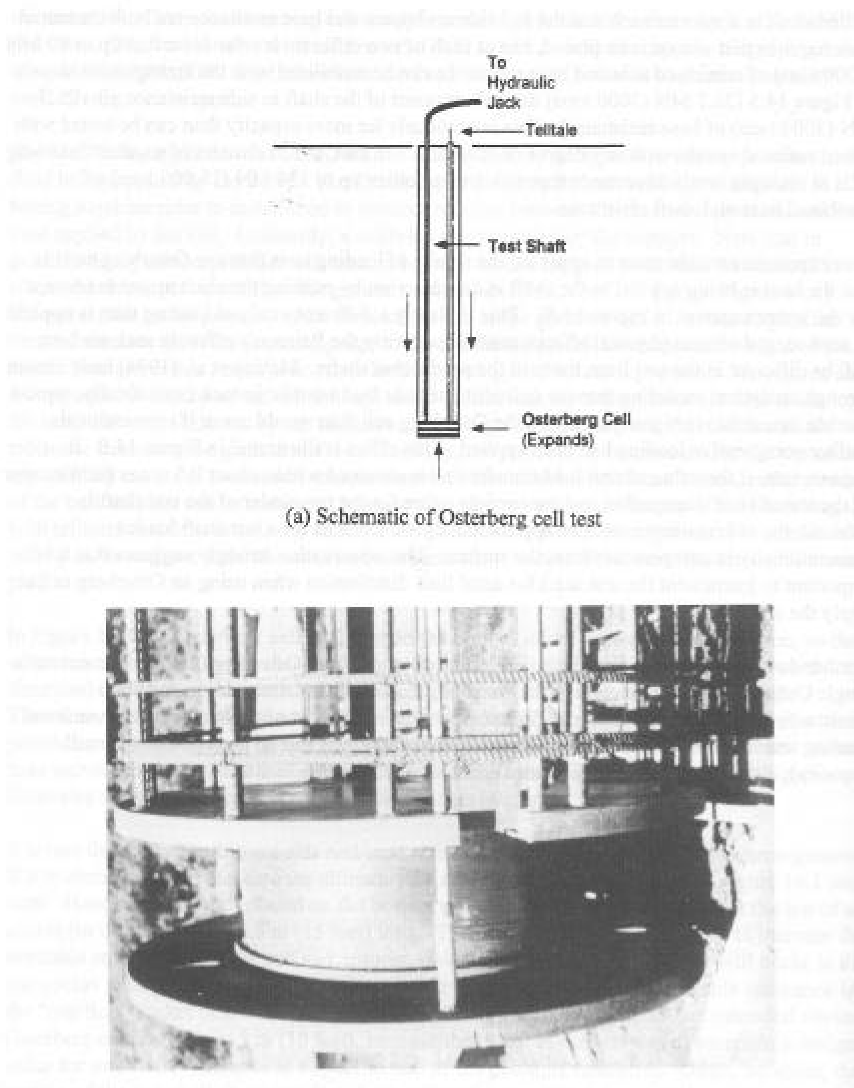


Figure 3.2. Osterberg Cell Test (from FHWA 1999)

3.2.3 CDOT Data Analysis of the O-Cell Load Test results

<u>Definitions of Tolerable Settlements and Ultimate Resistance</u>

The tolerable differential settlement between adjacent bents or abutments, recommended by AASHTO for bridge structures with continuous spans, is span/250. The T-REX requirements for settlements read as: "The contractor shall ensure that differential settlement shall not exceed 1/2 inch within a pier or abutment or span/400 between adjacent bents or abutments." The tighter criterion of span/400 recommended in the T-REX specifications is attributed to the critical locations of the T-REX structures over Interstate I-25. For a bridge with a very small span of 50 ft, the tolerable differential settlement is 1.5 in. According to the design criteria, the first criterion for differential settlement (0.5 in.) would dictate the design. Assuming conservatively that the percentage ratio between the tolerable differential settlement and total settlement is 75%, the tolerable settlement could be roughly estimated as 0.65" in. A tolerable settlement limit of 0.65" was selected in this study because it is common in the T-REX and Broadway structure to have the bridge column supported by more than one shaft. If single, large-diameter shafts were used to support bridge columns, settlement criterion larger than 0.65 in. might be used. The bridge structural engineers should be consulted for the appropriate tolerable settlements of each bridge structure. Then, the top-load settlement of that bridge structure could be used to determine the upper limits on the service (allowable) loads.

The proper selection of the definition for ultimate resistance values is controlled by three factors:

- Cost-effectiveness by using the highest possible resistance values. The tight FHWA definitions of ultimate resistance for cohesive IGM and rock (at 1 in. settlement) as presented in the previous chapter could lead to very conservative designs, as will be demonstrated in this study.
- Availability of load test data taken to large displacements. If a substantial number of load tests in which deflections were on the order of 5 percent of the base diameter, then that criterion might be substituted for the FHWA 1-inch (25-mm) deflection criterion as the definition for ultimate resistance values. It is important, however, that the same deflection criterion be used for interpreting all load tests.

- For shafts embedded in the harder claystone and sandstone bedrocks, ultimate unit base resistance and ultimate unit side resistance (for rough-sided socket only) should be defined based on displacement, not strength, in order to limit the shafts settlement at service loads. This is because the shaft ultimate "true" resistance may fully develop at very high settlements (see Chapter 2 and load test results for the Broadway shaft presented in Chapter 4).

Most of the load tests described in this report were carried out to a base settlement (~downward movement of O-Cell) of about 0.05D to 0.1 D for the soil-like claystone (I-225 and County Line sites) and 0.05D for the very hard claystone and sandstone (Franklin and Broadway). For the soil-like claystone at County Line and I-225 sites, the ultimate side and base resistance was mobilized to a large extent by the end of the test. This was not the case for the very hard claystone and sandstone at the Franklin and Broadway sites where the measured upward movement at the end of the test was small (0.1 to 0.4 inches). At this time, there is limited side resistance data for large side movement in the very hard Denver claystone and sandstone. For some bedrock formations, side resistance might be lessened past the peak resistance (referred to as brittle behavior). Turner et al. (1993) assumed for the Denver rock formation that the side resistance failure corresponded to a displacement of 0.4 in.

To address all issues presented above and consider the FHWA recommendations presented in the previous chapter, the adopted definitions of ultimate resistance in this study are:

- True base and side resistance values for the soil-like claystone to correspond to the full mobilization of the resistance in the plastic range. The values of f_{max} and q_{max} could be obtained through a conservative extrapolation of the plastic failure portion of the resistance-movement curves.
- For the very hard claystone and sandstone, q_{max} to correspond to displacement of 5% of the shaft diameter, but not to exceed 3 inches The 3 inches limit is suggested to limit excessive settlement of large diameter shafts.
- In the very hard claystone and sandstone, f_{max} to correspond to a displacement of 1% of the shaft diameter (0.42 inches for the Franklin and 0.54 inches for the Broadway test shafts), but

not exceed 0.6 inches. Once the ultimate side resistance was obtained, it was assumed to remain constant (level out) until a movement of 5% of the shaft diameter occurred.

These definitions of ultimate resistance may be adjusted in the future as more data are obtained to optimize the design methods.

Data Analysis:

The CDOT Research Office analyzed the raw data obtained from O-Cell load tests in order to obtain the most accurate load transfer curves for the weak rocks: settlement (w) versus base resistance (q) and settlement (or downward side movement, w) versus side resistance (f). A portion of the analysis described herein is new and the other portion is different from that performed by LOADTEST, Inc., leading in some cases to different final results from those reported by LOADTEST, Inc. (2002).

The movement of a given segment of the test socket is somewhere between the O-Cell upward movement and the top-of-shaft movement. At the Franklin site, the measured upward movement of the shaft at the end of the load test ranged from 0.154 inches at the O-Cell to 0.104 inches at the top of the shaft. The difference, due to compression of the shaft, was almost 33% (large) of the total measured upward movement of the O-Cell (0.154 inches). The average side movement of the shaft in the bedrock zone under consideration, w, was calculated and presented graphically against the average measured side resistance in that zone, f (Chapter 4).

There are many sources for errors in estimating the side resistance between O-Cell and Level 1 SG, Level 1 and Level 2 SGs, and Level 2 SG and the top of the shaft as described before. Estimated side resistance is dependent on E_c (Table 3.1), which is roughly estimated through the measured unconfined compressive strength of the shaft concrete at the time of the load test. The normal error associated with these values could be very large. There is a need in future tests to develop stress-strain relationships for the concrete and steel for a better estimation of E_c. In addition the distribution of side resistance could be nonlinear, especially immediately above the O-Cell. Findings of ongoing NCHRP project 21-08 suggest that the distribution of side resistance (interpreted from strain gages) is expected to be biased toward higher values nearest the O-Cell, which also was noted in this study. This is due to higher strains near the steel rebars, where the SGs are placed, than the average strains across the entire sections of the test shafts. For these reasons, it was deemed more accurate to estimate the average shaft side resistance along the entire shaft segment embedded in the competent rock, requiring no data from strain gages. This approach produced side resistance values smaller than those measured near the O-Cell. Because the resistance to working loads is provided mostly by means of side resistance, this approach was not only more accurate but also was more conservative than using side resistance values estimated with the strain gages. This approach for estimating side resistance was recommended by NCHRP project 21-08 for design purposes. For this research study, curves of average side resistance, f, versus average side movement, w, in the bedrock socket were extracted for each test shaft (see Chapter 4). In order to estimate the side resistance in the bedrock socket only, the small contribution of overburden to the overall side resistance measured in the O-Cell tests was neglected in the very hard claystone and sandstone at Franklin and Broadway shafts, and was roughly estimated in the soil-like claystone at I-225 and County Line shafts.

The maximum O-Cell downward load was resisted mostly by the shaft base resistance at County Line, I-225 and Franklin. At these sites, the base resistance, q, versus settlement, w, relation could be obtained directly from the test results. For the Broadway test shafts, the O-Cell downward load was resisted by both the end bearing at the tip of the shaft and by the side resistance of the shaft segment beneath the O-Cell (referred to as the reaction socket). The side resistance component of the reaction socket was significant in the Broadway shaft because the length of the reaction socket was large (6 ft). Extracting the base resistance versus settlement relation for the bedrock beneath the tip of the Broadway shafts required a method to estimate the contribution of the side resistance of the reaction socket to the shaft overall measured resistance beneath the O-Cell. Unfortunately, there were no side strain gages placed around the tip of the shaft to estimate the shaft side resistance of the reaction socket. Hence, there was much uncertainty in trying to reconstruct base resistance versus settlement curve for the Broadway test shaft. Side resistance vs. movement response beneath the O-Cell roughly could be estimated through two alternatives:

1. Use the side resistance vs. movement curve measured from O-Cell to Level 1 strain gages. This could be a reasonable assumption if the rock strength below and above the O-Cell is similar. However, this could also be questionable due to many sources of errors involved in estimating the side resistance between O-Cell and Level 1 strain gages as discussed before.
2. Use the side resistance vs. movement curve obtained for the entire bedrock socket above the O-Cell. This more accurate and conservative approach for estimating the side resistance was recommended by NCHRP project 21-08 for design purposes and will be utilized in this study to estimate the side resistance along the reaction socket. It should be noted that this approach could overestimate the base resistance.

The relatively large compression movement of the reaction socket of the Broadway test shaft was calculated and then subtracted from the downward movement of the O-Cell to estimate settlement at the tip of the shaft (w).

Applying the definitions for ultimate resistance as presented before, q_{max} and f_{max} were obtained and utilized to calculate the ultimate resistance load of the shaft Q_{max}, as $A_b q_{max}+A_s f_{max}$. The allowable design base and side resistance values and loads as determined from the O-Cell load test results can be determined as $q_{all}= q_{max}/2$, $f_{all}= f_{max}/2$, and $Q_{all}= Q_{max}/2$.

The side movement, w, was normalized by the diameter, D, and the resistance was normalized by ultimate resistance value to obtain the normalized load transfer curve for side resistance (w/D vs. f/f_{max}) and for the base resistance (w/D vs. q/q_{max}). Any future user can then construct foundation-specific resistance curves by plugging in the ultimate resistance value for each layer (as correlated to q_{ui}, N, etc.) and the diameter of the socket.

Construction of the Equivalent Top Load-Settlement Curve

Using the obtained q-w and f-w curves or the normalized versions of these curves, and the elastic stiffness of the shaft (E_c), load-settlement curves can be constructed accurately using several programs available in the market (e.g., SHAFT, APILE or SPILE). These programs account for the shaft compressibility using sophisticated load-transfer analyses. In constructing the equivalent top load-settlement curve, LOADTEST, Inc (2002) extrapolated the side resistance-movement curve for the Franklin and Broadway test shafts to large displacement values. A very conservative approach was adopted in this study by extrapolating the side resistance to a smaller displacement (0.01 D) that corresponds to the definition of ultimate side resistance and assuming this resistance to remain constant until displacement of 0.05 D that corresponds to the definition of ultimate base resistance.

The procedure adopted in this study to construct the load-settlement curve is similar to the procedure described in the LOADTEST, Inc. (2002) test reports and is based on the same assumptions. Assume initially that the shaft behaves as a rigid shaft and therefore w can be considered the shaft-head settlement. For arbitrary settlement, w, the corresponding side resistance and base resistance values are estimated from the obtained q vs. w and f vs. w (for the entire bedrock socket) curves or from the normalized version of these curves. Then, the shaft resistance load or the equivalent top load, Q, can be estimated as $A_b q+A_s f$. This should be repeated for several arbitrary values of w up to the ultimate resistance load or Q_{max}. The shaft head load-settlement curve should then be modified to take into account the compressibility of the shaft. The additional elastic compression generates more side movement leading to additional side resistance, requiring adjustment of the resistance load of the shaft. This elastic compression is only important in high-capacity drilled shafts embedded in hard claystone and sandstone bedrock (e.g., Franklin and Broadway shafts). The additional elastic compression movement of the shaft, δ, that corresponds to shaft head load, Q is estimated as follows. First, w is assumed to correspond to the settlement at the base of the shaft and the corresponding base resistance load (Q_b) can be estimated from the extracted q-w curve at the base of the shaft. Second, uniform side resistance distribution is assumed across the entire bedrock socket, estimated for any movement from the side resistance vs. movement curve obtained for the entire bedrock socket above the O-Cell. The shaft side resistance in the overburden is neglected. The average shaft axial load in the bedrock socket is $(Q+Q_b)/2$, and the shaft axial load in the overburden is Q. The additional elastic deformation can be calculated as $\delta =(Q/A_b E_c)\, L_o+ L/(A_b E_c)\, (Q+Q_b)/2$. Now a new point of $(Q, w+\delta)$ is obtained. According to LOADTEST, Inc (2002), this solution is adequate and slightly overonservative.

It would be of interest to get an approximate estimate for the shaft settlements, especially under working loads, as a function of the results of simple geotechnical tests. A head load versus settlement curve for rigid drilled shafts can be approximated as two linear segments with three points (0,0), (Q_d, 0.01D), and (Q_{max}, 0.05D). The ultimate shaft resistance load (Q_{max}) corresponds to settlement of 0.05D. At a settlement of 0.05 D, most of the base and side resistance for different types of weak rocks are mobilized. This study will define q_d, f_d, needed to calculate $Q_d = A_b q_d + A_s f_d$, respectively as the base resistance and side resistance that correspond to settlement equal to 0.01D. Then, the developed load-settlement curve can be adjusted for elastic deformation if necessary. The study will explore correlation relations between q_{max}, f_{max}, q_d , f_d and the test results of SPT, UC, and PM test data.

3.3 Geotechnical Investigation for Construction Projects

The four drilled shaft load test sites were explored by various geotechnical engineering consulting firms to determine the subsurface conditions (e.g., type and strength of the bedrock layers that would support the drilled shafts). The names of the engineering consulting firms that managed the geotechnical investigations at the load test sites and provided subsurface data and design recommendations for drilled shafts are listed in Section .1.3.

Test holes for all shafts were advanced with 4-inch inside diameter, solid stem and continuous flight power augers. Note that CDOT also drilled test holes to obtain SPT data using 7-inch diameter hollow-stem augers (ID= 3 ¼”). Samplers of different diameter were utilized to obtain blow counts per foot (bpf) and recover drive samples. The samplers were driven into the various strata with blows from a 140-pound hammer falling 30 inches, similar to ASTM D1586 (Standard Penetration Test, SPT). It is important to note that the blow counts obtained with different diameter samplers under the same energy provide different bpf.

At the County Line and I-225 sites, drive samples in the bedrock were collected at five-foot intervals using initially a ring barrel type sampler (ring diameter is 3 inches and height is 1 in) followed, at slightly greater depth, by a 1-3/8 inch SPT sampler. CDOT and the consultant SPT data were similar at the I-225 site. There was a substantial difference in SPT bpf values obtained by CDOT and those obtained by the geotechnical consultant for County Line (although both SPT sampler diameters were as specified by ASTM D1586). The consultant N values were approximately 50% greater than CDOT values. SPT values obtained by CDOT appear to correlate much better with the load test results and the Denver design practice described in Chapter 2. Possible reasons for these results are:

- The consultant performed each SPT immediately after recovering a drive sample with the ring sampler, which likely disturbed the weak rock. However, this disturbance should be minimal since the SPT N-value is derived from the last foot of an 18-inch drive, and this could explain the similar SPT data at I-225 obtained by CDOT and the consultant.
- Several studies have shown that significant variation in blow counts can occur between different drill rigs and operators.

The subsurface investigation at the Broadway site involved 18 test holes around all production piers, but no test holes were drilled at the exact location of the test shaft. Nine bedrock samples were recovered by driving the California sampler that also provided penetration resistance values. It is a common practice in Denver to utilize the California sampler (ID= 2 inches with brass ring liners) to obtain blow counts (considered by local practitioners to be equivalent to SPT) and to recover drive samples for the laboratory strength tests. It should be realized that the blow counts obtained from this could be different from those of the SPT because of different sampler diameters, yet equal applied energy during driving of both systems. In addition, this technique produces a very high degree of rock disturbance in the recovered samples. The following supports this statement. The measured unconfined compression strength of the California sampler specimens at the Broadway site ranged from 3.1 ksf to 29.2 ksf with an average of 17.4 ksf. When specimens were obtained through coring, much higher strength values were obtained by CDOT in this study (larger than 85 ksf and less than 312 ksf).

At the Franklin site, a test hole was located within 5 to 10 ft of the test shaft. The reported strength values of core specimens collected from that hole by the geotechnical consultant for the construction project were around 100 ksf. These values were larger than those obtained in the CDOT investigation. The difference could be attributed to different core sizes. Most of Colorado private geotechnical firms utilize the NX size cores (ID~ 2”), while CDOT utilizes the larger HQ size cores (ID~ 2.5”). The FHWA manual (1999) reported that the values of unconfined compressive strength for IGM is dependent on the size of the test core. The lower the RQD, the larger the diameter should be (RQD> 70% use 2” diameter cores, RQD<50%, use 3” to 4” diameter). Therefore, larger diameter cores are expected to produce more representative bedrock strength values, especially when the rock has been weathered. The HQ core string typically achieves better recovery and more weak rock is recovered, which also could explain the lower strength values of the CDOT procedure.

At I-225 and County Line, results of the subsurface geotechnical investigation were utilized to estimate the appropriate layout of the test shafts so that the O-Cell load test could yield the ultimate side and base resistance values. This unfortunately could not be done for the Franklin and Broadway shafts.

3.4 Geotechnical Investigation Administrated by the CDOT Research Office

The CDOT Research Office administrated extensive geotechnical investigations (relative to normal practice) at each of the four load test sites. Within 10 ft of the edge of each shaft (or pier), a CDOT drilling crew drilled at least three test holes to depths of 3 pier diameters below the base of the test sockets. They used a CME-55 truck mounted drill rig and a CME-55 track mounted drill rig. Subsurface geotechnical investigation methods at each test hole included auger drilling with standard penetration testing, coring with subsequent laboratory testing on recovered core specimens, and in-situ pressuremeter testing.

3.4.1 Standard Penetration Tests

The first test hole at each site was advanced with a 7-inch diameter hollow stem auger (ID= 3 ¼”). Standard penetration testing (SPT) with an automatic hammer, in accordance with ASTM D1586, was performed at 5-foot vertical intervals in the overburden and bedrock layers, or whenever a sand or friable sandstone layer was encountered, whichever occurred first. Disturbed soil and rock samples recovered with the split spoon sampler were identified and classified visually, in order to demonstrate correspondence with the samples taken in the second borehole.

This test hole remained open for a period of time sufficient to ascertain whether free ground water would seep into the borehole and, if so, the final piezometric level of the ground water was determined.

In the hard sandstone at the Broadway site, the maximum blow count of 50 was reached during the first penetration interval at less than 6 inches (e.g., 50/3”). In this case, the reported penetration depth (e.g., 3”) is higher than it should be due to seating problems early in the test.

3.4.2 Coring and Collection of Bedrock Samples

Cores from the weak rock were collected using two coring techniques:

1. Triple-walled core barrel with the wire line technique producing 5 ft core runs.
2. Double-walled core barrel with the wire line technique producing 5 ft core runs.

CDOT and Colorado geotechnical testing firms on occasion utilize the double-walled core barrel to collect cores for lab strength testing. The triple-walled core barrel technique has not been utilized on a routine basis by CDOT or by geotechnical consulting firms in Colorado. The triple-walled core barrel is similar to the double-walled barrel, but also has a plastic tube insert that fits inside the inner barrel of the double-walled sampler. The inner plastic tube is thinner than the inner barrel of the double-walled core barrel, allowing the sample to be retrieved without breaking apart along joints or seams. Hence, it was expected that the triple-walled core barrel technique would produce enhanced core recovery, less core breakage, and protected storage for cores during handling and transportation of the core samples to the lab. It was very difficult to extrude the core run from the plastic tube in the laboratory, and this resulted in disturbance to the recovered cores. In some cases, it was also difficult to see the core through the plastic tube because of moisture condensation in the annulus between the tube and cores. Difficulties in extruding the core runs in the field and in the lab could be attributed to wetting and swelling of the bedrock samples.

3.4.3 Unconfined Compression (UC) and Other Tests

The recovered samples were preserved, either in the plastic tubes for the triple-wall barrel and the continuous sampler, or wrapped with a plastic cover and stored in a core box for the double-walled barrel sampler. The core samples were transported to the lab as soon as possible to prevent cracking or expanding due to moisture changes. Bedrock RQD and core percentage recovery can provide a relative measure of the quality of both the bedrock layer and the sampling technique. Geocal, Inc. (2002) examined the core runs and performed the laboratory testing on bedrock core specimens. Percentage recovery and RQD for each core run were determined as soon as possible. The core runs were used in the lab to log the testing hole and identify the presence of joints or fractures in the rock mass.

The dimensions and weight of each specimen are used to obtain the unit weight of the specimen. The unconfined compressive (UC) strength of bedrock core specimens was determined according to ASTM D2166. This is the procedure for strength testing of soil specimens, not rock specimens as per ASTM D2938. In opinion of the research team, ASTM D2938 would not be appropriate for Denver's weak bedrock formation. The data obtained from the test program included the unconfined strength, initial Young's modulus of the stress-strain curve, and the strain at the ultimate strength. Natural moisture content was performed on the broken core specimens in accordance with ASTM standards, and the results were used to calculate dry density. Finally, Atterberg Limits and percent passing the No. 200 sieve tests were performed in accordance with ASTM D4318 and ASTM D1140.

O'Neill et. al. (1996) indicated that the most reliable test to measure the strength and stiffness properties of soft rock core specimens is the unconsolidated, undrained triaxial compression test (UU test). For this study, the UC test was selected because it is common, simple and low in cost, which allowed many tests to be performed within the budget limitations. When compared to UU test results, strength and stiffness values measured from UC will probably be conservative if the core sample is relatively undisturbed, and very conservative if the sample is disturbed. The confinement of the core will increase the strength and stiffness of the core, and its influence will be significant in soft geomaterial and much smaller, maybe negligible, for very hard bedrock.

In this and future similar investigations, the unconfined compressive strength should be selected as the "Standard" to obtain the UC strength (q_{ui}) and Young's modulus (E_i) of intact rock core using specimens having diameter of around 2.5 inches (HQ size cores) and length equal to twice the diameter. Since design factors are developed in this study using results of UC tests, it is inappropriate to use compressive strength data determined by other means.

Bedding planes, fractures, and slickensides were sometimes present within the relatively small UC test samples, indicative of similar features in the rock mass or resulting from the sampling process. If these factors seemed to influence significantly test results of some samples, the test results for the samples were ignored, as the objective was to obtain the strength of the intact bedrock mass.

3.4.4 Menard Pressuremeter Tests

The standard soil Menard pressuremeter shown in Figure 3.3 is a test hole deformation device used in this study to measure the in-situ (mass) stress-strain and strength characteristics of the weak rock formations. As a supplement to the primary and routine geotechnical investigation, 12 pressuremeter tests were conducted at the sites of the O-Cell load tests.

Test Hole Preparation

Careful test hole or test pocket preparation was important to the success and quality of the pressuremeter test results.

At the I-225, Franklin, and County Line sites, the test holes were initially advanced with 7-inch hollow stem augers (inside diameter of 3 ¼-inches). Then, the pressuremeter test pocket was prepared by drilling with a 3-inch solid flight auger about 5 feet below the base of the hollow stem auger hole. Dry drilling was used in this case because it was faster and kept the rock at its natural moisture content. Drilling with water or mud could soften the weak rocks, especially the weaker claystone as at County Line and I-225 sites.

At the Broadway site, wet drilling was utilized to prepare the test hole because the bedrock was very hard and located under the groundwater level. It was expected that the use of water would not significantly affect the very hard sandstone at that site. The test holes were advanced to the top of the test pocket using an N casing (ID= 3”) string. The test hole was not advanced using the hollow stem auger due to the risk of sand and cuttings flowing inside the auger, according to the CDOT drilling crew’s experience at the site. The pressuremeter test pocket was prepared by rotary drilling with a 2 15/16-inch tricone rock bit and a drilling fluid to a depth of about 5-feet below the casing. The hole for the test pocket was approximately 2 15/16 to 3 inches in diameter. The use of the tricone bit was preferable to the 3-inch solid flight auger because it resulted in a smaller diameter for the test pocket that was closer to the diameter of the PM probe (2.76-inches). Additionally, the rock was very hard for the solid flight auger to penetrate.

The diameters of all PM test pockets were within the tolerance on diameters (more than 1.03 probe diameters (2.84 inches) and less than 1.2 probe diameters (3.31 inches) recommended by ASTM D-4719.

Testing Procedure and Analysis

URS conducted the pressuremeter tests and interpreted the test results as described in this section (from URS, 2002).

The pressuremeter tests were performed using the Menard G-Am pressuremeter system. The apparatus (Figure 3.3) consists of a probe volume measurement and pressure-control instrument, and nitrogen gas-supply tank. The 70-mm or 2.76-inch (N size) diameter and 15-inch long cylindrical probes contain an expandable rubber membrane (measuring cell) and two contiguous, independently expandable guard cells. Prior to each test, the measurement system and the measurement cell were filled with water. During testing, the volume change induced in the measuring cell for each pressure applied was measured using a sight tube volumeter (Figure 3.3). Expansion of the measuring cell was controlled by applying gas pressure from the gas-supply tank to the fluid column through a pressure regulator system. The maximum pressure capacity of the pressuremeter system is 100 t/ft^2 (or 200 ksf).

Immediately after the pressuremeter test hole was prepared, the probe was lowered to the designated testing depth. The pressuremeter tests were performed in general accordance with the recommendations of ASTM D-4719. The test was performed by expanding the probe in equal applied pressure increments and measuring the change in volume of the measuring cell with time for each increment. In each test approximately 10 pressure/volume readings were obtained before the yield stress in the rock was reached. For a given applied pressure, volume readings were taken at 15, 30, and 60 seconds for the standard short-term test in an attempt to model undrained conditions. An unload-reload cycle was performed in each test, typically at the initiation of yield and plastic deformation (Figure 3.4). The pressure was increased in equal increments until the pressure (100 tsf or 200 ksf) or volume change (500 cc) capacity of the system was reached. The pressure and volume change measurements obtained during the pressuremeter tests were corrected for inertia of the probe, expansion of the measurement system, and the hydraulic pressure of the column of fluid between the instrument and the measuring cell.

The results of the pressuremeter tests are usually presented as a plot of corrected probe pressure change versus corrected volume change. A typical pressuremeter test plot is shown in Figure 3.4. The pressure-volume change curve is typically made up of three components: (1) the initial reloading portion -as the probe expands to the test hole walls, meets the test hole walls and restresses the soil back to its original in-situ (or at rest) condition; (2) the linear pseudo-elastic portion; and (3) the plastic portion where the soil exhibits substantial nonlinear deformation. An unload-reload cycle is also typically included in the pressuremeter test. The pressure at the initiation of plastic deformation is termed the yield pressure (P_f)*;* whereas, the asymptotical axis of the plastic deformation curve is interpreted as the limit pressure (P_L). The pressure at the inception of the pseudo-elastic response is generally interpreted as the in-situ horizontal total stress (P_o). Unavoidable test hole wall disturbance generally makes the interpretation of P_o less reliable than other parameters. The coefficient of earth pressure at rest (K_o) and the overconsolidation ratio (OCR) were calculated as:

$K_o = s'_{ho} / s'_{vo}$, $OCR = (P_f - u) / s'_{ho}$ 3.1

where s'_{ho}=P_o - u, is the in-situ horizontal effective stress; s'_{vo} is the vertical effective overburden stress equals to s_{vo} – u, where s_{vo} is the vertical total overburden stress and u is the pore water pressure, assumed hydrostatic below the groundwater table.

The URS report presents other geotechnical data, including initial modulus, unload modulus, reload modulus, undrained shear strength, cohesion and internal friction angle. According to FHWA (1989), the methods available to determine the cohesion and internal friction of geomaterials from PM results are not agreed on by the research community; therefore these parameters will not be reported in this document. Other parameters will be discussed in the following section.

CDOT Analysis of the Pressuremeter Test Results

The CDOT Research Office analyzed the raw data obtained from the pressuremeter tests by URS and determined the initial, unload, and reload modulus of the bedrock.

The corrected volume (V) of the probe was calculated as the initial volume of the probe (786 cc) plus the corrected volume change that corresponded to the corrected pressure P. The average radius of the probe (R), which corresponds to probe volume V, was also computed. The volume (V_o), average radius of the probe (R_o), and pressure (P_o) that correspond to the at-rest conditions were determined. Radial strains from the at-rest condition, ε_r, were determined as $(R- R_o)/R_o$. This study presents the result as P vs. ε_r, with the first reading shown as (P_o, 0). A typical plot of pressuremeter results from the current study is shown in Chapter 4, which is very similar to the plot of Figure 3.4.

Several elastic deformation moduli can be calculated from linear portions of the pressuremeter test curves, including an initial modulus *E*, a reload modulus E_r, and an unload modulus E_u. The initial modulus is determined from the slope of the "pseudo elastic" portion of the PM curve using:

$$E= (1+\nu)\, \Delta P/\Delta\varepsilon_r \quad \text{......} 3.2$$

Where ν is the Poisson's ratio assumed in this study to be 0.33, ΔP is the change in pressure, and $\Delta\varepsilon_r$ is change in radial strains. The reload (E_r) and unload (E_u) moduli are calculated using the preceding equation with the slope of the reload and unload portions of the cycled pressuremeter curves (see Figures 3.4 and 4.6). These elastic deformation moduli were determined to estimate the mass Young's modulus of the weak rock (E_m).

O'Neill et. al. (1996) indicated that the mass elastic modulus of the weak rock could be obtained from the pressuremeter test. This study will investigate the appropriate moduli (initial, reload, or unload) to represent the mass modulus of the weak rock (E_m). Unloading and reloading at the initiation of yield erases the effects of the hole disturbance, and a low strain response of the undisturbed geomaterial is measured. For this reason, research studies reported by URS (2002) and FHWA (1989) recommend the use of the reload modulus or to correct the initial modulus, in order to offset the test hole disturbance effect. Hughes Insitu Engineering, Inc., an international PM testing company, reported the average of reload and unload moduli as the most representative modulus for the geomaterial mass Young's modulus. But, for analysis of a test shaft, O'Neill et al. (1996) indicated that the initial modulus might better represent the properties in the geomaterial surrounding a drilled shaft, including any disturbance at the borehole-concrete interface, than the reload modulus.

Since the pressuremeter test is a total stress test, undrained shear strength, S_u, can be determined from the plastic failure portion of the pressuremeter curve. Two procedures will be investigated in this study. First, the relation proposed by Gibson and Anderson (1961) and utilized by URS (2002) in the test report as:

$(P_l - P_o)/S_u = 1 + \ln(E/(2S_u(1+v)))$ 3.3

and the second procedure is the method recommended by FHWA (1989) for soils as:

$S_u = 0.25\,(P_l - P_o)^{0.75}$, (with units of ksf) 3.4

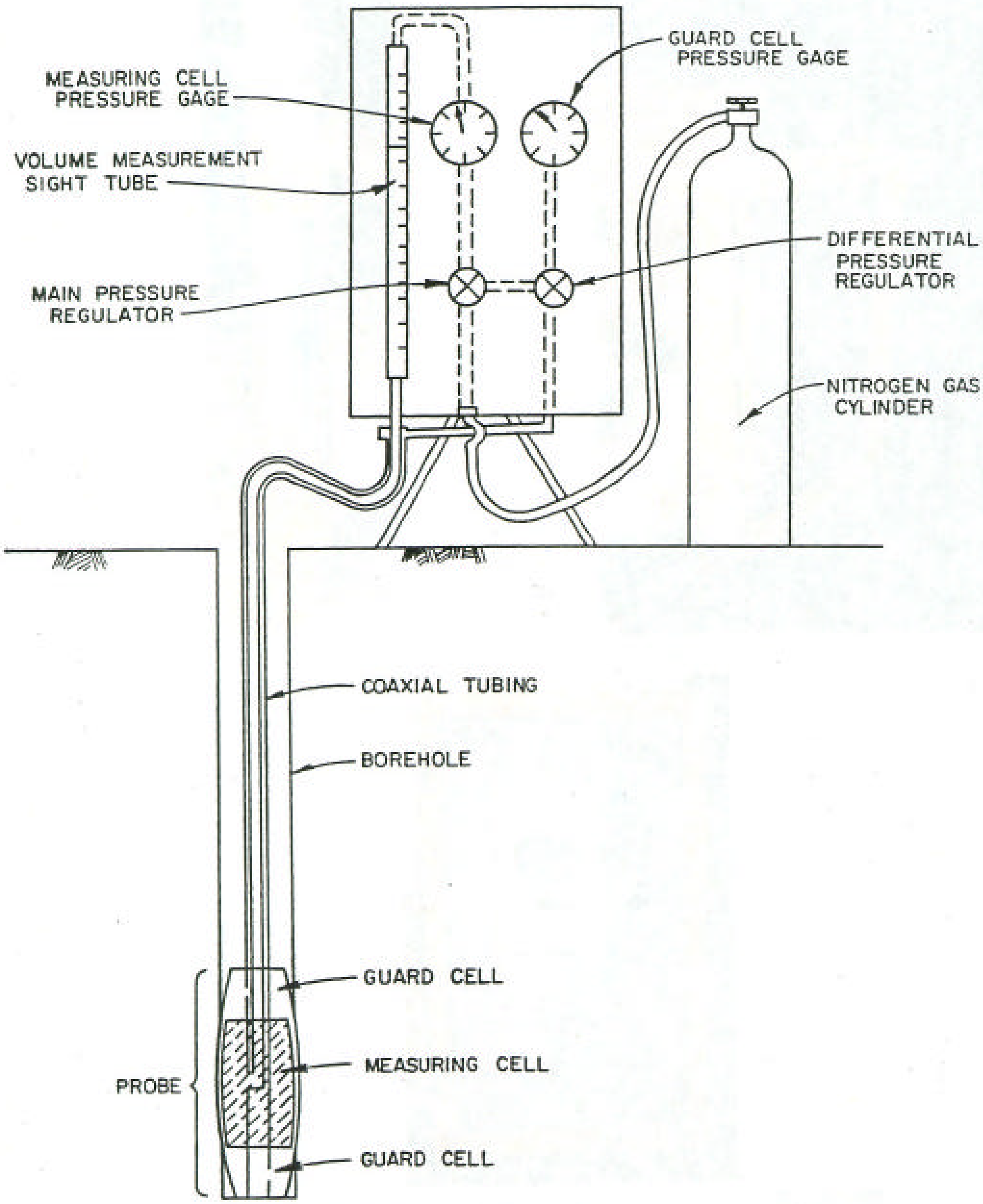


Figure 3.3. Apparatus Schematic of Pressuremeter Tests

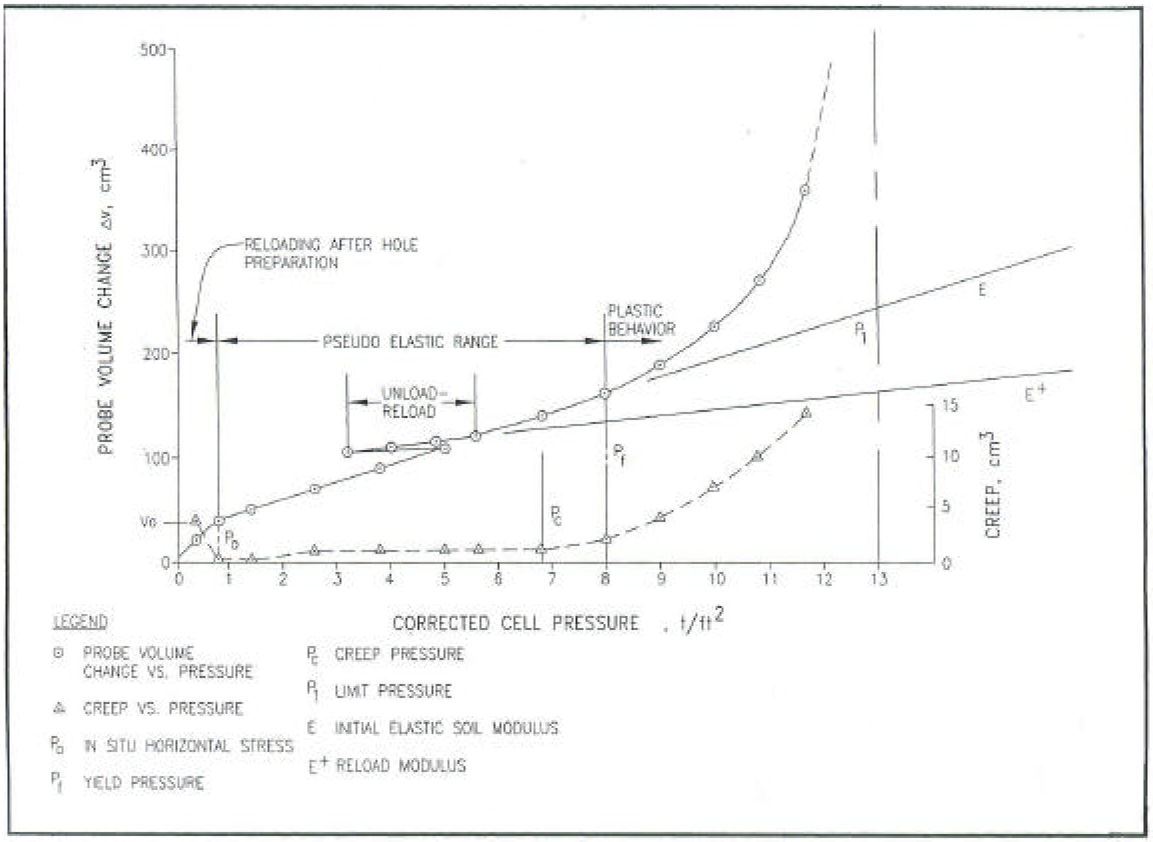


Figure 3.4. Typical Pressuremeter Plot (from URS, 2002)

4. TESTING RESULTS

The results of all tests (SPT, UC test, PMT, and load tests) are presented in this chapter. For the I-225 site (Site # 1), Figure 4.1 shows a section of the instrumented test shaft. The figure also shows a log of subsurface materials encountered, locations of core specimens tested in the lab (e.g., 1T-A), and locations of pressuremeter tests (using symbol “PMT”). Figure 4.1 shows most of the shaft layout information presented in Table 3.1 including elevation of the ground surface and depths to competent rock and GWL. Similar information for the County Line site (Site #2), Franklin (Site #3), and Broadway (Site # 4), are shown in Figures 4.2, 4.3, and 4.4, respectively.

4.1 Results of SPT and UC Tests

Figures 4.1, 4.2, 4.3, and 4.4 show the SPT test results, the extent and type of encountered geomaterial layers, and location of core specimens tested in the lab (e.g., 1T-A). The first number of the specimen identification number, shown in the logs and tables, indicates the site number. The first letter indicates the coring technique used to collect the specimen with T for triple-walled coring technique and D for double-walled coring technique. The second letter indicates the specimen number with A for the first specimen. Tables 4.1 to 4.4 list all the laboratory results for the core specimens. The test results listed in Tables 4.1 to 4.4 for each specimen include: identification number, depth to the specimen, natural moisture content, natural dry unit weight, % of sand and % fines (silt and clay), liquid limit, plastic limit, plasticity index, percentage recovery (R) and RQD of the core run from which the specimen was collected, a material description, and results of UC tests on *intact* core specimens including: unconfined compressive strength (q_{ui}), strain at peak strength, and initial modulus from the UC stress-strain plot (E_i). Because the rock at the Broadway site was too cemented, no gradation and Atterberg Limit tests were performed and sample descriptions are based solely on visual examination. Figure 4.5 shows typical plots of stress-strain curve obtained from UC tests on the, respectively, soil-like claystone at I-225, for the harder claystone at Franklin, and the very hard sandstone at Broadway.

The County Line and I-225 bedrock can be classified as very stiff clays according to AASHTO (1998) and CFEM (1992) manuals, and at the boundary area between stiff clays and cohesive intermediate geomaterial (IGM) according to the FHWA (1999) manual. Therefore, design methods for stiff clays are appropriate for these weathered rocks, requiring no RQD information (as discussed in Chapter 5). Indeed, no reliable cores could be obtained at the I-225 and County Line sites, indicating that any measured RQD values for these geomaterials are meaningless or questionable. Because bedrock at I-225 and County Line are described and will be analyzed as "soil-like claystone," rock terms R and RQD are not appropriate for these materials and they are not presented in Tables 4.1 and 4.2. This is contrary to the FHWA manual (1999) where it is recommended to obtain RQD for the cohesive IGM.

In the following, the soil/rock layers are described following CDOT Description Standards presented in Chapter 2.

4.1.1. I-225 Site

As shown in Figure 4.1, 10 feet of stiff sandy clay was found to overlie 2.5 ft of weathered and firm claystone. For the stiff sandy clay layer, the average SPT N-value was 14 bpf and the measured UC strength on one specimen from that layer was 1.9 ksf. No lab test information was obtained for the weathered claystone layer. All test holes were dry during drilling to a depth of 39 ft. After allowing sufficient time for water to seep into the test boring hole, the GW level was measured at depth of 15.5 ft. The competent claystone started at a depth of about 12.5 ft.

The encountered bedrock layers at the I-225 site (see Figure 4.1 and Table 4.1) are:

1. Layer 1 from depths of 12.5 to 22.5 ft: Firm to medium hard brown claystone bedrock. The SPT test results were nearly uniform, with an average of 32 bpf. No reliable core specimens could be obtained in this layer for lab strength tests.
2. Layer 2 from depths of 22.5 to 30.5 ft: The bedrock consisted of medium hard to hard brown claystone bedrock with sandstone seams. The SPT test results were almost uniform, with an average of 58 bpf. A strength of 13.1 ksf was measured in one UC test in this layer.

3. Layer 3 from depths of 30.5 to 32.5 ft: A fine-grained sandstone lense. The measured UC test strength from a specimen in that very hard layer was 85.9 ksf.
4. Layer 4 from depths of 32.5 to 40 ft: Hard brown claystone bedrock. The specimen from this zone broke in the UC test at a relatively low value of 4.5 ksf. The breakage of this core specimen was along an existing vertical fracture, so it was considered non-representative of intact rock strength and therefore was excluded from further consideration.

4.1.2 County Line Site

All test holes were dry during drilling. These holes remained dry when checked several days after drilling (different than I-225 site). Medium hard, brown silty and very weak sandstone bedrock was encountered from the ground surface to a depth of about 8 feet. The sandstone was predominantly fine to medium grained and non-cemented. Some iron staining was observed. The measured SPT N value in this layer was 30 bpf. This layer was considered as an overburden soil layer in the analysis as recommended in the FHWA (1999) description system described in the previous chapter.

The encountered uniform bedrock layers at the County Line site (see Figure 4.2 and Table 4.2) are:

1. Layer 1 from depths of 8 ft to 22 ft (toe of the shaft): Medium hard claystone bedrock layer with colors ranging from olive to light gray. Some iron staining was noted. SPT test results ranged from 33 to 41 bpf, with an average value of 38 bpf. The results of three UC tests in this layer were 2.2 ksf (initial modulus of 164), 10.4 ksf (E_i= 547 ksf), and 5.4 ksf (E_i=278 ksf), with an average of 6 ksf.
2. Layer 2 from depths of 22 ft to the maximum depth explored of 35 feet: Hard olive to gray colored claystone bedrock layer was. The average measured SPT value in that layer was 61 bpf. An average UC strength of 16.8 ksf was measured in that layer through two UC tests.

4.1.3 Franklin Site

At a depth of 4 ft, the GW was encountered. 4.5 feet of sand and clayey sand soils overlie the very hard bedrock. The testing results indicated no clear mark for change in the bedrock strength at any level, but that weaker zones could be found at any given depth between harder rock layers. The UC, SPT, and PM test results also suggest a general decrease in the rock strength with depth. Therefore, it was decided to assume a uniform rock layer within the test socket and a second uniform rock layer beneath the test socket. The extent, description, and test results for these two bedrock layers (see Figures 4.3 and Table 4.3) are:

1. Layer 1 from depths of 4.5 ft to 25.3 ft (toe of the test shaft): Very hard, thinly bedded, sandy to very sandy claystone layer with very clayey sandstone interbeds. Bedrock above a depth of 16.5 ft was brown to olive-brown with occasional calcareous or iron-oxide stained joints. Bedrock below a depth of 16.5 ft was dark gray to bluish gray in color (Denver blue shale). In this layer, SPT testing was terminated in the second interval with 50 blows per 4 inches of penetration (or 50/4”, corresponding roughly to an SPT N value of 150). R and RQD were approximately 100% and 80%. The peak strength measured from five UC tests ranged from 38.7 ksf (E_i= 4384 ksf) to 87.2 ksf (E_i= 9622 ksf), with an average strength of 64 ksf (E_i=7485 ksf).
2. Layer 2, below the toe of the test shaft (depth of 25.3 ft): Very hard, thinly bedded, dark gray, and very sandy claystone. In this layer, SPT was terminated in the second interval with an N value of 50/5” (N in bpf is roughly estimated as 120 bpf). R and RQD were approximately 100% and 80%. The peak strength measured from one UC test was 35.3 ksf (E_i= 5204 ksf).

Note that a core specimen with a UC strength of 27.2 ksf was not considered in the analysis. The description for that core specimen is blocky (Table 4.3), which implies a high degree of weathering. Note also that generally the RQD is relatively high beginning at a depth of 20 feet with the exception of the RQD at the depth this blocky specimen was obtained. It has an RQD of 65 compared to 88-95 for the other bedrock below 20 feet. It seems that there is (for whatever reason) a zone of relatively weak bedrock where the specimen was obtained, but it is not representative of the rock strength around the test shaft.

Note that the reported strength values of core specimens collected at the Franklin test site by the geotechnical consultant for the construction project were around 100 ksf, larger than those obtained in the CDOT geotechnical investigation.

4.1.4 Broadway Site

This site is close to the Platte River with the GWL at a depth of 17 ft. At this site, 17 feet of man placed sandy/gravelly dark colored fill (9 ft) and a mixture of fine to coarse-grained sand and gravel (8 ft) overlie very hard bedrock. For analysis purposes, the rock layers are divided into three bedrock layers (see Figure 4.4 and Table 4.4):

1. Layer 1 from depths of 17 ft to 35 ft: Light brown claystone (17 ft to 20 ft), and clayey to very clayey, fine to medium grained, well-cemented sandstone (20 ft to 35 ft). This layer has occasional siltstone seams and ranges in color from light gray to dark gray to bluish gray. The maximum SPT blow counts of 50 was achieved during the second interval with penetration less than 3 inches (N value 50/3”). Measured average R and RQD were 80% and 75%. The average strength obtained from two UC tests was 97 ksf (E_i= 10663).
2. Layer 2 from depths of 35 ft to approximately the toe of the test shaft (47.1 ft): Blue, clayey to very clayey, sandstone bedrock layer, with lenses of claystone and siltstone, that was much harder than the first rock layer. In this layer, the SPT test was terminated during the first interval with penetration of more than 100 blows required to penetrate 5.5” (N-value 100/5.5”). Because the SPT was terminated in the first interval, these are not the true SPT results for the very hard rock, where the first 6 inches of the drive would normally be disregarded. SPT results where this occurred are noted on the log with an asterisk. R and RQD are approximately 100% and 85%, respectively. The measured UC strength from 6 tests was very consistent (narrow range from 272 to 312 ksf) with an average of 293 ksf (E_i= 48609 ksf).

3. Layer 3 below the toe of the test shaft (depth of 47.1 ft): Very hard blue, clayey to very clayey, sandstone bedrock layer (harder than Layer I but not as hard as Layer 2). The SPT test was terminated during the first interval (N-Value 83/6”). R and RQD were approximately 100% and 75%, respectively. From two UC tests, the measured strength ranged from 194.8 ksf to 242.7 ksf, with an average of 219 ksf and an average initial modulus of 29660 ksf.

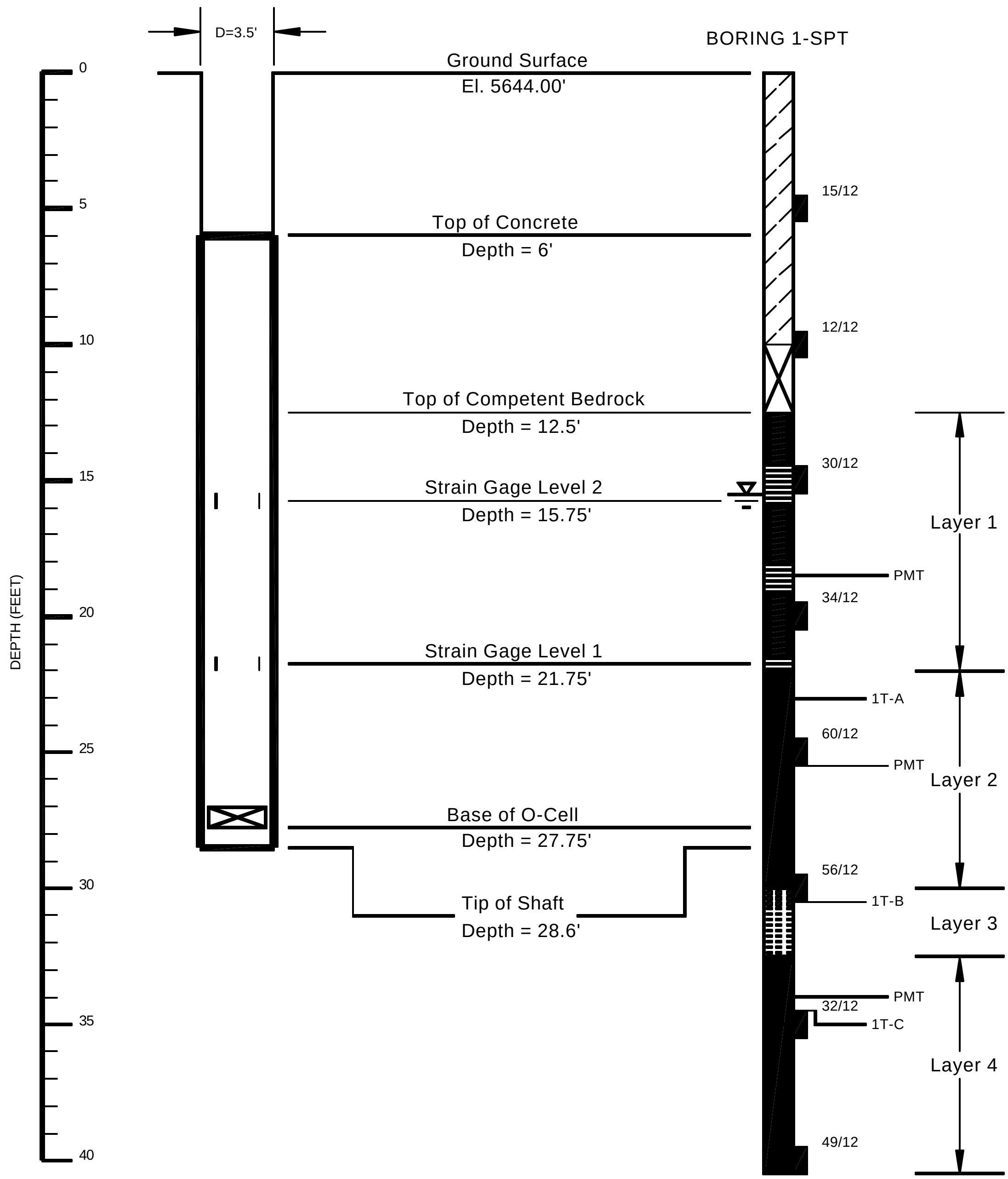


Figure 4.1. Instrumented Test Shaft Schematic and Boring Logs for I-225 Site

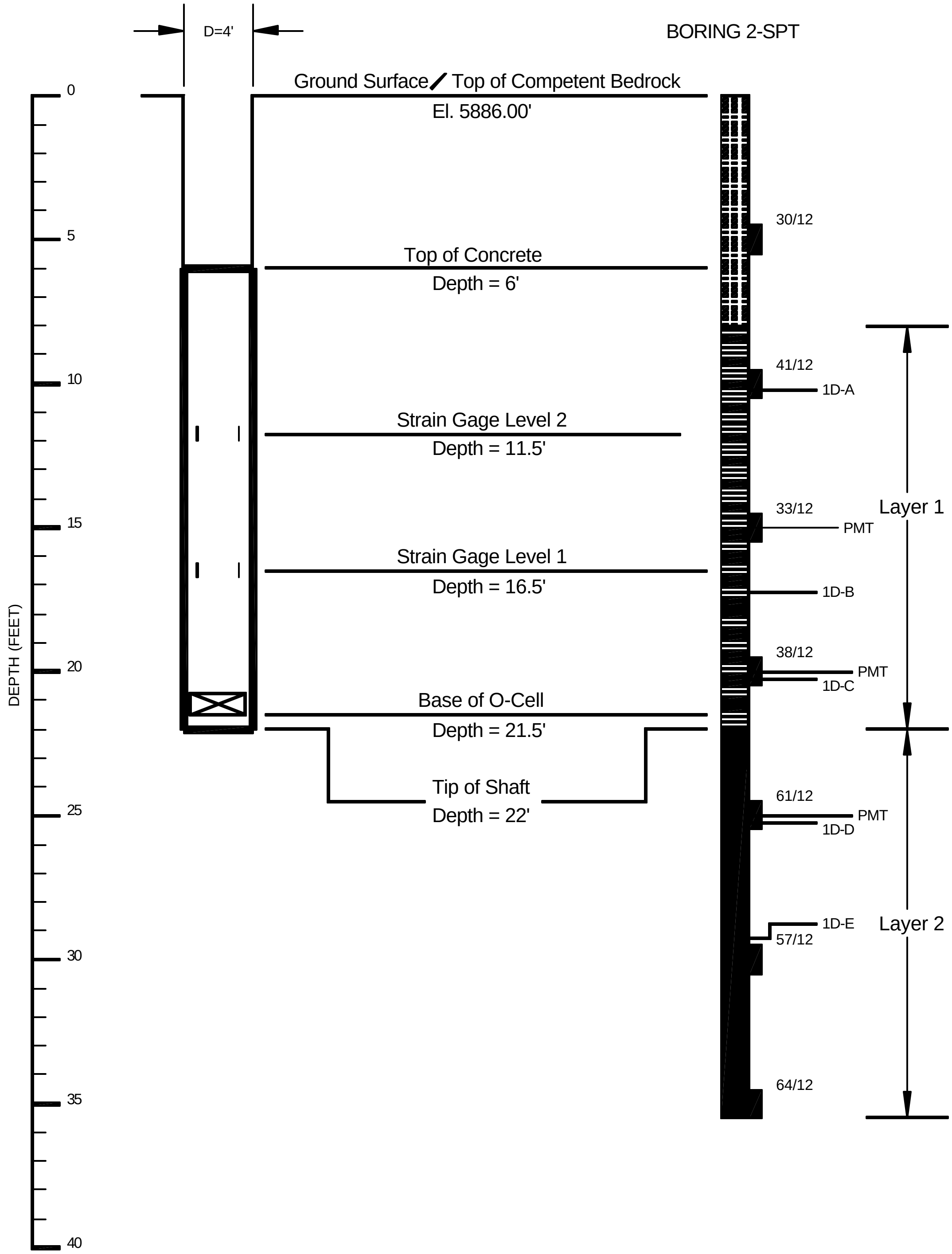


Figure 4.2. Instrumented Test Shaft Schematic and Boring Logs for County Line Site

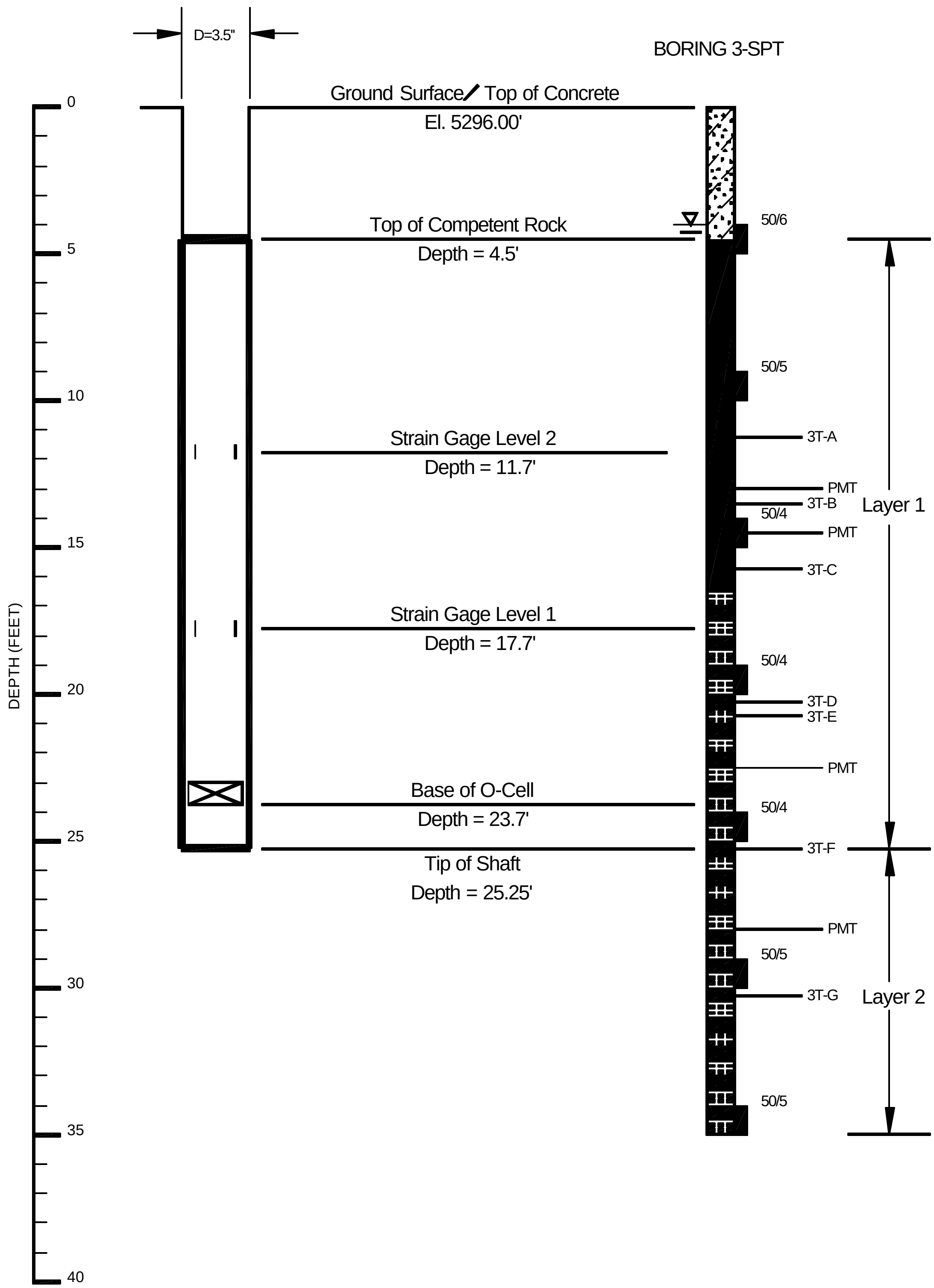


Figure 4.3. Instrumented Test Shaft Schematic and Boring Logs for Franklin Site

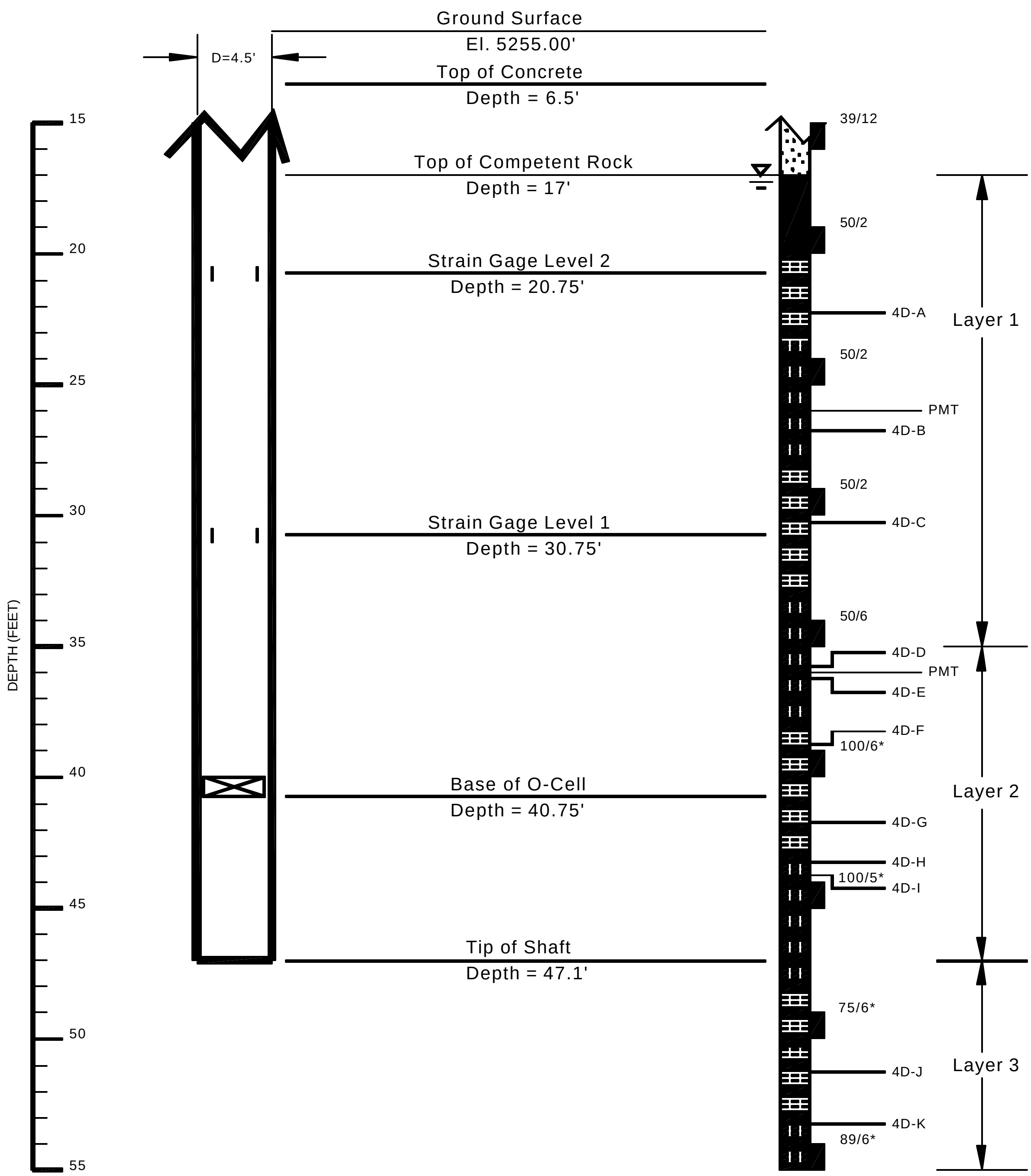


Figure 4.4. Instrumented Test Shaft Schematic and Boring Logs for Broadway Site

Table 4.1. Laboratory Data Summary at the I-225 Site

Specimen Identification	1T-A	1T-B	1T-C
Depth (ft)	23.25	30.75	34.75
Natural Moisture Content (%)	17.9	18.4	23.9
Natural Dry Unit Weight (pcf)	112	113	97
Sand (%)	-	-	-
Silt/Clay (% -200)	76	15	88
Liquid Limit	30	-	48
Plastic Limit	21	-	22
Plasticity Index	9	NP	26
UC Strength (ksf)	13.1	85.9	4.5*
Strain at Peak UC Strength (%)	1.2	1.1	3.5
Initial Modulus, E (ksf)	1056	7174	181
Sample Description	CLAYSTONE, sandy, fractured (random), oxide stained, moist, brown	SANDSTONE, clayey, fine to med. grained, moderately cemented, moist, reddish brown.	CLAYSTONE, slightly sandy, fractured (primarily vertical), slightly blocky, oxide stained, moist, brown.

* Broke along a fracture, was not considered in the analysis.

TABLE 4.2. LABORATORY DATA SUMMARY AT THE COUNTY LINE ROAD SITE

Specimen Identification	2D-A	2D-B	2D-C	2D-D	2D-E
Depth (ft)	10.25	17.25	20.25	25.25	29.25
Natural Moisture Content (%)	28.4	23.7	20.0	17.2	17.4
Natural Dry Unit Weight (pcf)	103	103	112	113	109
Sand (%)	-	-	-	-	-
Silt/Clay (% - 200)	82	81	80	78	88
Liquid Limit	53	54	61	55	53
Plastic Limit	22	21	18	18	20
Plasticity Index	31	33	43	37	37
UC Strength (ksf)	2.2	10.4	5.4	18.9	14.8
Strain at Peak UC Strength (%)	3.7	2.8	6.0	3.5	8.0
Initial Modulus (ksf)	164	547	278	1280	618
Sample Description	CLAYSTONE, sandy, slightly blocky, calcareous, very moist, olive.	CLAYSTONE, sandy, slightly blocky, iron stained, moist, olive.	CLAYSTONE, sandy, moist, olive-orange-brown.	CLAYSTONE, sandy, blocky, iron stained, moist, olive.	CLAYSTONE, slightly sandy, iron stained, moist, olive and brown.

TABLE 4.3. LABORATORY DATA SUMMARY AT THE FRANKLIN SITE

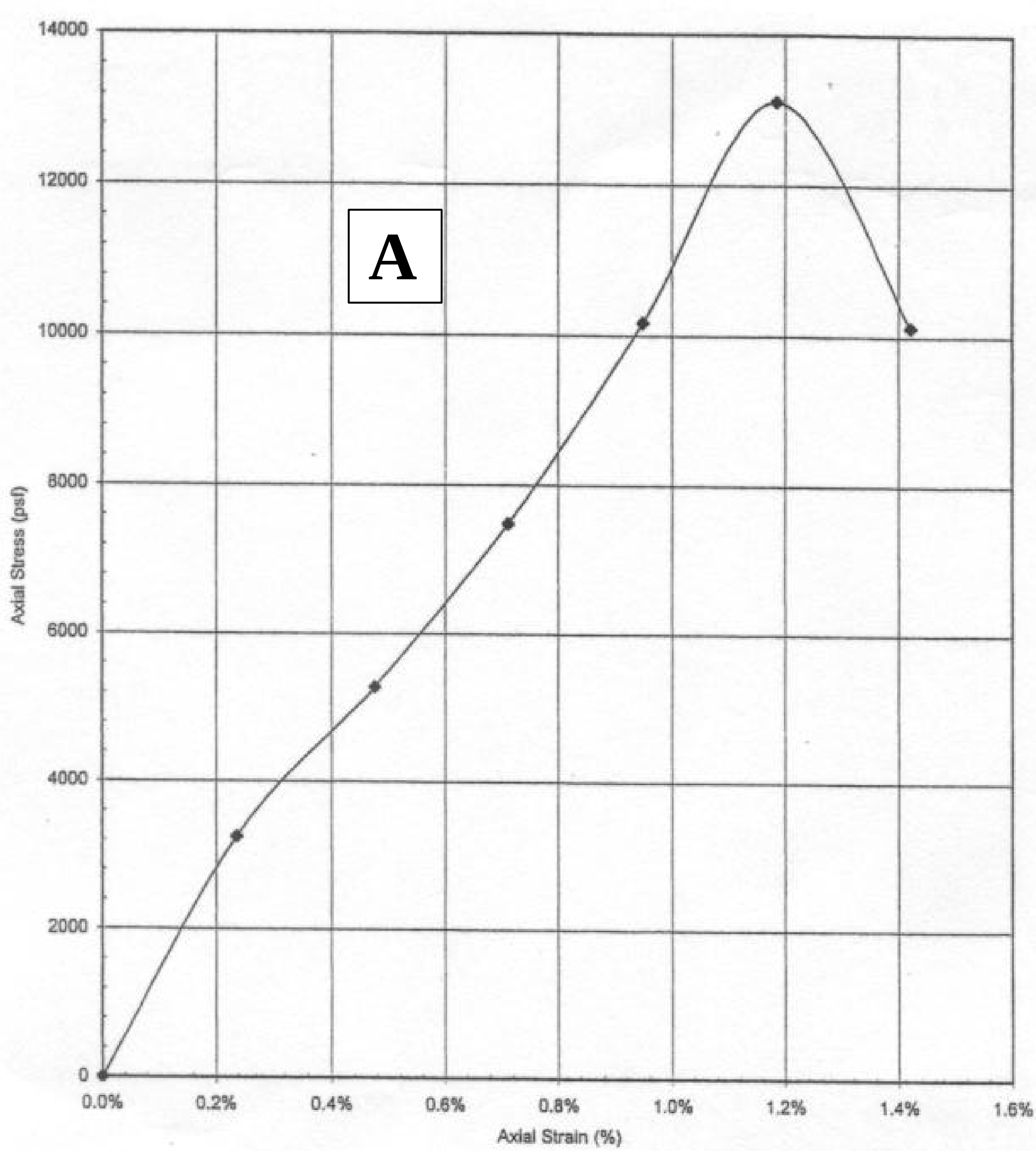
Specimen Identification	3T-A	3T -B	3T -C	3T -D
Depth (ft)	11.3	13.4	15.75	20.25
Natural Moisture Content (%)	18.5	23.8	20.8	16.7
Natural Dry Unit Weight (pcf)	111	100	108	111
Sand (%)	51	-	-	-
Silt/Clay (% -200)	49	80	66	-
Liquid Limit	43	58	50	-
Plastic Limit	20	27	17	
Plasticity Index	23	31	33	-
UC Strength (ksf)	70.7	87.2	38.7	54.5
Strain at Peak UC Strength (%)	1.5	1.8	0.9	0.9
Initial Modulus (ksf)	9622	7593	5206	9333
Recovery (5 ft run where sample obtained) %	85	95	95	100
RQD (5 ft run where sample obtained) %	35	60	60	95
Sample Description	SANDSTONE, fine to medium grained, lightly cemented, moist, brown.	CLAYSTONE, sandy, oxide stains, moist, brown.	CLAYSTONE, very sandy, oxide stains, moist, brown.	CLAYSTONE, sandy, dark gray, moist.

Specimen Identification	3T-E	3T -F	3T – G	
Depth (ft)	20.75	25.25	30.25	
Natural Moisture Content (%)	19.9	18.6	15.6	
Natural Dry Unit Weight (pcf)	108	110	117	
Sand (%)	-	-	-	
Silt/Clay (% -200)	69	82	54	
Liquid Limit	50	58	39	
Plastic Limit	17	17	17	
Plasticity Index	35	41	22	
UC Strength (ksf)	68.4	27.2	35.3	
Strain at Peak UC Strength (%)	1.2	0.9	1.0	
Initial Modulus (ksf)	8770	4383	5204	
Recovery (5 ft run where sample obtained) %	100	100	100	
RQD (5 ft run where sample obtained) %	95	65	88	
Sample Description	CLAYSTONE, sandy, dark gray, moist.	CLAYSTONE, sandy, blocky, moist, dark brown to dark gray	CLAYSTONE, very sandy, moist, dark gray.	

TABLE 4.4. LABORATORY DATA SUMMARY AT THE BROADWAY SITE

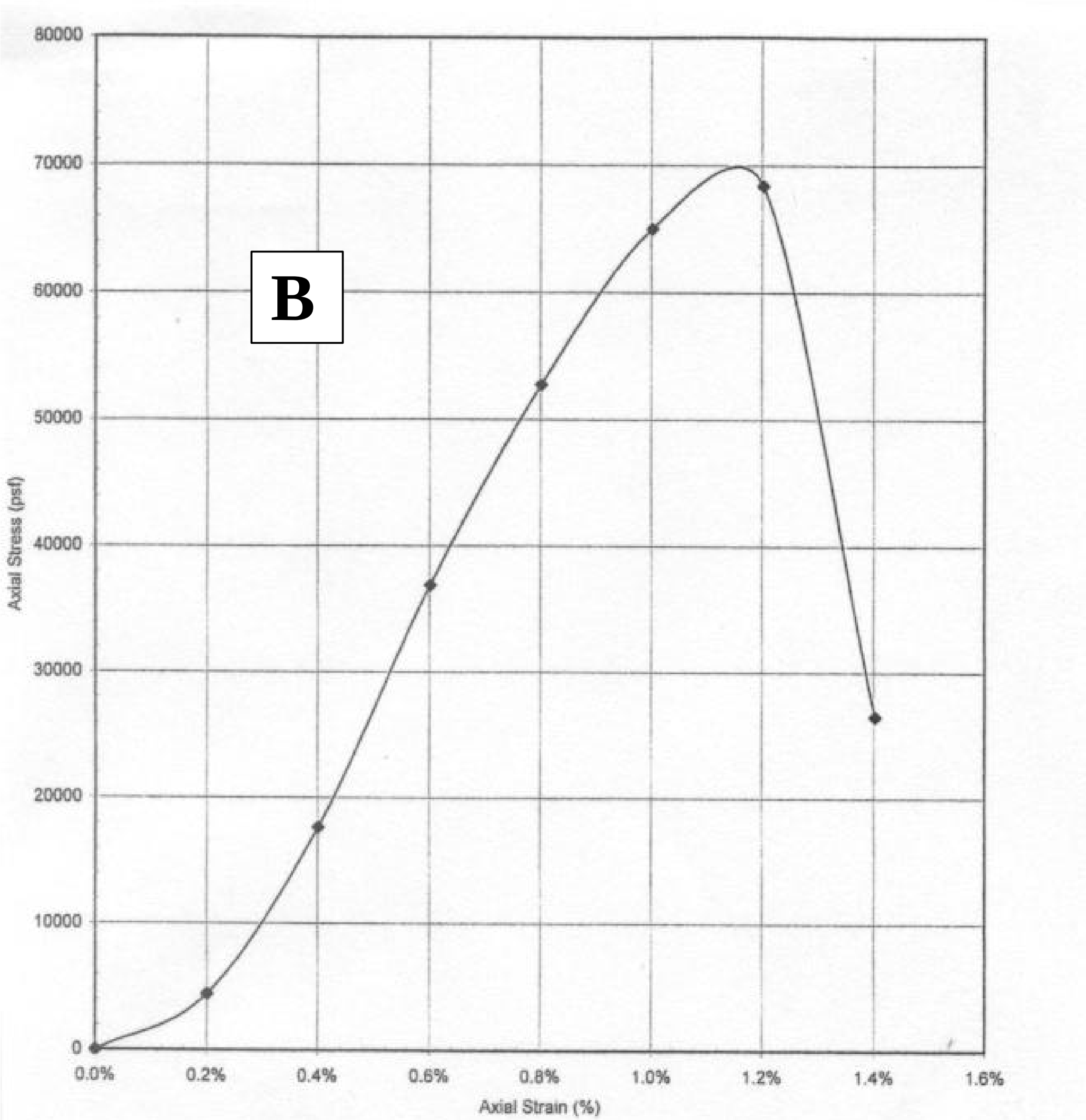
Specimen I.D.	4D-A	4D-C	4D-D	4D-E	4D-F
Depth (ft)	22.25	30.25	35.75	36.25	38.75
Natural M.C. (%)	19.3	17.3	16.1	12.6	14.1
Natural Dry Unit Weight (pcf)	114	113	118	122	121
Sand (%)	-	-	-	-	-
Silt/Clay (%)	16	-	-	-	-
Liquid Limit	-	43	-	-	-
Plastic Limit	NP	36	-	-	-
Plasticity Index	NP	7	-	-	-
UC Strength (ksf)	109.4	85.1	295.4	272.9	294.9
Strain at Peak UC Strength (%)	1.2	1.2	1.1	1.1	1.0
Initial Modulus (ksf)	10322	11003	47871	40274	56105
Recovery (5 ft run where sample obtained), %	20	78	100	100	100
RQD (5 ft run where sample obtained) %	12	73	57	57	93
Sample Description	SANDSTONE, silty, moist, dark gray.	SILTSTONE, slightly sandy, moist, gray.	SANDSTONE, fine to medium grained, very clayey, moist, gray to dark gray.		

Specimen I.D.	4D-G	4D-H	4D-I	4D-J	4D-K
Depth (ft)	41.75	43.25	43.75	51.25	53.25
Natural M.C. (%)	12.6	14.8	13.3	11.1	18.3
Natural Dry Unit Weight (pcf)	129	119	119	121	111
Sand (%)	-	-	-	-	-
Silt/Clay (%)	-	-	-	-	-
Liquid Limit	-	-	-	-	-
Plastic Limit	-	-	-	-	-
Plasticity Index	-	-	-	-	-
UC Strength (ksf)	295.2	286.7	312.2	242.7	194.8
Strain at Peak UC Strength (%)	0.9	1.1	1.4	1.4	1.4
Initial Modulus, E (ksf)	58638	40547	48220	36494	22820
Recovery (5 ft run where sample obtained) %	100	100	100	100	100
RQD (5 ft run where sample obtained) %	93	75	75	62	80
Sample Description	SANDSTONE, fine to medium grained, moist, gray.				CLAYSTONE moist, dark brown.



Boring No. 2-HQ
Depth Interval (feet) 23.0-23.5
Sample Description CLAYSTONE

Dry Density (pcf) 112
Moisture Content (%) 17.9



Boring No. 3-HQ
Depth Interval (feet) 24.0-24.5
Sample Description CLAYSTONE

Dry Density (pcf) 108
Moisture Content (%) 19.9

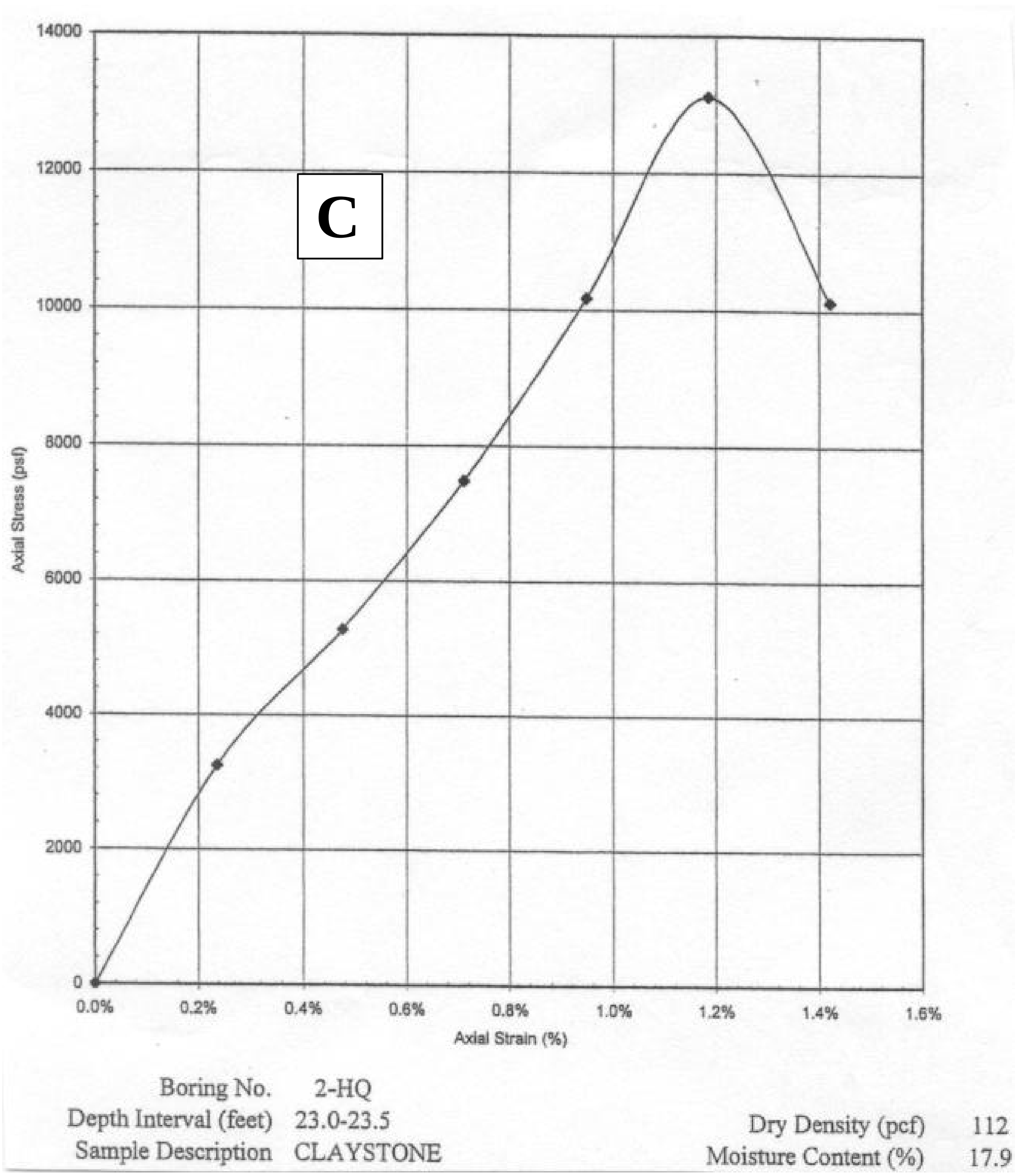


Figure 4.5. Typical Plots of Stress-Strain Curve Obtained from UC Tests for the: A) Soil-Like Claystone at I-225 Site, B) Very Hard Sandy Claystone at Franklin Site, and C) Very Hard Clayey Sandstone at Broadway Site.

4.2. Results of Pressuremeter Tests

Plots of the pressuremeter test results, in terms of radial strain from the at-rest condition vs. corrected pressure for all sites are shown in Figures 4.6, 4.7, and 4.8. The depth and number of the rock layer (as described in the previous section) where each of these PM test was performed are listed in Table 4.5. Geotechnical parameters interpreted from the pressuremeter test curves are also summarized in Table 4.5, including at-rest lateral earth pressure, P_o, at-rest coefficient of lateral pressure, K_o, yield pressure, P_f, overconsolidation ratio, OCR, initial modulus, E, unload modulus, E_u, reload modulus, E_r, and limit pressure, P_l. The values for the undrained shear strength, S_u, estimated through Eqs. 3.3 and 3.4 are also listed in Table 4.5.

For the I-225, County Line and Franklin tests, the linear elastic and most of the plastic portions of the PM test curves were well-developed up to the pressure or volume change limits of the system. The limit pressures for these tests were interpreted based on data at the end of the test or through an extrapolation of the plastic failure portion of the test curves. It is interesting to note that the first and second PM test results at the County Line site, performed in the same uniform bedrock layer, matched each other. Results of two PM tests in a layer at the Franklin site were also very similar. This supports the repeatability features of the PM tests.

For the 1st PM tests performed at the Broadway site (depth of 26 ft, Layer 1), the yield pressure was detected toward the end of the PM tests (Figure 4.8). This allowed rough extrapolation to estimate the limit pressure. The unloading curve at the end of the test could not be obtained because the membrane burst at the end of the test. The second PM test was performed on bedrock layer 2 (depth of 36 ft, in the hardest rock layer investigated in this study with UC strength of 293 ksf). It was discontinued because the membrane burst before reaching the yield strength. Only the initial modulus was obtained from this test, as presented in Table 4.5, but no graphical results are presented for this test. The limit pressure for this test was roughly estimated using the measured lab UC strength at depths of 26 ft and 36 ft.

During the unload-reload cycle for all PM tests, the measured unload modulus was extremely high, especially at the Broadway site (Table 4.5). Upon unloading at the end of the PM test, unload modulus from the first increment was also large, but in subsequent unloading increments, there were greater volume changes, leading to smaller unload moduli values. In addition, in some PM tests in this study, the unloading and reloading cycle was not performed at the end of the elastic range. The PM tester also suspected that the high pressures utilized in the Broadway and Franklin sites could cause volume changes within the two measuring cells and therefore contaminate the results. All these problems could lead to errors with the measured reload and unload modulus. In order to avoid these problems in the future, the following recommendations are made:

- The unloading phase in the PM test should be performed along several small pressure increments (not one large increment) as performed during the loading phase;
- Collect the PM test data through a data acquisition system;
- Perform more than one unload-reload cycle. One cycle must be performed at the end of the elastic range; and
- Use special PM test system for the very hard rock with UC strength larger 100 ksf as recommended in Chapter 3.

Table 4.5. Extracted Geotechnical Parameters from Results of Pressuremeter Tests

Depth (ft)	P_o (ksf)	K_o	P_f (ksf)	OCR	E (ksf)	E_u (ksf)	E_r (ksf)	P_l (ksf)	S_u (ksf) Eq. 3.4	S_u (ksf) Eq. 3.3
I-225 Site										
18.5, layer 1	3	1.1	19.2	6.5	970	3600	2400	39	3.7	7.3
25.5, layer 2	5.6	1.6	35.4	6.8	2550	9100	6000	76	6	13.2
34, layer 4	4.4	0.9	56	16	1900	6500	6500	112	8.4	14.3
County Line Site										
15, layer 1	3	1.4	23	7.6	1900	10000	3600	54	4.8	9.6
20, layer 1	3.2	1.1	23	7.1	1700	4600	3000	54	4.8	9.6
25, layer 2	7.5	2.2	35.4	4.9	3200	7200	6000	104	7.7	18.8
Franklin Site										
13, layer 1	10.8	5.4	80	8	14900	23000	12000	220	13.8	35
14.5, layer 1	10.2	5.4	80	9	14300	28600	11400			
22.5, layer 1	11.4	3.6	80	7.8	7500	20300	12200	180	11.7	30.5
28, layer 2	8.4	4	50	7.3	4700	14500	8400	120	8.6	20.5
Broadway Site										
26, layer 1	9.0	2.5	150	17	8900[c]	51600	21300	280	16.7	56
36[a], layer 2	7.6	1.6			34200			>840[b]	39[b]	153[b]

[a] Consider test results with caution; [b] roughly estimated; [c] initial modulus varies from 8900 to 5200 to 7400 ksf.

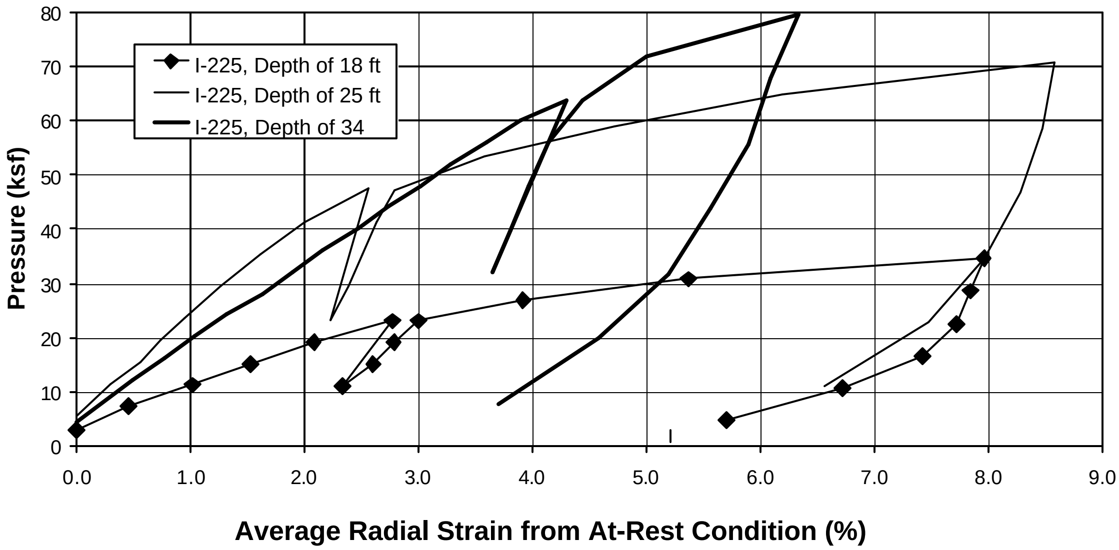


Figure 4.6. Pressuremeter Test Results at the I-225 Site.

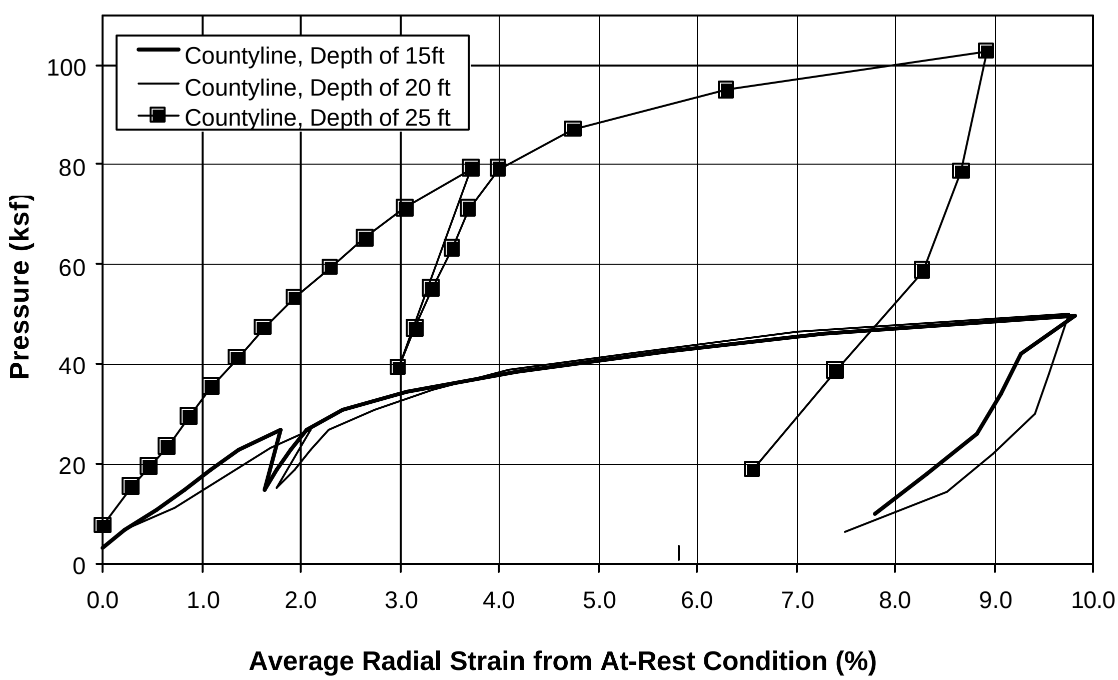


Figure 4.7. Pressuremeter Test Results at the County Line Site.

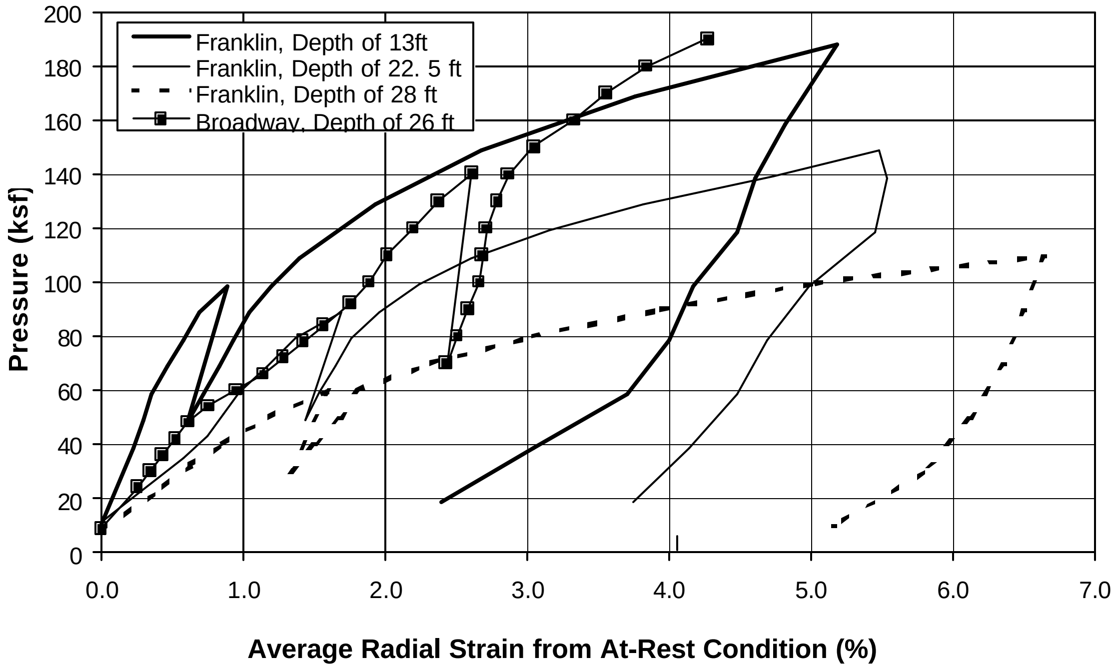


Figure 4.8. Pressuremeter Test Results at the Franklin and Broadway Sites

4.3 Results of O-Cell Load Tests

The O-Cell load test results for the I-225 and County Line test shafts are summarized in Figures 4.9 through 4.26. Figures 4.9, 4.11, 4.18, and 4.20 present the gross load versus the upward and downward movement of the O-Cell for all the test shafts. Figures 4.10, 4.12, 4.19, and 4.21 present for all test shafts the measured average side resistance versus average measured side movement for the zones located between the O-Cell and Level 1 strain gages, between Level 1 and Level 2 strain gages, and between Level 2 and top of the shaft. For the zone between the O-Cell and Level 1 strain gages for the Broadway test shaft, this study measured the largest: side resistance from O-Cell test, modulus from PM test, SPT values, and core strength values. This and many other similar observations suggest a strong correlation between measured test data from SPT N-values, strength from UC tests, stiffness modulus from PM tests, and resistance values from load tests.

4.3.1 I-225 and County Line Tests

Measured average side resistance vs. side movement data in the entire soil-like socket of I-225 and County Line shafts and the corresponding normalized load transfer curves are shown, respectively, in Figures 4.13 and 4.14. The maximum side movement at the end of the O-Cell load test was 1.6 inches for the I-225 shaft and 0.8 inches for the County Line shaft. The side resistance data level out by the end of the O-Cell load test for the I-225 shaft, but resistance continues to increase for the County Line shaft. Measured base resistance vs. settlement data in the soil-like claystone beneath the I-225 and County Line shafts, and the corresponding normalized load transfer curves are shown, respectively, in Figures 4.15 and 4.16. Ultimate resistance for the soil-like claystone was defined in Chapter 3 to correspond to the full mobilization of the resistance in the plastic range. Figures 4.13 and 4.15 suggest that the full side and base resistance was almost mobilized by the end of the load tests for both shafts. Therefore, and to be conservative, the measured side and base resistances at the end of the O-Cell load tests were called the ultimate resistance values, or f_{max} and q_{max}, respectively.

4.3.2 Franklin and Broadway Tests

Measured average side resistance vs. side movement data in the entire bedrock socket of the Franklin and Broadway shafts and the corresponding normalized load transfer curves are shown, respectively, in Figures 4.22 and 4.23. Figure 4.22 suggests that the side resistance is continuing to develop at the end of the O-Cell test (0.34 inch movement for Broadway and 0.13 inch for Franklin). In other words, the side shear resistance did not reach ultimate conditions at the end of the tests, but did start to show signs of being hyperbolic. This is consistent with the behavior observed in the harder units of the Eagle Ford clay-shale in Dallas, where it typically takes in excess of 0.5 inches of movement to fully mobilize the side shear resistance. The reason for high deflections in the Eagle Ford is that it is highly laminated, and the laminations open when the rock is excavated. They close up again when the shaft is loaded, but that results in unusually high deformations at side shear failure.

In Chapter 3, the side resistance for very hard claystone and sandstone was defined to correspond to 1% of the shaft diameter (0.54" for the Broadway shaft and 0.42" for the Franklin shaft). The measured final portions of the side resistance (f in ksf) vs. side movement data (w in inches) curves shown in Figure 4.22 were fitted to two logarithmic relations as: f=7.6Ln (w)+28.7 for the Broadway shaft and f=5.4 Ln (w)+23.7 for the Franklin shaft. The estimated side resistance from these two relations at w that correspond to 1% the shaft diameter was called the ultimate side resistance, $f_{max.}$

Measured base resistance vs. settlement data in the very hard sandstone and claystone beneath the Broadway and Franklin shafts, and the corresponding normalized load transfer curves are shown, respectively, in Figures 4.24 and 4.25. The curves in this figure suggest that the Broadway shafts settled around 0.2 inches before base resistance was engaged. This could be attributed to dirty bottoms of the shafts. Although the construction specifications require the shaft bottom to be clean and the contractor did his best to ensure that, it was very difficult to inspect and even to clean very deep shafts such as the Broadway shaft. Engineers seldom consider construction technique.

For the purpose of data analysis, it was assumed that the bottoms of the shaft were clean. The corrected base resistance-settlement curves for cleaned bottoms at the Broadway and Franklin shafts are shown in Figure 4.24. At the end of the O-Cell load test on Broadway shaft, the base resistance continued to develop and increased almost linearly with settlement without any sign of yielding (Figure 4.24). This was noticed in the corresponding Broadway PM test (depth of 36 ft). These observations suggest that the base resistance load measured at the end of the O-Cell load test on the Broadway shaft was far from the true ultimate base resistance load of the shaft. For the Franklin shaft, the base resistance almost leveled out after reaching the yield point (Figure 4.24) as was noticed in the corresponding PM test at depth of 28 ft (Figure 4.14). This indicates that the base resistance load measured at the end of the O-Cell test for the Franklin shaft was very close to the ultimate true base resistance load of the shaft. From the curves of Figures 4.15 and 4.24, the base resistance that corresponds to settlement equal to 5% of the shaft diameter for all shafts, which is the study definition of q_{max} for very hard claystone and sandstone, was determined and called the ultimate base resistance, q_{max}.

Test results from SPT, PM, and UC tests suggest that the bedrock at the Broadway site is much stronger and stiffer than the bedrock at the Franklin site. The ultimate base and side resistances are primarily controlled by the ultimate strength of the geomaterial. This explains why the true ultimate base resistance at the Broadway site was much higher than at the Franklin site. However, f-w and q-w relationships seem to coincide in the small range of measured movements (Figures 4.22 and 4.24). The early (elastic) portion of the load transfer curves is often controlled by many variables, not only the ultimate strength of the material. The most important variable is the mass modulus of the rock material, E_m. It was observed that the average weighted initial modulus values for the bedrock socket around the Franklin and Broadway shafts are very close (discussed more in Chapter 5). This could explain the early match in side resistance vs. movement data for the two shafts. Other factors that influence the initial response of the load transfer curves are: type of weak rock, degree of interface roughness, and length and diameter of the socket. The Franklin rock consists mostly of thinly bedded, bluish gray, sandy claystone. The Broadway bedrock consists predominately of well-cemented, bluish gray, clayey sandstone.

4.3.3. Summary of Load Test Results

The extracted shaft-head load versus shaft-head settlement for the I-225 and County Line shafts are shown in Figure 4.17, and for the Franklin and Broadway shafts they are shown in Figure 4.26. Note that the elastic compression of the I-225 and County Line shafts embedded in the soil-like claystone is negligible and could be neglected. For the two high-capacity shafts (Franklin and Broadway) embedded in very hard claystone and sandstone, the elastic compression movements of the shafts were significant and settlement curves for the compressible shafts are shown in Figure 4.26. Note that the abrupt change in the settlement curves for the Franklin and Broadway shafts occurs at settlement that corresponds to 1% the shaft diameter. At this settlement, it was assumed in the analysis that the ultimate side resistance is fully mobilized.

A summary of all O-Cell load test results is shown in Table 4.6. These results are presented for different zones: between Level 1 and Level 2 strain gages, between O-Cell and Level 1 strain gages, the entire bedrock socket, and beneath the base of the shafts. Note that these zones could conceivably be different from the rock layers defined in Section 4.1, since, unfortunately, the strain gages were not placed at the boundaries between different rock layers. The test results listed in Table 4.6 include:

- The measured side and base resistance values at the end of the test.
- The ultimate side resistance (f_{max}) and base resistance (q_{max}), and the corresponding ultimate resistance force of the shaft ($Q_{max}= A_s f_{max}+A_b q_{max}$). Note that A_s and A_b can be calculated from the shaft layout data presented in the Table 3.1.
- The allowable side and base resistance values (f_{all} and q_{all}), and the corresponding allowable design load (Q_{all}) and settlement, w_{all}.
- The base resistance, side resistance, and shaft resistance load that correspond to a displacement equal to 1% the shaft diameter (q_d and f_d, $Q_d= A_s f_d+A_b q_d$).

The lessons learned from the results presented in Table 4.6 are:

- ❑ The settlements of all shafts that correspond to the design loads (w_{all}) were smaller than the tolerable settlement of 0.65 inch, suggesting that the design loads calculated based on the Strength Limit control the design. For the Broadway shaft, this settlement was relatively large (0.5 inch). If the ultimate resistance for the very hard sandstone at the Broadway site was selected to correspond to higher displacements than those employed in this study, the settlements that will be developed at the design loads could exceed 0.65 inch and the Serviceability Limit would control the design.
- ❑ In the soil-like claystone at County Line and I-225 sites, 70% of the resistance to working loads is provided by means of side resistance.
- ❑ In the very hard claystone and sandstones at, respectively, Franklin and Broadway sites, 90% to 95% of the resistance to working loads is provided by means of side resistance. Therefore, the priority in future load tests is to acquire most or the entire side resistance-displacement curve.
- ❑ With the data shown in Table 4.6, a simplified version of the load-settlement curve can be drawn with three points (0,0), (Q_d, 0.01D), and (Q_{max}, 0.05D), as demonstrated in Figure 4.17 for the County Line shaft. If the compressibility of the shaft is significant (e.g., high-

capacity Franklin and Broadway shafts), the shaft elastic compression can be estimated as described in Chapter 3.

Table 4.6. Results of O-Cell Load Tests

Name and Layout of Test Shaft	Depth: from to (ft)	**Side Resistance (ksf)**				Depth Below	**Base Resistance (ksf)**			
		End of Test	f_{max}	f_{all}	f_d		End of Test	q_{max}	q_{all}	q_d
I-225, D= 3.5 ft, L_o = 6.5 ft, L= 16.1 ft	15.8 to 21.8	2.6	2.6	1.3	1.8	28.6 ft	55	55	27	27
	21.8 to 27.8	3.6	3.6	1.8	2.8					
	Socket: 12.5 to 27.8	3.1	3.1	1.6	2.3					
	Q_{max}= 1078 kips, Q_d= 662 kips, Q_{all} = 539 kips (70% from side resistance), w_{all} = 0.24”									
County Line D= 4 ft, L_o = 2, L= 14 ft	11.5-16.5	4.8	4.8	2.4	4	22 ft	53	53	27	22
	16.5-21.5	3.8	3.8	1.9	3.3					
	Socket: 8 to 21.5	3.4	3.4	1.7	3					
	Q_{max}= 1340 kips, Q_d= 876 kips, Q_{all} = 670 kips (76% from side resistance), w_{all} = 0.25”									
Franklin, D= 3.5 ft, L_o = 4.5, L= 20.8 ft	11.7-17.7	12.9	18.6	9.3	18.6	25.3 ft	259	236	118	71
	17.7-23.7	25.2	36.3	18.1	36.3					
	Socket: 4.5 to 23.7	13.2	19	8.5	19					
	Q_{max}= 6612 kips, Q_d= 5024 kips, Q_{all} = 3306 kips (90 % from side resistance), w_{all} = 0.2".									
Broadway D= 4.5 ft, L_o = 10.5, L= 30.1 ft	23.3-33.3	15.9	17	8.5	15.9	47 ft	331	318	159	71
	33.3-43.3	32.8	35.1	17.5	35.1					
	Socket: 17 to 43.3	22.4	24	12	24					
	Q_{max}= 15276 kips, Q_d= 11362 kips, Q_{all} = 7638 kips (95 % from side resistance), w_{all} = 0.5".									

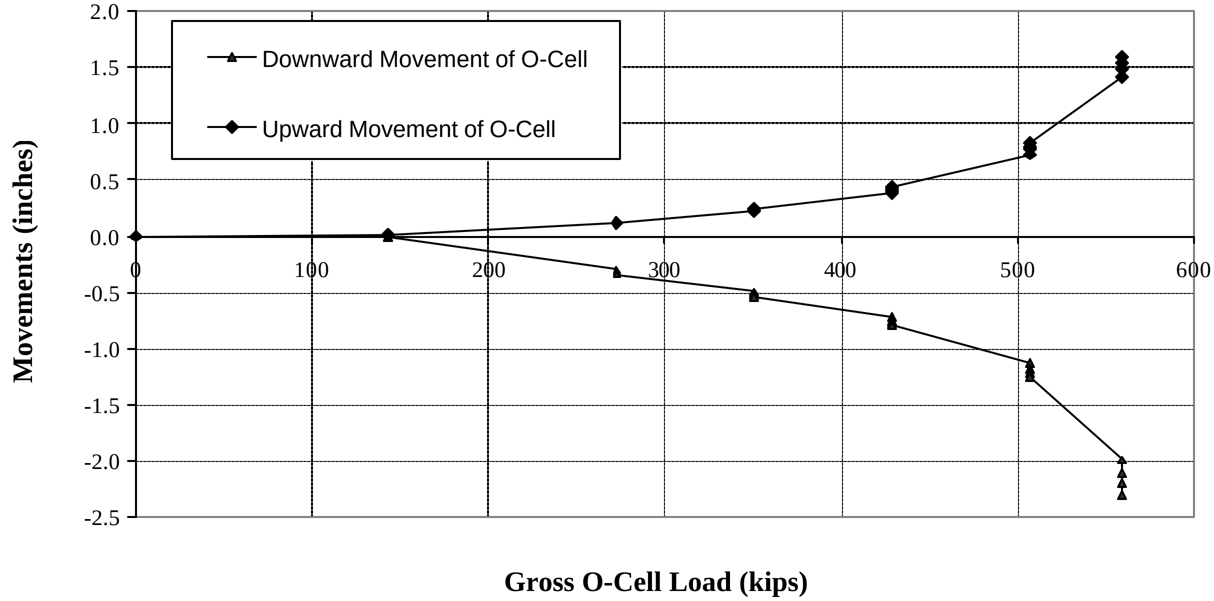


Figure 4.9. Results of O-Cell Load Test at the I-225 Test Shaft

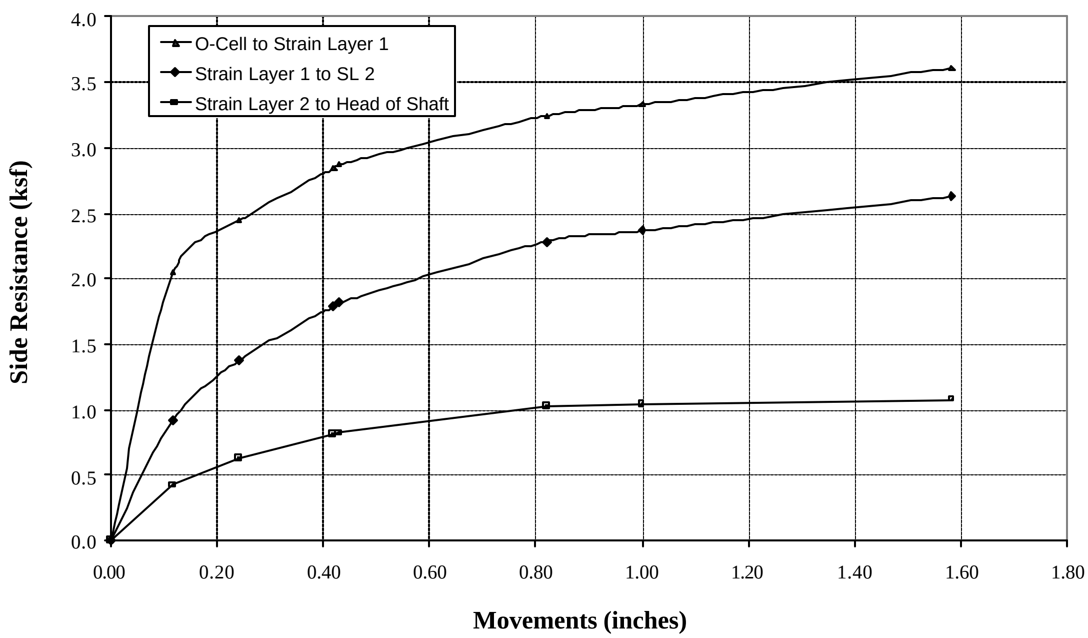


Figure 4.10. Side Resistance vs. Upward Movement at the I-225 Test Shaft

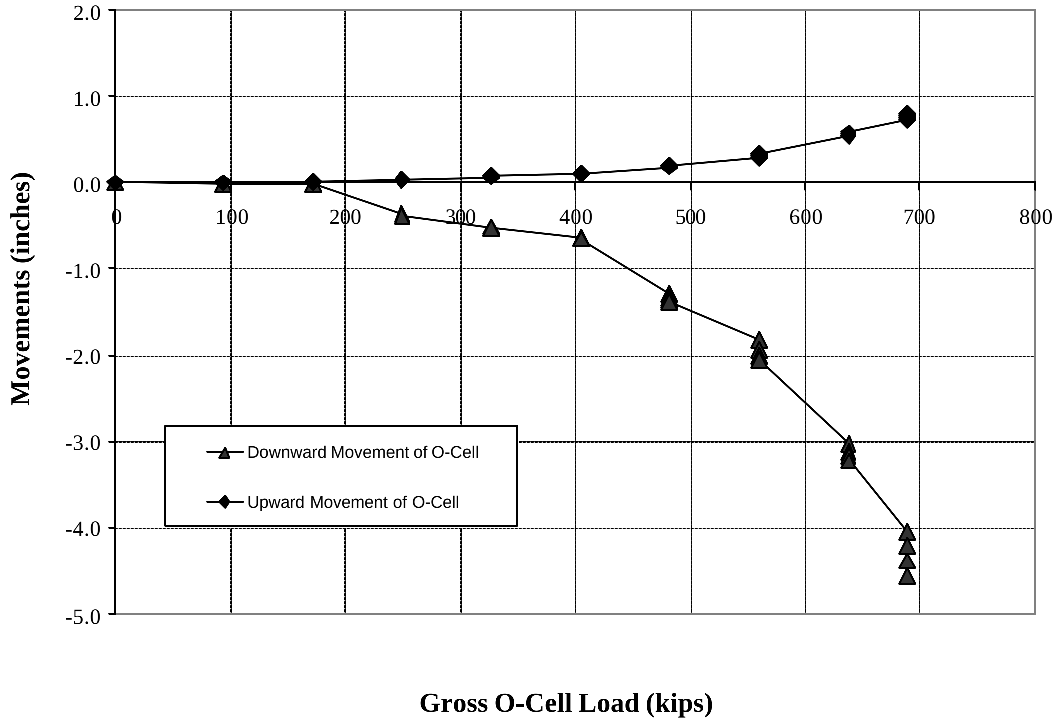


Figure 4.11. Results of O-Cell Load Test at the County Line Test Shaft

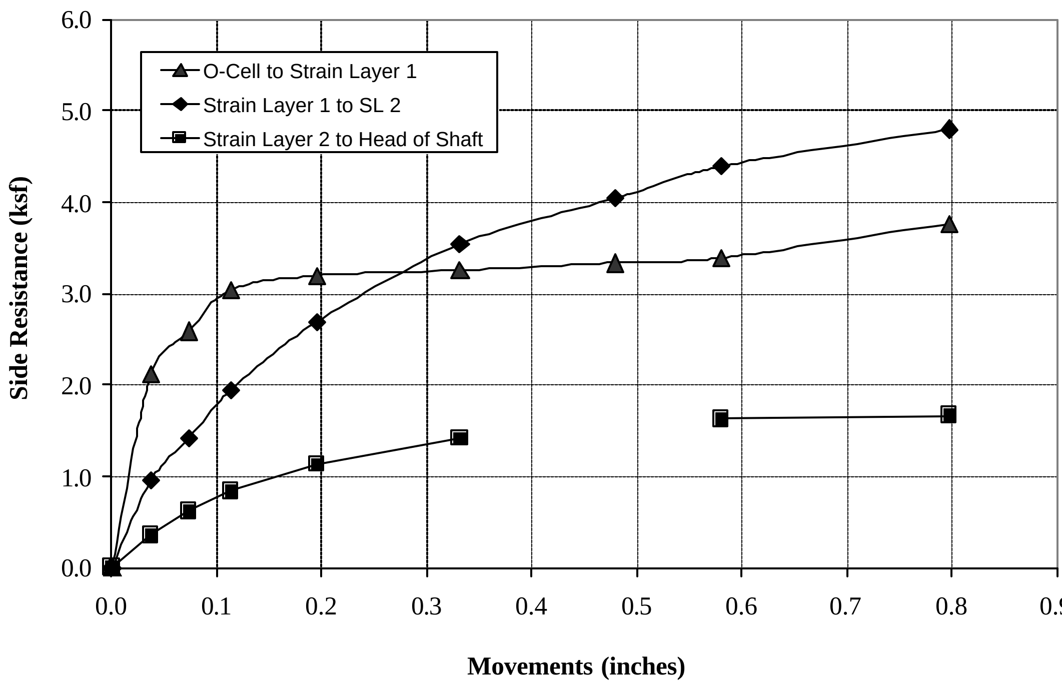


Figure 4.12. Side Resistance vs. Upward Movement at the County Line Test Shaft

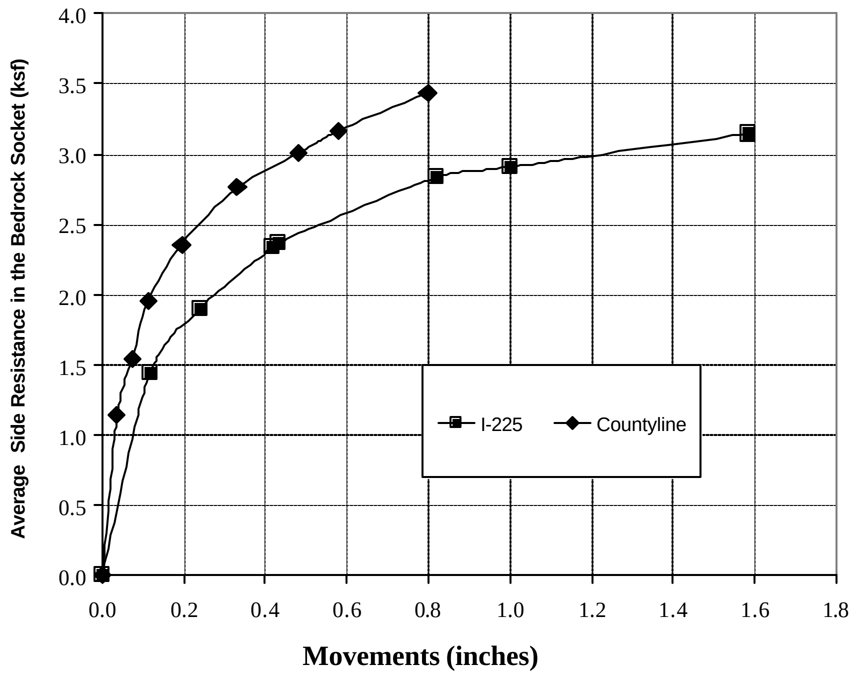


Figure 4.13. Side Resistance vs. Movement in the Entire Bedrock Socket: I-225 and County Line Test Shafts

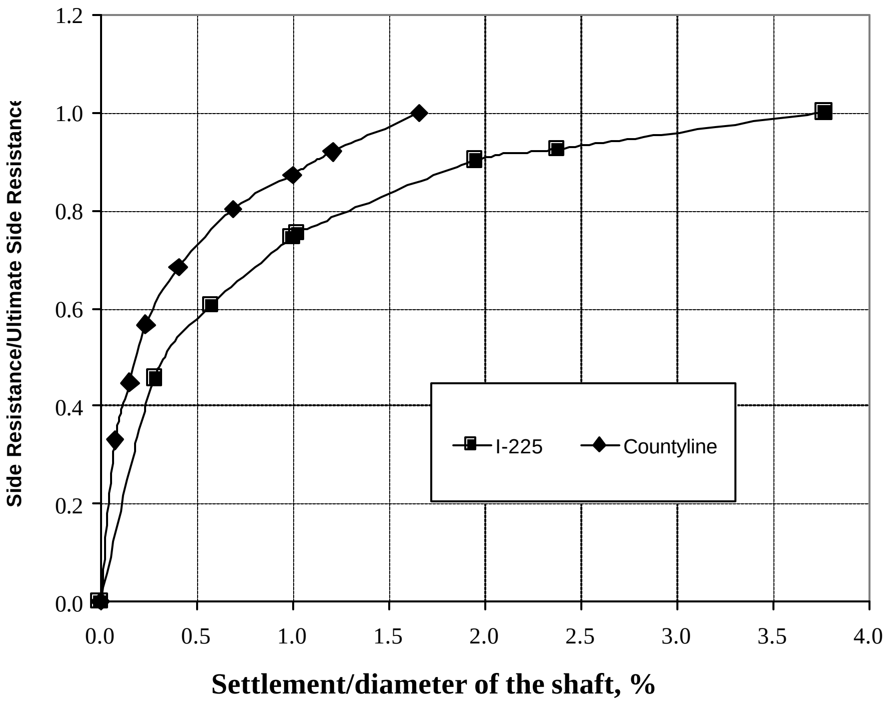


Figure 4.14. Normalized Load Transfer Relations for Side Resistance in the Entire Bedrock Socket: I-225 and County Line Test Shafts

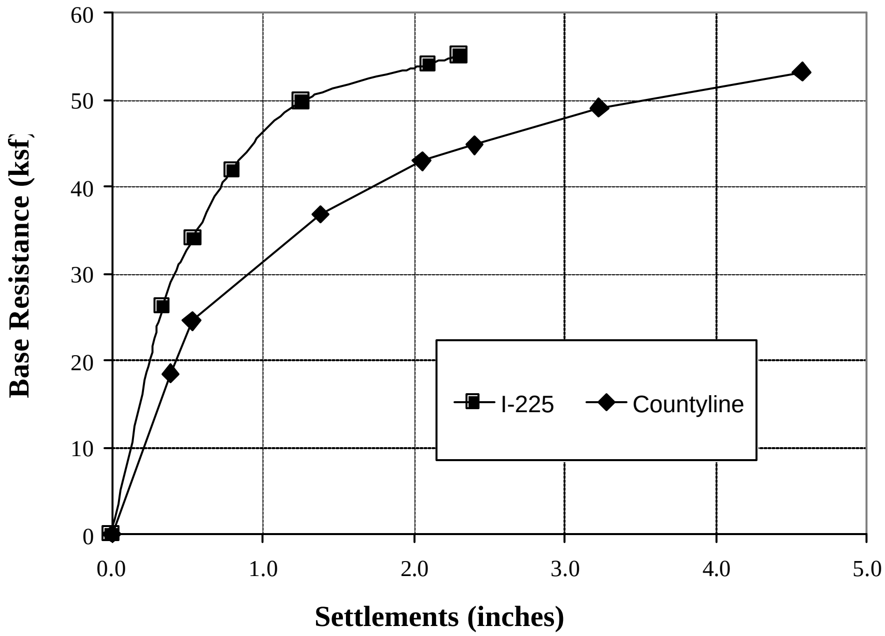


Figure 4.15. Base Resistance vs. Settlement: I-225 and County Line Test Shafts

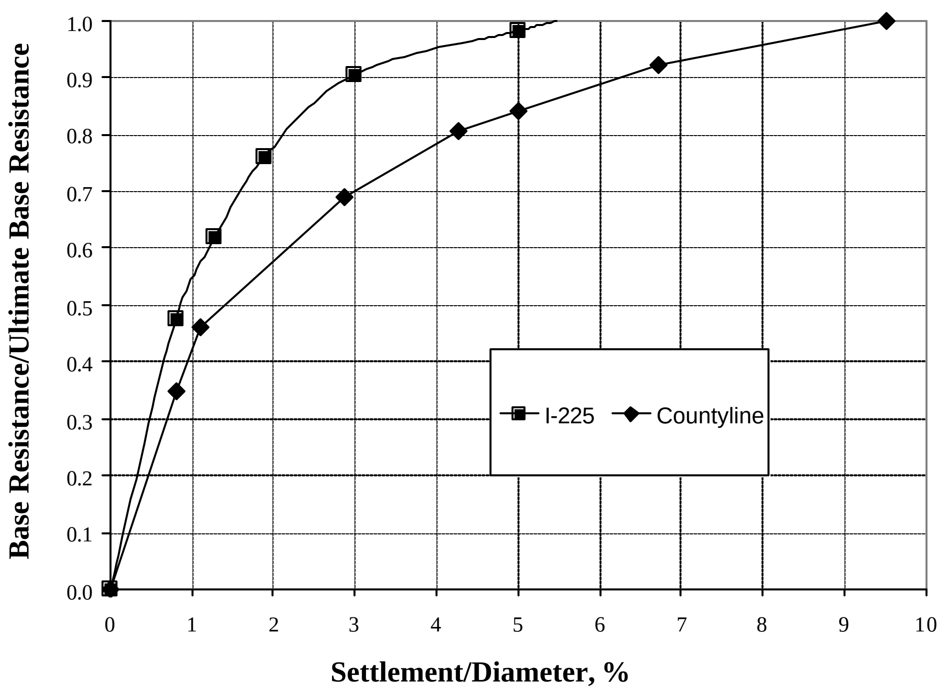


Figure 4.16. Normalized Load Transfer Relations for Base Resistance: I-225 and County Line Test Shafts

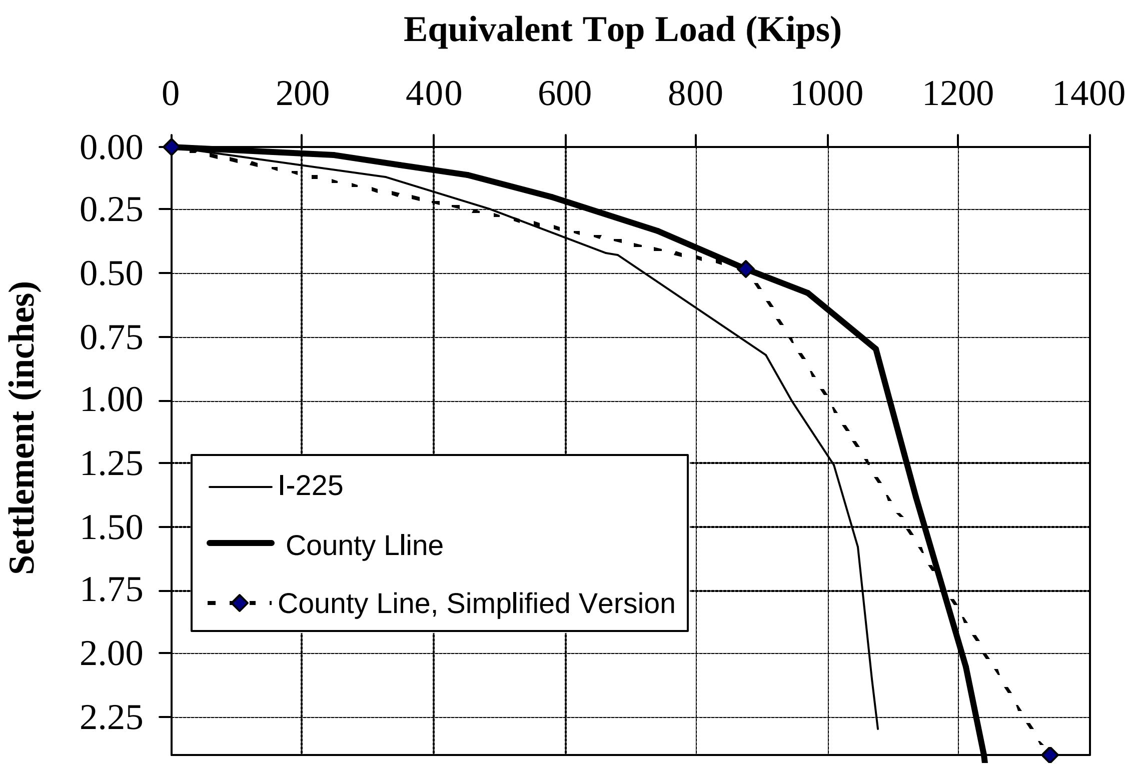


Figure 4.17. Extracted Load-Settlement Curves: I-225 and County Line Shafts

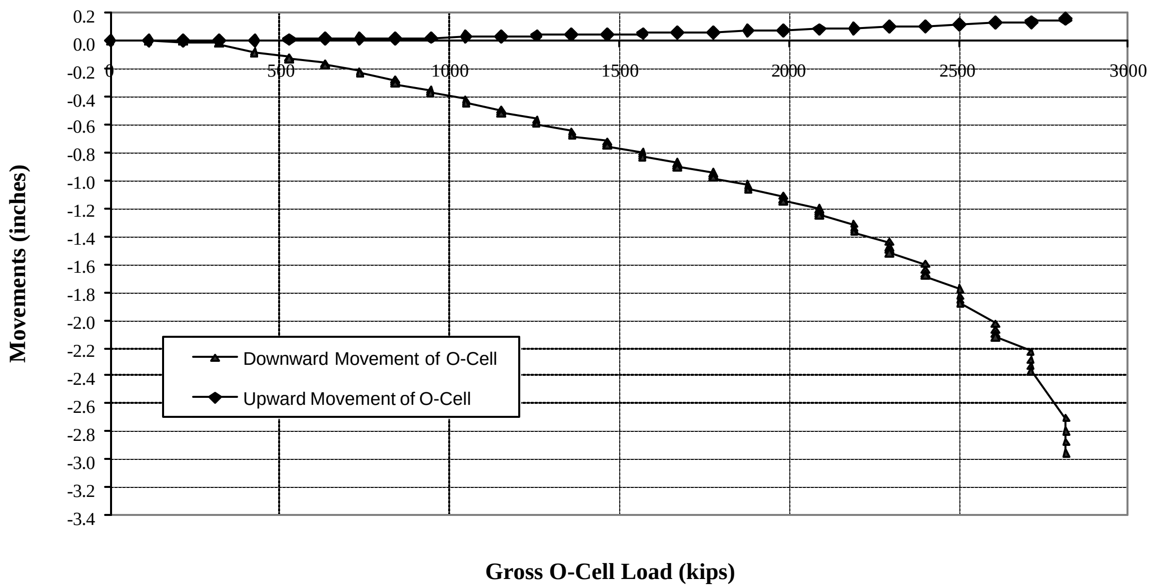


Figure 4.18. Results of O-Cell Load Test at the Franklin Test Shaft

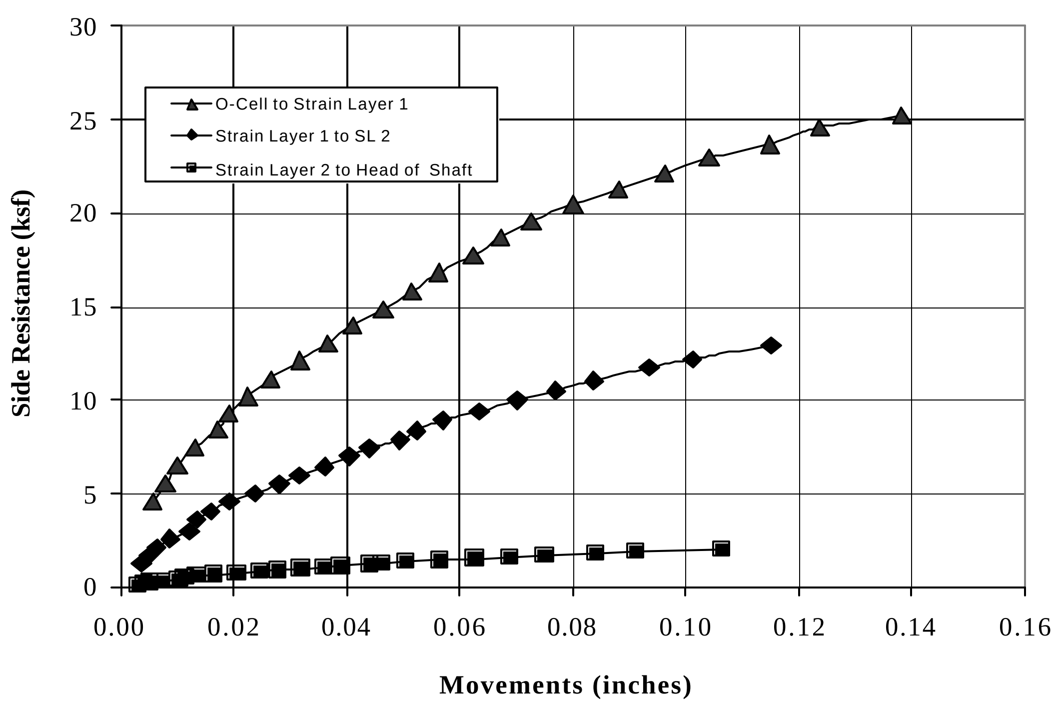


Figure 4.19. Side Resistance vs. Upward Movement at the Franklin Test Shaft.

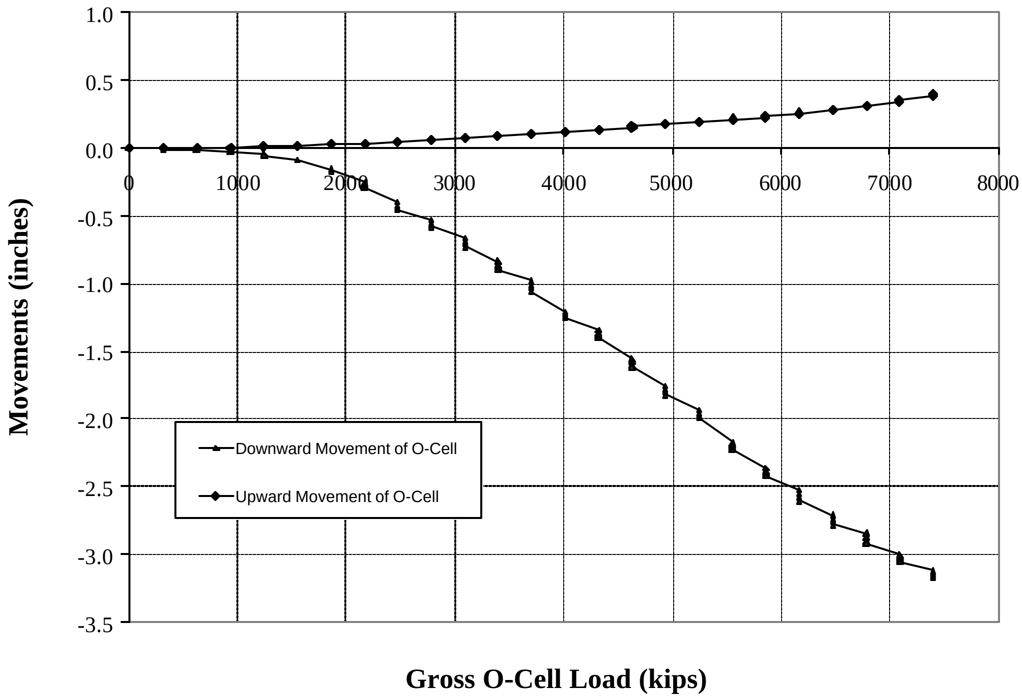


Figure 4.20. Results of O-Cell Load Test at the Broadway Test Shaft

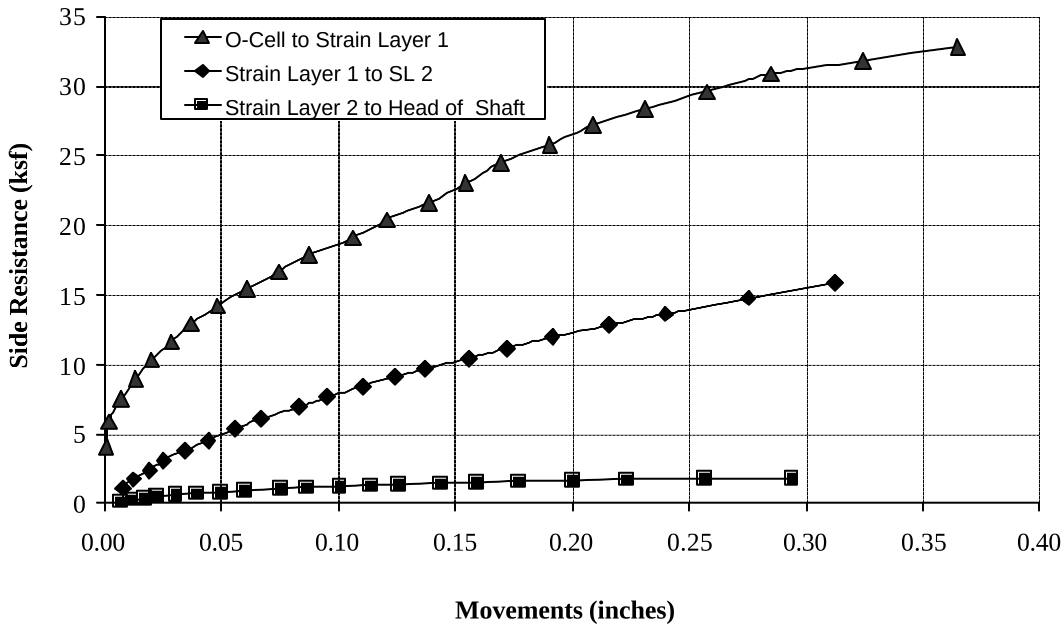


Figure 4.21. Side Resistance vs. Upward Movement at the Broadway Test Shaft

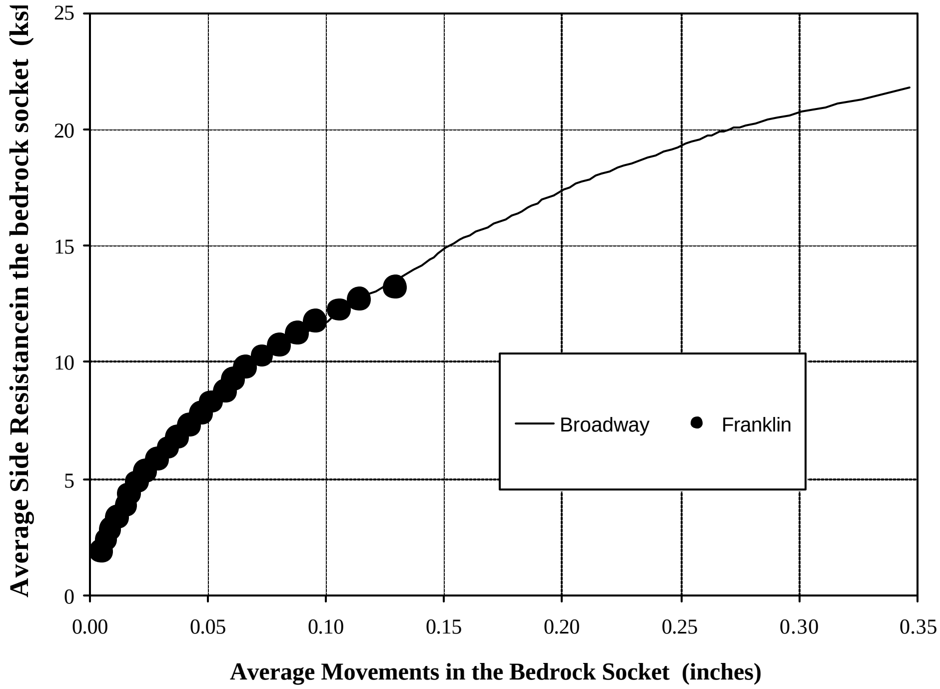


Figure. 4.22. Side Resistance vs. Upward Movement in the Entire Bedrock Socket: Franklin and Broadway Test Shafts

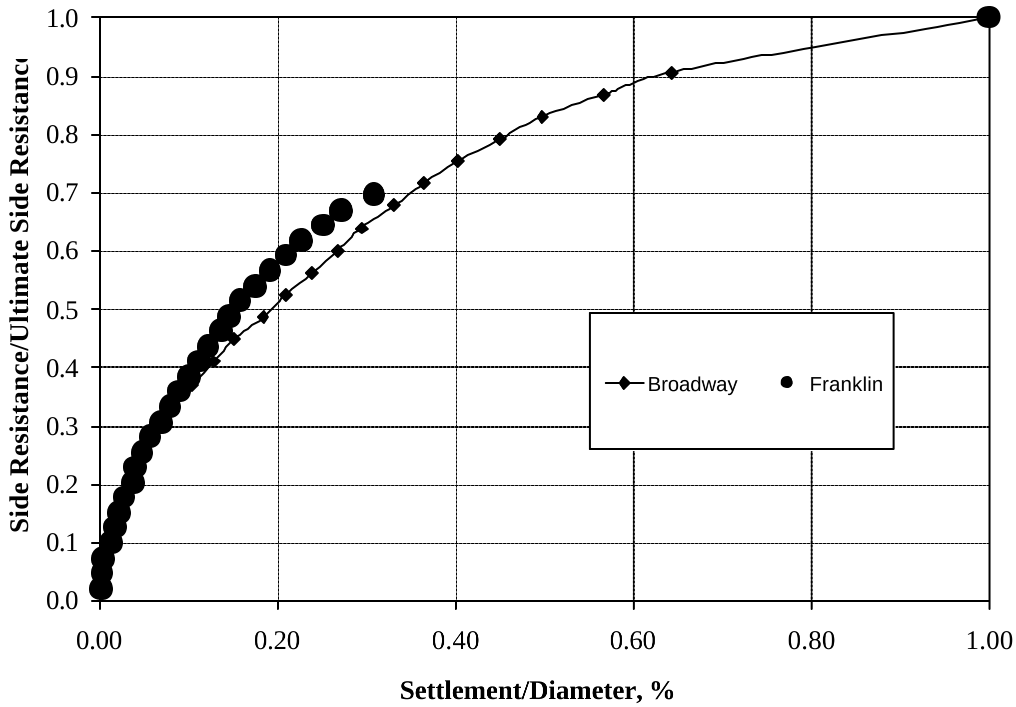


Figure. 4.23. Normalized Load Transfer Relations for Side Resistance in the Entire Bedrock Socket: Franklin and Broadway Test Shafts

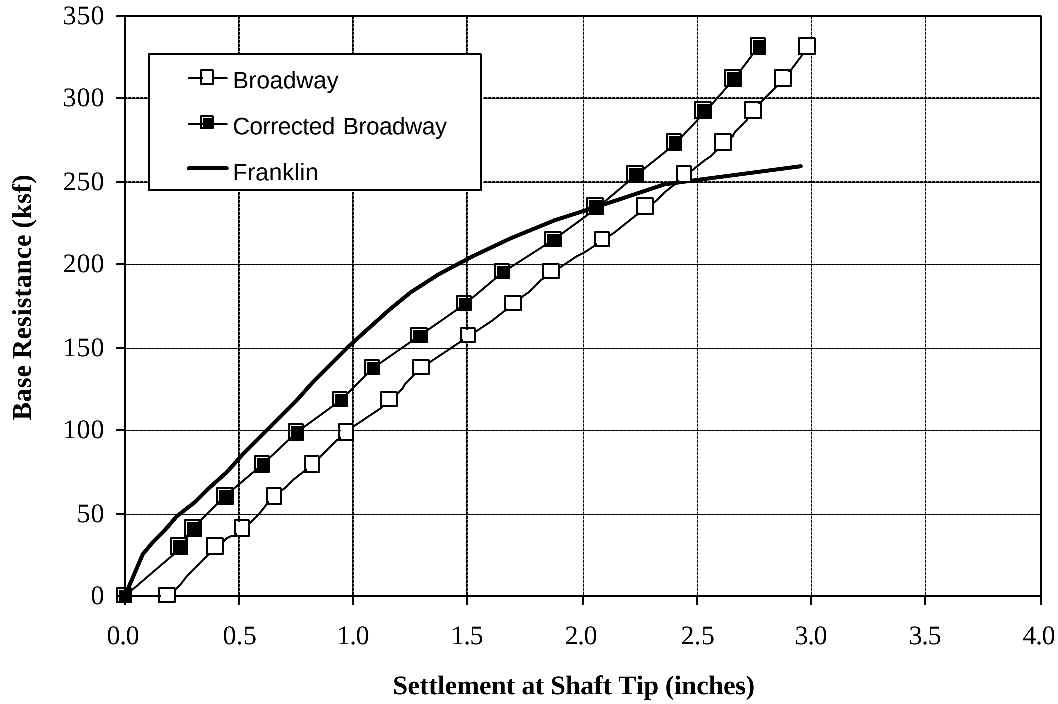


Figure 4.24. Base Resistance vs. Settlement for the Franklin and Broadway Test Shafts

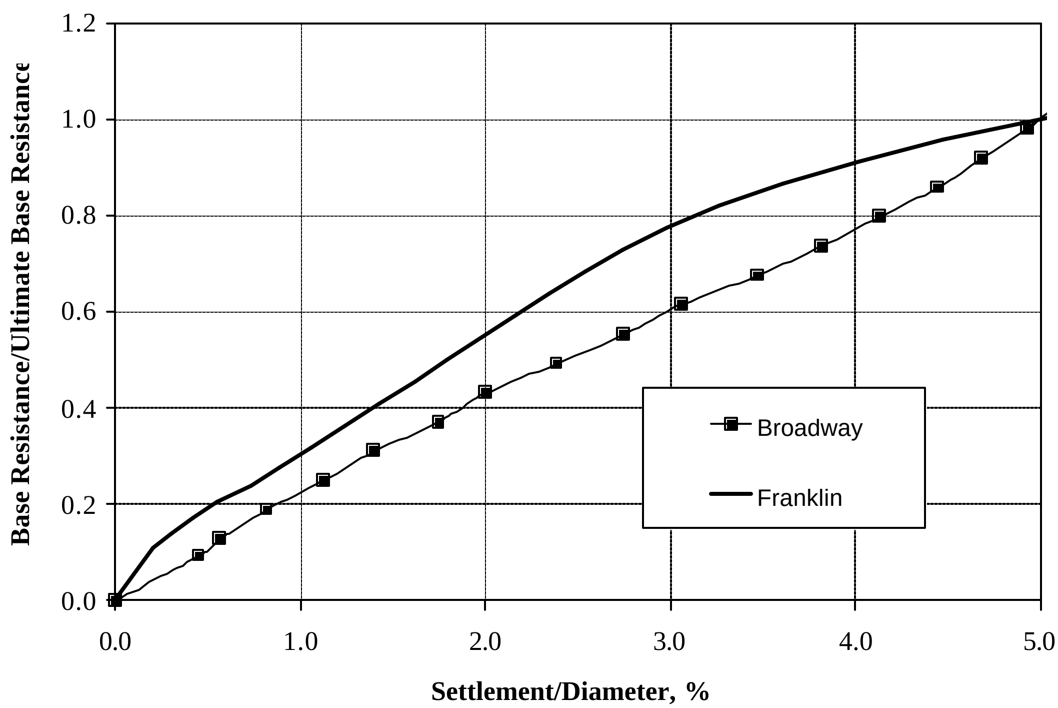


Figure 4.25. Normalized Load Transfer Relations for Base Resistance: Franklin and Broadway Test Shafts

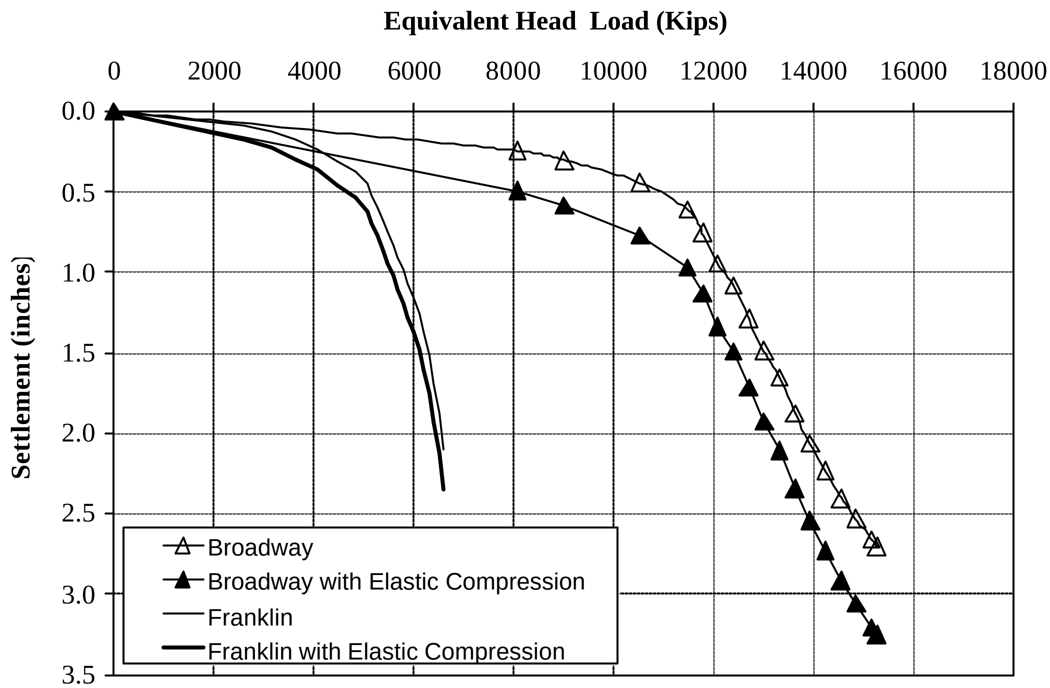


Figure 4.26. Extracted Load-Settlement Curve for the Franklin and Broadway Test Shafts

5. ANALYSES AND DISCUSSION OF TEST DATA

5.1 Introduction

The findings and conclusions presented in this chapter are based only on the limited number of data collected at the four load test sites, and therefore should be considered with precaution.

5.2 Correlation between SPT, PM, and UC Test Data

Table 5.1 shows the test results of the UC and PM tests at each load site for the rock layers described in the previous chapter. The unconfined strength data of the entire rock mass estimated indirectly from PM test results (q_{um}) were obtained by multiplying the estimated undrained shear strength (S_u) data listed in Table 4.5 by 2. Comparison of the UC strength data obtained from PM and UC tests presented in Table 5.1 revealed the following:

- The FHWA expression for indirect estimation of the undrained shear strength from PM test results (Eq. 3.4) is appropriate for the soil-like claystone at I-225 and County Line Sites.
- The Gibson-Anderson expression for indirect estimation of the undrained shear strength from PM test results (Eq. 3.3) is appropriate for the competent sandstone and claystone at the Franklin and Broadway sites.
- It was very difficult to collect reliable core specimens for strength tests in the soil-like claystone (e.g., Layer I at I-225 site) and in the fractured rock (experience from a load test not documented in this report). The PM tests produced more consistent strength data for these geomaterials and could be made very reliable if the correlation equations suggested above are investigated further and made more accurate.
- Test data from the in-situ PM test and SPT for the soil-like claystone were more consistent than the strength data collected from the core specimens. For example, the core strength data in Layer 1 of the County Line site ranges from 2.2 to 10.4 ksf. However, in the same rock layer, SPT N values range from 33 to 41 and the results of two PM tests matched each other. This suggests that for the soil-like claystone, a factor of safety based on SPT and PM test data can be smaller than if it is based on strength data collected from cores. In the uniform very hard sandstone at the Broadway site, the strength data obtained from core specimens were very consistent with small coefficient of variation.

Table 5.2 shows all the UC, PM, and SPT test results on different uniform rock layers that were considered in the analysis. All these data were collected from the test holes drilled around the test shaft. The UC strength data on rock core specimens were not available for Layer 1 in I-225 site and vary from 2.2 ksf to 10.4 ksf for Layer 2 in the County Line site (Table 5.1). Based on correlations with SPT and PMT, the strength data were selected for these layers as shown in Table 5.2. Rock Layer 2 at I-225 site was selected in this study to represent the zone beneath the I-225 socket because a very hard and relatively thin sandstone layer existed beneath that layer (Section 4.1.1). Only one UC strength test (13.1 ksf) was available in that layer and was very close to the value estimated from one PM test (12 ksf). The larger strength data (13.1 ksf) was selected (Table 5.2) to be on the conservative side (i.e., the correlation factor between strength and resistance, like α, would be smaller with the larger strength data). Beneath the socket of the County Line shaft, two UC strength tests were available (14.8 to 18.9 ksf) with an average (16.8 ksf) that is very close to the value estimated from one PM test (15.4 ksf). Around the Franklin socket (Layer 1), the average strength from five UC tests was 64 ksf, which is close to the average value estimated from two PM tests (66 ksf). Beneath the Franklin socket, one UC test generated a strength of 35 ksf that was very close to the value estimated from the PM test (41 ksf). The larger strength value of 41 ksf was selected in the analysis. The strength data obtained from the Broadway load test site for each rock layer were consistent (small coefficient of variation), so the mean value of these data was considered in the analysis.

Table 5.1. Results of UC and PM Tests

Weak Rock Description	Location	UC Test Results on Intact Core Specimens (ksf)		PM Test Results on Rock Mass (ksf)			
		q_{ui}	E_i	q_{um}, Eq. 3.4	q_{um}, Eq. 3.3	E	E_r
Soil-Like Claystone	I-225, Layer 1			7.4	14.6	970	2400
	I-225, Layer 2	13.1	1055	12	26.4	2550	6000
	County L. Layer 2	2.2-10.4		9.6	19.2	1800	3300
	County L. Layer 3	14.8-18.9	1100	15.4	37.6	3200	6600
Very Hard Sandy Claystone	Franklin, Layer 1	38.7-87.2	7485	25.5	66	11050	11900
	Franklin, Layer 2	35.3	5204	17.2	41	4700**	8400
Very Hard Clayey Sandstone	Broadway, Layer 1	97	10663	33	112	8900	21300
	Broadway, Layer 2	293	48609	78	306	34200	
	Broadway, Layer 3	219	29600				

* SPT terminated in the 1st 6-inches penetration interval.
**The results of the O-Cell load test do not support the use of this value.

Table 5.2. Results of Simple Geotechnical Tests Considered in the Analysis

Weak Rock Description	Location	Analysis Data (units in ksf)						SPT N-Value
		q_{ui}	E_i	E_i / q_{ui}	E_m	E_m /E_i	E_m/q_{ui}	(bpf)
Soil-Like Claystone	I-225, Layer 1	8.3			970			32
	I-225, Layer 2	13.1	1055	81	2550	2.4	195	58
	County L. Layer 1	10.4			1800		173	38
	County L. Layer 2	16.8	1100	71	3200	2.9	189	61
Very Hard Sandy Claystone RQD= 80%-100%	Franklin, Layer 1	64	7485	113	11050	1.5	173	50/4” or ~ 150
	Franklin, Layer 2	41	5204	127				*50/5” or ~120
Very Hard Clayey Sandstone RQD= 80%-100%	Broadway, Layer 1	97	10663	110	8900	0.8	92	50/3
	Broadway, Layer 2	293	48609	166	34200	0.7	117	*100/5.5”
	Broadway, Layer 3	219	29600	135				*83/6

* SPT terminated in the first 6-inches penetration interval (data are not accurate due to seating problem, see Section 6.5.2).

Figure 5.1 shows SPT N values (bpf) plotted against the measured unconfined compressive strength on intact cores. The most conservative form of the relation between q_{ui} and N values (not the best fit relation) is

q_{ui} (ksf) =0.24 N 5.1

where no strength data can be obtained on cohesive IGM, the FHWA manual (1999) recommends as a last resort the use of SPT blow counts. In such case, the compressive strength (in ksf units) can be crudely estimated as 0.27 N. This correlation, according to the FHWA manual, is unreliable and should be used only as a last resort. The correlation found in this study is very close to that recommended in the FHWA manual.

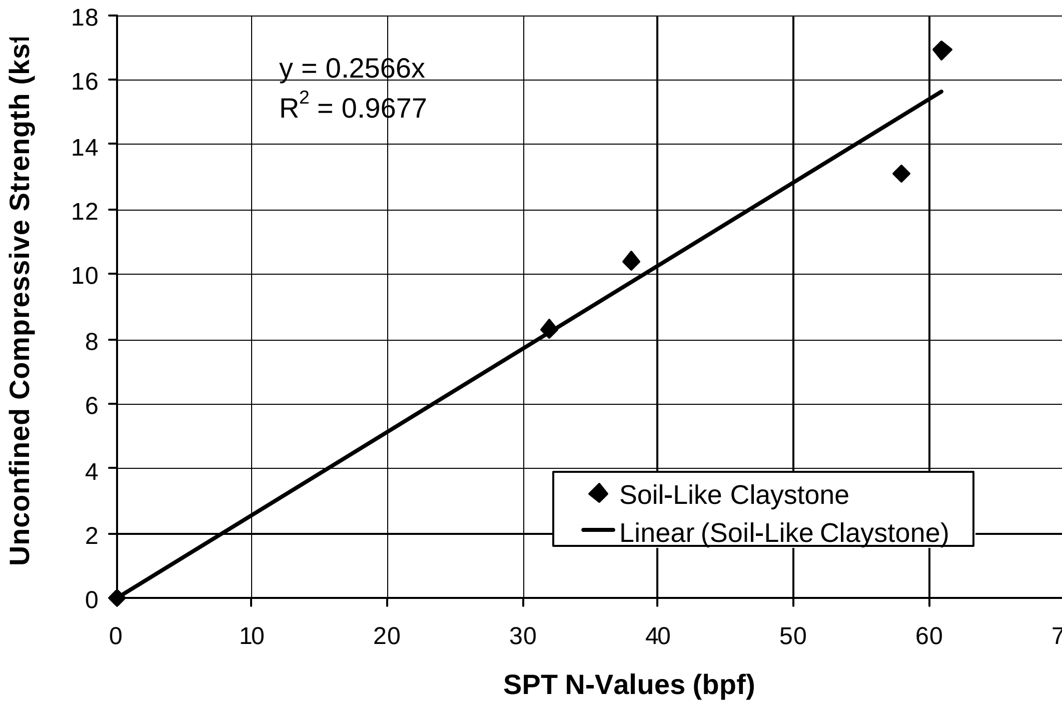


Figure 5.1 Relation between SPT-N value and the Unconfined Compressive Strength for Soil-Like Claystone

5.3 Stiffness Modulus of the Rock Mass (E_m) Considered in the Analysis

The initial modulus measured from the PM test (E) was selected over the reload modulus (E_r) to represent the mass Young's modulus of the rock mass (E_m) for the following reasons (see Table 5.1 to support ideas presented below and note that E_i is the modulus of intact rock core specimen measured from the UC test):

- For analysis of a test shaft, O'Neill et al. (1996) indicated that the initial modulus better represents the mass modulus of the rock, even with the disturbance at the borehole-concrete interface. In drilled shafts, the confining lateral pressure is released and then stressed back, exactly as would occur in the test pocket of the PM tests, which causes rock disturbance at the borehole sides. The disturbance in weak rocks is less than in soils, and it was corrected in the data analysis. The measured ratios of PM reload to initial moduli in this study range from 1 to 3, much smaller that those reported in the literature for soils (4 to 8). For the first two PM tests at Franklin, the initial modulus was comparable with the reload modulus (Table 4.5), suggesting that the disturbance of the rock mass was very minimal.
- The PM initial modulus (E) was closer (Table 5.1) to the initial modulus measured from UC tests on intact cores (E_i) than the reload modulus (E_r). In the soil-like claystone, the initial moduli obtained from UC tests were lower than those obtained from the in situ PM tests. This could signify the importance of the confining pressure in the soil-like claystone. In the very hard claystone and sandstone at Franklin and Broadway, in which the influence of confining pressure may be less important, E was close to or smaller than E_i. This could be attributed to the presence of fractures in the in-situ rock mass.
- Problems with measurements of reload and unload modulus occurred as presented in the previous chapter. The initial modulus was established using many points, whereas the unload modulus was calculated based on two points, and the reload modulus was based on three to four points.
- The initial PM modulus was measured during the early stage of elastic response and the reload modulus was often measured at the end of the elastic phase, which could explain the differences between the two moduli. Close reload and initial moduli were obtained when the unload-reload cycle was performed early in the elastic range. Hence, E is more appropriate

than E_r to predict the response of the test shaft in the initial design load range, where small strains and movements occur. E_r could be more appropriate to represent stiffness of the weak rock at conditions closer to ultimate strength.

- Data analysis presented later in this chapter using E to represent E_m yielded more reasonable correlation equations and results than using E_r to represent E_m.

If the intermediate geomaterial is relatively uniform with depth, E_m can be back-calculated from the results of load tests (load-settlement curve) as fS/D, where S is the initial slope of load-settlement curve and D is the diameter of the test shaft (O'Neill et al., 1996). The factor f is 0.13 for q_{ui} =10 ksf and 0.33 for q_{ui}=100 ksf. It is hard to assume that any of the test sockets investigated were uniform. From the Franklin load test results, E_m was estimated as 11000 ksf for the Franklin socket, which is very close to E_m measured for layer 1 around the test socket from the PM test (Table 5.1). The load test results do not support the low E_m measured beneath the Franklin test socket (Layer 2), and therefore, this E_m value was not included in Table 5.2.

Based on the results of Table 5.2, it is recommended to use a conservative ratio between the mass stiffness of the rock and unconfined compressive strength of intact rock (E_m/q_{ui}) equal to 150 for claystone (soft to very hard) and 100 for the very hard sandstone. For the soil-like claystone, the ratio (E_m (ksf) /N (bpf)) can be taken as 40. For all materials, a best-fit expression can be taken as E_m (ksf) = 1016 $q_{ui}^{0.5}$ and for design purposes as E_m (ksf) = 600 $q_{ui}^{0.5}$, where q_{ui} is also in ksf.

The FHWA (1999) and AASHTO (1998) design manuals provide several relations to estimate E_m as a function of q_{ui}, E_i, and RQD (Figures C10.8.3.5-2 and 3 in AASHTO 1998, and Table B.5 in the FHWA manual). According to O'Neill et al. (1996), E_m can be taken as 115q_{ui} for Category 2 cohesive intermediate geomaterial (IGM) and 250 q_{ui} for Category 1 cohesive IGM, provided soft seams and open fractures are not present. For many cohesive IGMs, E_i is between 100 and 200 q_{ui}. The most direct method of determining properties of the rock mass and interface properties is to conduct a field load test. Based on back analysis of load test data, Rowe and Armitage (1987), recommended a best fit expression as E_m (ksf) = 987 $q_{ui}^{0.5}$ and for design purposes E_m (ksf) = $690q_{ui}^{0.5}$. All these observations seem to be consistent with the findings of this study.

5.4 Data Analysis

Test data obtained within or around the bedrock socket for all test shafts (q_{ui}, E_m, N-value, f_{max}, f_{all} and f_d) are summarized in Table 5.3, where f_{max}, f_{all}, f_d, are, respectively, the maximum side resistance, allowable side resistance, and side resistance at settlement equal to 0.01D (where D is the socket diameter). Note that for very hard claystone and sandstone, it is assumed in the analysis of load test data that $f_{max}=f_d$. For the I-225 and Broadway test shafts, the side resistance results around the socket are presented for three zones: between the O-Cell and Level 1 SG (strain gages), between Level 1 and Level 2 SGs, and within the entire rock socket. The second and third zones cross different uniform rock layers presented in Tables 5.1 and 5.2. Therefore, the weighted averages of strength, stiffness, and SPT-N for these two zones are calculated and presented in Table 5.3. For example, the SPT N value within the entire rock socket at I-225 site was calculated as 41 based on the measured SPT N value for Rock Layer 1, that extends from 12.5 ft to 22.5 ft (N= 32), and Rock Layer 2 that extends from depth 22.5 to 27.8 ft (N= 58). For the County Line and Franklin test shafts, test data for the entire bedrock socket are presented (Rock Layer 1 in Table 5.2). Test data obtained below the bedrock socket of all test shafts (q_{ui}, E_m, N-value, q_{max}, q_{all}, and q_d) are summarized in Table 5.4, where q_{max}, q_{all}, q_d, are, respectively, the maximum base resistance, allowable base resistance, and base resistance at settlement equal to 0.01D. Note that E_m beneath the Broadway Socket listed in Table 5.4 was estimated (not measured from PM test) based on measured UC strength data of rock beneath and around the rock socket.

For the soil-like claystone, Figure 5.2 shows measured f_{max} vs. q_{ui} and measured f_d vs. q_{ui}, and Figure 5.3 shows measured f_{max} vs. N and measured f_d vs. N. The best-fit design relations are shown in these figures with their corresponding R-squared values. From these figures and the results shown in Tables 5.3 and 5.4, recommended design equations for side and base resistance, which are based on results of SPT, UC, and PM tests, are listed in Table 5.5. Also shown in this table is the procedure to construct an approximate load-settlement curve as a function of the SPT- N values (bpf) and the unconfined compressive strength (q_{ui}). Note that the strength-based design equations for the soil-like and hard claystone are the same (Table 5.5). The appropriateness of q_{max}= 3.8 q_{ui} for the claystone will be discussed later. Use q_{ui} (ksf)= 0.24 N to convert from SPT-based design equation to strength-based design equations for soil-like claystone.

Table 5.3. Test Data Obtained within or around the Bedrock Socket for all Test Shafts

Site	Rock Type	O-Cell Load Test			UC Test	PM Test	SPT-N
		f_{max} (ksf)	f_{all} (ksf)	f_d (ksf)	q_{ui} (ksf)	E_m (ksf)	(bpf)
I-225	Soil-Like Claystone	2.6	1.3	1.8	8.3	970	32
		3.6	1.8	2.8	12.3	2550	55
		3.1	1.6	2.3	10	1513	41
County line		3.4	1.7	3.0	10.4	1800	38
Franklin	Very Hard Sandy Claystone	19	8.5	19	64	11050	50/4"
Broadway	Very Hard Clayey Sandstone	17	8.5	15.9	97	8900	50/3”
		35.1	17.5	32.8	210	23448	*50/2”
		24	12	22.4	145	15025	*50/2.5”

* Roughly estimated data based on the results of Table 5.2.

Table 5.4. Test Data Obtained below the Bedrock Socket of all Test Shafts

Site	Rock Type	O-Cell Load Test			UC Test	PM Test	SPT-N
		q_{max} (ksf)	q_{all} (ksf)	q_d (ksf)	q_{ui} (ksf)	E_m (ksf)	(bpf)
I-225	Soil-Like Claystone	55	27	27	13.1	2550	58
County line		53	27	22	16.8	3200	60
Franklin	Very Hard Sandy Claystone	236	118	71	41		50/5"
Broadway	Very Hard Clayey Sandstone	318	159	71	219	21900**	*83/6”

* SPT terminated in the first 6-inches penetration interval.
** Estimated based on Results of UC tests.

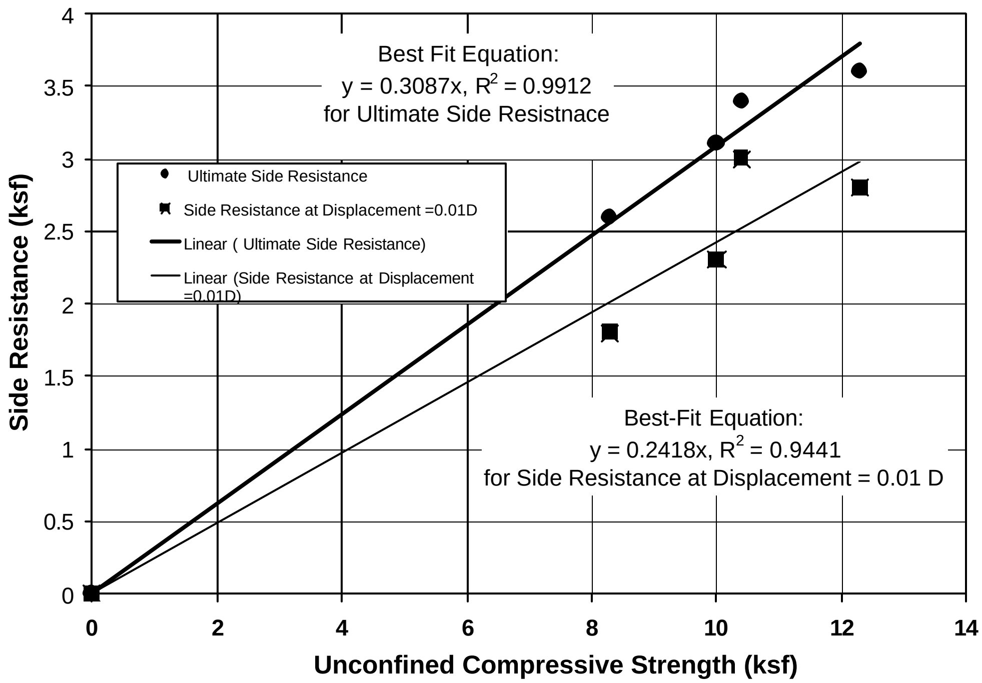


Figure 5.2 Side Resistances vs. Unconfined Compressive Strength (ksf) for Soil-like Claystone

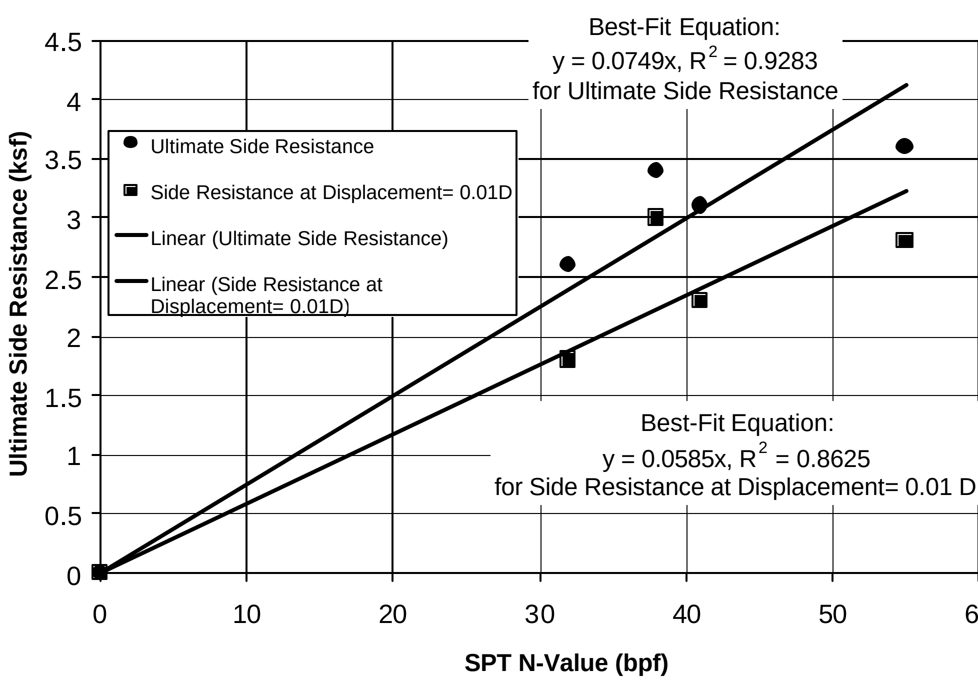


Figure 5.3 Side Resistances vs. SPT–N Value for Soil-like Claystone

Table 5.5. Best-Fit Design Equations for Drilled Shafts Based on the Results of Load Tests, and SPT, UC, and PM Tests

<table>
<tr><th>Description</th><th>Soil-Like Claystone (I-25@ I-225 and County Line)</th><th>Very Hard Sandy Claystone (I-25@ Franklin)</th><th>Very Hard Clayey Sandstone (I-25@ Broadway)</th></tr>
<tr><td colspan="4">Note: Units are ksf for all strength, resistance, and stiffness values, bpf for SPT- N values, ft for D, and ft^2 for A_s and A_b</td></tr>
<tr><td>SPT- Based Design Method for Side Resistance</td><td>f_{max}= 0.075 N, f_{all}= 0.037 N, f_d= 0.06 N</td><td colspan="2" rowspan="2">N/A. Future research should investigate design methods based on SPT N values for the very hard claystone and sandstone bedrock, (see Chapter 6).</td></tr>
<tr><td>SPT- Based Method for Base Resistance</td><td>q_{max}= 0.92 N; q_{all} = 0.46 N, q_d = 0.42 N</td></tr>
<tr><td rowspan="2">Strength- Based Design Method for Side Resistance</td><td colspan="2">f_{max}= 0.30 q_{ui}, f_{all}= 0.15 q_{ui}</td><td rowspan="2">f_{max}=f_d=0.17 q_{ui}; f_{all}= 0.09 q_{ui}</td></tr>
<tr><td>f_d= 0.24 q_{ui}</td><td>f_d= f_{max} =0.3 q_{ui}</td></tr>
<tr><td>Strength- Based Design Method for Base Resistance</td><td colspan="2">q_{max}= 3.8 q_{ui} ; q_{all}= 1.9 q_{ui}; q_d= 1.7 q_{ui}</td><td>q_{max}= 1.45 q_{ui} ; q_{all}= 0.73 q_{ui}; q_d= 0.32 q_{ui};</td></tr>
<tr><td rowspan="2">Stiffness- Based Design Equations</td><td>f_d=0.0019 E_m</td><td colspan="2">f_d=f_{max}=0.0017 E_m</td></tr>
<tr><td colspan="3">f_d=0.0018 E_m</td></tr>
<tr><td>Very Approximate Load-Settlement Curve for Rigid Shafts as a function of q_{ui} and SPT-N values. *</td><td colspan="3">Three points: (0,0), (Q_d, 0.01D), (Q_{max}, 0.05D)
Q_d (ksf) = $A_s f_d + A_b q_d$;
Q_{max} (ksf) = $A_s f_{max} + A_b q_{max}$
The relations for f_{max}, f_d, q_{max}, q_d as a function of N and q_{ui} are listed above; D is the shaft diameter; A_b and A_s are, respectively, the base and side areas of the rock socket.</td></tr>
</table>

* Follow the procedure in Chapter 3 to account for the compressibility of high-capacity shafts embedded in very hard claystone and sandstone. No correction is needed for the low-capacity shafts embedded in soil-like claystone.

The measured side and base resistances at the relatively small displacement of 0.01D (f_d and q_d) are expected to be dependent on the initial elastic modulus of the geomaterial or E_m. Also, because f_{max} = f_d for the very hard claystone and sandstone, it could be argued that f_{max} for these materials is more dependent on E_m than q_{ui}. This is very clear in the results shown in Table 5.3. In addition, the strength-based side resistance design equations for the hard claystone are different from those in the hard sandstone. Results of E_m vs. f_d for all materials investigated in this study (soil-like and hard claystone and hard sandstone) are summarized in Figure 5.4. A universal best-fit equation for different types of weak rocks between f_d side resistance at displacement equal to 0.01 D (f_d = f_{max} for very hard claystone and sandstone bedrocks) and E_m is obtained (Table 5.5) as: f_d= 0.0018 E_m (not 0.0016 E_m shown in Figure 5.4 because the least-square-fitting put more weight on the large values). More accurate fitting could be obtained by taking f_d= 0.0019 E_m for the soil-like claystone and f_d= f_{max}= 0.0017 E_m for the very hard claystone and sandstone.

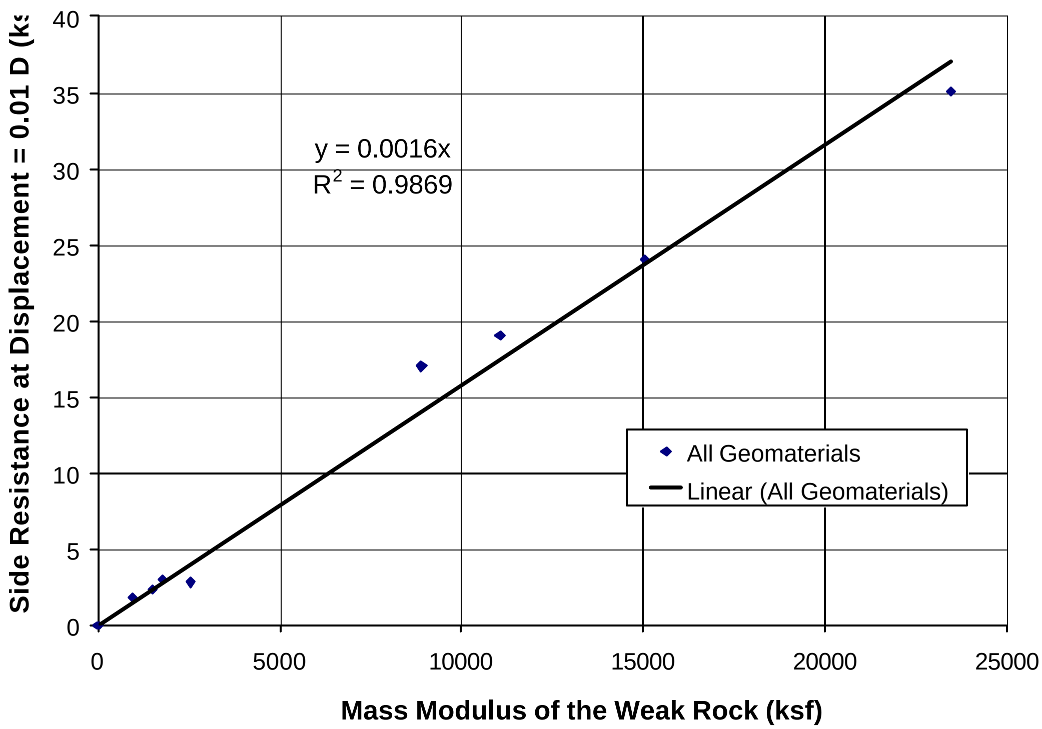


Figure 5.4. Ultimate Side Resistance at Displacement= 0.01 D vs. Elastic Modulus for Soil-Like Claystone, Very Hard Sandy Claystone, and Very Hard Clayey Sandstone

5.5. Evaluation of the Colorado SPT-Based (CSB) Design Method

Table 5.6 lists the measured f_{max} and q_{max} values and the values predicted with the Colorado SPT-based (CSB) design method (described in Chapter 2), together with the calculated factor of safety (FS). The FS was calculated as the measured ultimate resistance value from the load test divided by the predicted allowable resistance value from the CSB method. This FS should be equal to or larger than 2. This factor of safety is called "apparent" because the applied CSB design method by Colorado geotechnical engineers is often more conservative than what is assumed in the calculations of the FS in Table 5.6. Table 5.5 provides the best-fit design equations to predict the allowable and ultimate side and base resistance values for soil-like claystone that are based on SPT N values. These design equations are the basis for what is called in this study the “Updated Colorado SPT-Based (UCSB) Design Method." The FS values associated with this method are also listed in Table 5.6.

The results in Table 5.6 reinforce early findings of Turner et al. (1993) and Attwooll (2002). The conclusions are:

- There is a large difference between the measured and predicted ultimate resistance values (explained in next chapter).
- The predicted allowable base resistance values for the soil-like claystone from the current CSB method (q_{all} = 0.5 N) are very close to those measured from the load tests (q_{all} = 0.46N).
- The CSB side resistance design method for the soil-like claystone resulted in “an apparent” factor of safety of less than 2 but above 1, ranging from 1.3 to 1.8. This suggests that this design method worked well with less than normal factor of safety of 2 (discussed more in next chapter).
- Analysis of load tests performed by Turner et. al. (1993) indicates very low factors of safety, in the range of 0.8 to 1.6, which were not observed in this study. Turner et. al. assumed f_{max} to correspond to displacements of 0.5 inches, but in the current study f_{max} for soil-like claystone corresponds to (or are very close to) the true side resistance, which occurs at greater displacements (see Chapter 3). This suggests that Turner et. al. underestimated the factors of safety.
- The factor of safety associated with the “Updated Colorado SPT-Based (UCSB) Method”, is approximately 2, and it is higher than the FS for the CSB design method currently employed in Colorado. Other AASHTO and FHWA design equations presented in the next section for the soil-like claystone employ higher factors of safety, ranging from 2.3 to 3. However, it is recommended to use the UCSB design method for reasons presented in the next chapter.
- On the other hand, the CSB design method is very conservative when drilled pier sockets are constructed in the very hard claystone/sandstone formations (i. e., the “Denver Blue”). This method results in apparent FS ranging from 3.4 to 7.
- The R-squared value for the SPT-design method was 0.93 (Figure 5.3), but it was 0.99 (Figure 5.2) for the strength-based design method. This suggests that the SPT based design

methods for side resistance in claystone are not as accurate as the strength-based design methods (this is assuming that reliable strength data can be obtained). However, for soil-like claystone, where it is very difficult to collect reliable samples for strength testing, the UC strength values for use in a strength-based design method can be estimated from an improved correlation between strength data and SPT N-value (i.e., q_{ui} (ksf)=0.24 N). Alternatively, SPT-based design methods can be used directly, as recommended in the UCSB method.

Table 5.6. Assessment of Colorado SPT-Based (CSB) Design Method

	Soil-Like Claystone (I-25@ I-225 and County Line)				Very Hard Sandy Claystone (I-25@ Franklin)	Very Hard Clayey Sandstone (I-25@ Broadway)		
Side Resistance								
SPT- N value (bpf)	32	55	41	38	>100	>100	>100	>100
Measured f_{max} (ksf)	2.6	3.6	3.1	3.4	19	17	35.1	24
CSB Method, f_{max} (ksf)	4.8	8.3	6.1	5.7	15	15	15	15
Measured f_{all} (ksf)	1.3	1.8	1.5	1.7	9.5	8.5	17.5	12
CSB Method, f_{all} (ksf)	1.6	2.7	2.0	1.9	5	5	5	5
Factor of Safety (Measured f_{max}/Estimated f_{all})	1.6	1.3	1.6	1.8	3.8	3.4	7	4.8
Updated CSB Method, f_{all} (ksf)	1.2	2.0	1.5	1.4	N/A			
Factor of Safety	2.2	1.8	2.1	2.4				
Base Resistance								
SPT- N Value (bpf)	58		61		>100	>100		
Measured q_{max} (ksf)	55		53		236	318		
CSB Method, q_{max} (ksf)	87		90		150	150		
Measured $q_{all,}$ (ksf)	27		27		118	159		
CSB Method, q_{all} (ksf)	29		30		50	50		
Factor of Safety (Measured f_{max}/Estimated f_{all})	1.9		1.8		4.7	6.4		
Updated CSB Method, q_{all} (ksf)	26.7		28.1		N/A			
Factor of Safety	2.1		1.9					

5.6 Evaluation of the FHWA and AASHTO Design Methods

In this section, f_{max} and q_{max} for test shafts are predicted from the FHWA and AASHTO design methods described in Chapter 2, using the measured results of simple geotechnical tests, and are compared with f_{max} and q_{max} measured from the O-Cell load tests.

The presence of discontinuities in the rock mass reduces the mass strength of the rock mass as discussed in Chapters 1 and 2. For the soil-like claystone, the influence of discontinuities is not important, as these materials will be analyzed as soils. The presence of discontinuities in the bedrock at Broadway and Franklin sites, if any, is expected to very small for the following reasons:

- The high percentages of RQD reported for the Broadway and Franklin bedrock (80% to 100%, Table 5.2).
- The high ratios of (E_m/E_i) seen in Table 5.2.
- The strength values predicted from PM test (q_{um}), performed on the rock mass, and from UC tests, performed on intact cores (q_{ui}), were very close.
- The analysis presented in this section provides good agreement between the measured resistance values and the predicted resistance values from methods that do not assume the presence of discontinuities in the rock mass.
- The study will develop/select design equations that best fit Colorado specific geological conditions. So, if discontinuities are present in the rock mass, their influence will be accounted for in the developed design equations.

Therefore, it is assumed in the design methods that no reduction is needed for the strength data (q_{ui}) obtained from laboratory testing or the estimated resistance values due to the presence of discontinuities in the rock mass. It was assumed that the rock is intact in the side resistance design methods, and massive rock with an insignificant number of joints or closed joints spaced more than 10 ft apart for the base resistance design methods.

5.5.1 Side Resistance Design Methods

Table 5.7 lists the measured ultimate side resistance values and the predicted values from the FHWA and AASHTO design methods.

For the Denver weathered claystone formations with unconfined compressive strength less than 12 ksf (called in this study "soil-like claystone"), Turner et al. (1993) recommended the use of Eq. 2.5 to predict f_{max}. The results shown in Table 5.7 support this finding as better predictions are generated from this equation than from the AASHTO equation for stiff cohesive soils (Eq. 2.4). Surprisingly, the Horvath and Kenny method for smooth rock sockets generated excellent predictions for f_{max}. The recommended resistance factor for the Horvath and Kenny method is 0.65 (equivalent approximate factor of safety is FS= 1.5/0.65 = 2.3). As more data become available, the accuracy of these two methods can be improved so that the factor of safety can be reduced to 2, which would allow this method to compete with the UCSB design method recommended at the present time for soil-like claystone.

A rough-sided rock socket could have three times the side resistance capacity of a smooth-sided rock socket -side resistance is strongly influenced by socket roughness. *Therefore, it is important to perform a roughness test in addition to performing SPT, UCT, and PMT tests.* Rough sockets could be achieved through normal drilling or artificially by the use of shear rings (Chapter 2). The Carter and Kulhawy method (Eq. 2.10) allows for three levels of roughness (using units of ksf, see Chapter 3 and Table 5.7): μ=0.92 for smooth socket (close to the Horvath and Kenny method), 2.05 for intermediate roughness level under normal drilling, or 2.7 for high roughness level under normal or artificial drilling. The O'Neill et al. method (1996) allows for smooth or rough socket but the level of roughness is not accounted for in the analysis. The O'Neill method was developed for roughness pattern as observed in auger-cut clay-shales, expected to be close to the intermediate roughness level of Carter and Kulhawy.

For rocks with UC strength larger than 12 ksf, Turner et al. (1993) recommended the use of the Horvath and Kenny method (Eq. 2.8). The results in Table 5.7 do not support this recommendation for the hard claystone and sandstone of the Franklin and Broadway shafts. The Horvath and Kenny method, as discussed in Chapter 3, was recommended for rocks that drill very smooth or when the rock is drilled under slurry. It is clear from Table 5.7 that the f_{max} predictions from the Horvath and Kenny (FHWA, 1999) and the O'Neill et al. (1996) methods for smooth socket are also much smaller than the measured values. Better predictions were obtained from methods that assume a rough bedrock socket. Table 5.7 results seem to suggest an intermediate roughness level for the Broadway very hard sandstone and a high roughness level for the Franklin very hard claystone. This roughness was generated under normal drilling procedures. Indeed, the driller of the Franklin test shaft indicated that normal drilling in the blocky bedrock of the Franklin site would create a very rough socket. This also could explain the early match in the f-w curves measured from the load tests on the Franklin and Broadway test shafts (Figure 4.22), although the Broadway bedrock is almost three times stronger than Franklin's bedrock. Because the Carter and Kulhawy design method was endorsed by AASHTO LRFD (1998) with a resistance factor of 0.55, the use of this method with intermediate roughness level is recommended for both the very hard claystone and sandstone.

5.5.2 Base Resistance Design Methods

Table 5.8 lists the measured ultimate base resistance values and the predicted values from the FHWA and AASHTO design methods.

Table 5.8 suggests that the predictions for q_{max} for the same material vary significantly among different methods. The primary reason is in the definition of the ultimate base resistance as presented in Table 5.8 and discussed in Chapters 2 and 3. For example, for the Broadway test shaft, if the one-inch displacement criterion is selected to define ultimate resistance as suggested in the FHWA manual (1999), q_{max} would be 128 ksf. If the 0.05D displacement criterion is selected, q_{max} would be 318 ksf, while the true base resistance for massive cohesive rock could be higher than 900 ksf (as estimated using q_{max}=4.5 q_{ui}). The use of q_{max}= 2.5$q_{ui,}$ in which ultimate resistance corresponds to the occurrence of fracturing in the rocks (not true resistance), will underestimate the measured q_{max} for the Franklin shaft and overestimate the measured q_{max} for the Broadway shaft. Closer predictions are obtained when methods with the same definition of ultimate resistance are grouped together as seen in Table 5.8. The study definitions of ultimate resistance for different geomaterials are discussed in Chapter 3.

Table 5.7. Measured Unit Ultimate Side Resistance Values and Predicted Values from the FHWA and AASHTO Design Methods (all units are in ksf)

	Soil-Like Claystone (I-25@ I-225 and County Line)				Very Hard Sandy Claystone (I-25@ Franklin)	Very Hard Clayey Sandstone (I-25@ Broadway)		
q_{ui} (ksf)	8.3	13.1	10	10.4	64	97	210	145
Measured f_{max} (ksf)	2.6	3.6	3.1	3.4	19	17	35.1	24
Predicted from (ksf):								
AASHTO (1998) Equation (Eq. 2.4) for Stiff Cohesive Soils (ksf).	2.1	2.5	2.3	2.4				
Equation (Eq. 2.5) recommended by Turner et al. (1993) for stiff Cohesive Soils (ksf).	2.8	3.8	3.2	3.1				
Horvath and Kenny Method (Eq. 2.8) for smooth rock sockets recommended in AASHTO 1998 with resistance factor of 0.65.	2.7	3.4	3.0	3.1	7.6	9.4	13.8	11.4
Carter and Kulhawy Method (Eq. 2.10) recommended in AASHTO (1998) with resistance factor of 0.55								
1. For smooth socket (close to the Horvath and Kenny method), μ=0.92.					7.5	9.1	13.4	5.1
2. For regular clean rock sockets with grooves between 0.04" and 0.4" occurred under normal drilling, μ = 2.05.					16.7	20.2	29.7	24.7
3. If the socket is rough, either through normal drilling or artificially, μ = 2.75.					22.3	27.1	39.9	33.1
O'Neill et al. Method (1996) for Cohesive IGM:								
1. Smooth (Eq. 2.6).					5.9	7.8	16.8	11.6
2. Rough Socket at displacements equal to 1% the shaft diameter.					12.8	14	32.8	21.8

Table 5.8. Measured Unit Ultimate Base Resistance Values and Predicted Values from the FHWA and AASHTO Design Methods (all units are in ksf)

	Soil-Like Claystone (I-25@ I-225 and County Line)		Very Hard Sandy Claystone (I-25@ Franklin)	Very Hard Clayey Sandstone (I-25@ Broadway)
q_{ui} (ksf)	13.1	16.8	41	219
Measured q_{max}, ksf	55	53	236	318
Predicted from (ksf)				
A. Methods where q_{max} corresponds or very close to the full plastic mobilization of the resistance (theoretical ultimate resistance).				
1. $q_{max} = 4.5\ q_{ui}$ (Eq. 2.11 for very stiff Clays) with resistance factor of 0.55 (AASHTO 1998). AASHTO (2002) recommends 4.3 q_{ui} for intact claystone and sandstone, and 5 q_{ui} for intact sandstone.	59	75.6	185	986
2. Canadian method for Rocks endorsed by AASHTO (1998) with Resistance Factor of 0.5. $q_{max} = 4.08\ q_{ui}$ for massive rocks (or joints are closed and spaced more than 10 ft) with L/D>6	45	48	167	893
B. Method for rocks where q_{max} correspond to the occurrence of fracturing in the rock				
$q_{max} = 2.5\ q_{ui}$ (FHWA Method for massive and cohesive IGM or rock, Eq. 2.12)	33	42	103	547
C. Methods where q_{max} is defined by displacement criteria.				
O'Neill et al. (1996) Method for Cohesive IGM at displacements equal to 5% the shaft diameter.			145	271
Zhang and Einstein method (Eq. 2.15), q_{max} (ksf) = $21.4\ (q_{ui})^{0.51}$			142	334

For the soil-like claystone (I-225, County Line) the measured q_{max} listed in Table 5.8 corresponds to the development of most of the base resistance. Therefore, the predictions of q_{max} from the AASHTO recommended design equation for stiff clay ($q_{max} = 4.5\ q_{ui}$, Table 5.8) provide reasonable predictions of the measured resistance values. The measured q_{max} and stiffness of the q-w curve at the I-225 shaft was slightly larger than that for the County Line shaft (Figure 4.15), although the bedrock beneath the County Line shaft was stronger (Table 5.8). Also, the predicted

q_{max} for the County Line shaft from q_{max}= 4.5 q_{ui} (76 ksf) was much larger than the measured value (53 ksf). These observations can be explained with the following:

- The load test results were collected at the end of the test, that were close to, but did not correspond to, the true theoretical base resistance when the entire plastic resistance is mobilized (definition of failure for soil-like claystone). To be conservative, extrapolation of the load test results was avoided.
- A limited amount of strength data for the two shafts was available. Better predictions could be obtained at the County Line shaft if the lower strength (14.8 ksf), not the average (16.8 ksf), are used in the analysis. For layers with varying strength within 2D below the base, AASHTO recommended for design purposes considering the layer with the lower strength value.
- Presence of discontinuities in the rock mass at the County Line Site.
- Layout of the shafts. Diameter and length of the rock socket was 4 ft and 14 ft for the County Line shaft (L/D= 3.5), and 3.5 ft and 16.1 ft for the I-225 shaft (L/D= 4.6). In the Canadian design method, q_{max} increases with the increase of L/D until L/D reaches a ratio of 6, so better predictions for q_{max} at the two shafts were obtained with this method
- Presence of a hard sandstone layer within the influence area of the base of the I-225 shaft, which might influence its base capacity (Layer 3, see section 4.1.1).
- Most of the rock socket for the I-225 shaft exists under the ground water level, but the GWL is located well below the base of the County Line shaft.
- The I-225 site is located within the Denver-Arapahoe (Undifferentiated) Formation, and the County Line Road test site is located at the northern margin of the Dawson Formation.

Better and conservative predictions for the maximum base resistance for the soil-like claystone can be obtained from the best-fit design equation of q_{max}=3.8 q_{ui}. With q_{ui} (ksf)=0.24 N, this equation is equivalent to using q_{max} (ksf) =0.92 N. Until more data become available, it is recommended in this study to use the updated Colorado SPT-based design method for the soil-like claystone.

The measured values of q_{ui} and E_m beneath the Franklin shaft were smaller than the results of the load test results suggest (41 ksf <236/4.5). These could be attributed to the limited number of strength and stiffness data (just one data point). As presented in Chapter 3, the geotechnical engineering consultant for the construction project reported higher q_{ui} than was obtained in this study, and also indicated that the rock in the Franklin site varies horizontally and vertically, and that weaker rock could be encountered anywhere. This is a typical condition for IGM and soft rock in Texas. The Canadian method (q_{max}= 4.08 q_{ui} for rocks with joints closed and spaced more than 10 ft and L/D>6, see Chapter 3) seems to provide reasonable and conservative predictions of the measured base resistance values at the Franklin site (Table 5.8). This method is endorsed in AASHTO LRFD with resistance factor of 0.5. Therefore, this design method is recommended for the very hard claystone.

For the Broadway shaft, the measured q-w curve for the very hard sandstone was linear with no signs of yielding up to a settlement that corresponded to 5% the shaft diameter. For this shaft, the displacement criterion of 0.05D was selected to define the ultimate base resistance (Chapter 3), resulting in q_{max} = 1.45 q_{ui}.. The q_{max} for this shaft was well predicted with the O'Neill et al. method (1996) and the Zhang and Einstein method (Eq. 2.15, Table 5.8). No resistance factors are available for these methods, and the O'Neill method is more difficult to use than the Zhang and Einstein method. Therefore, for the very hard sandstone, it is recommended to use a conservative version of Zhang and Einstein design method (with units of ksf) as q_{max}= 17 $(q_{ui})^{0.5}$, with a conservative low resistance factor of 0.5 (or FS=3.0). The allowable design base resistance with this approach for the Broadway shaft is 84 ksf, much more than the 50 ksf assumed with the CSB design method.

The O'Neill et. al. (1996) method is an advanced design method that predicts f-w and q-w curves as a function of L, L/D, E_m, (E_c/E_m), q_{ui}, and other parameters. This method was programmed in an Excel worksheet called CDOTSHAFT. Predictions of this method for the base and side resistance values at displacements that correspond to the study definition of ultimate resistance values are shown in Tables 5.7 and 5.8. Figure 5.5 shows the measured q-w curve for the Broadway shaft, and the predicted curve from that method. Reasonable agreement can be observed

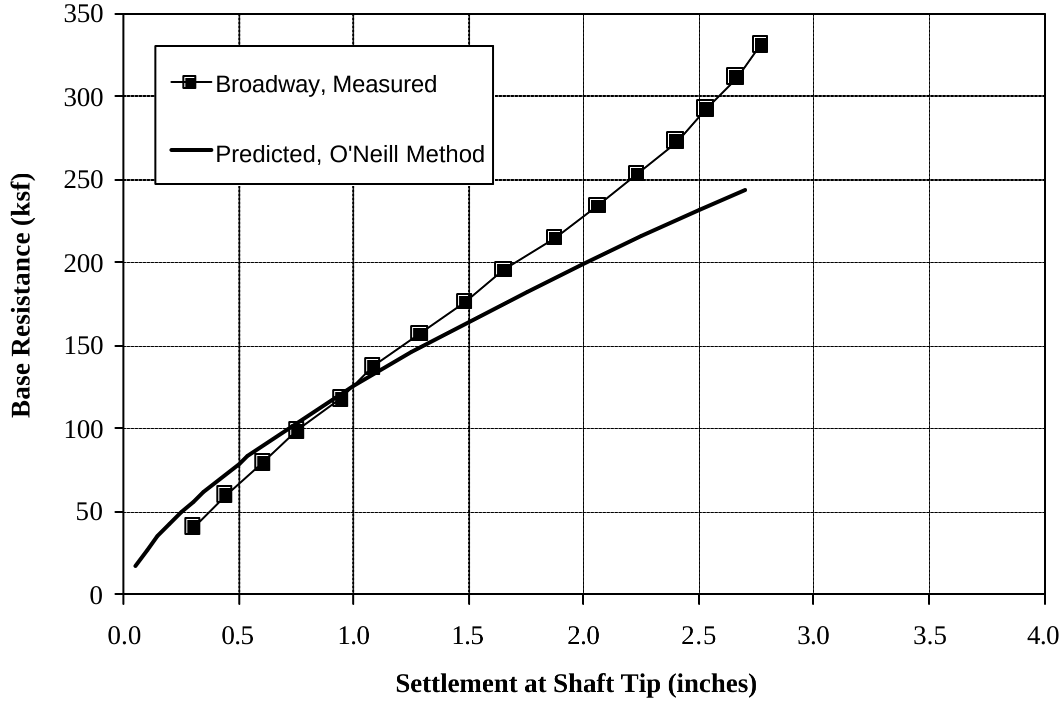


Figure 5.5. Measured and Predicted Base Resistance-Settlement for the Broadway Test Shaft Using O'Neill et al. (1996) Method

6. STUDY FINDINGS AND RECOMMENDATIONS

6.1 Overview

Tables 6.1 through 6.4, respectively, provide a description of foundation bedrock, construction, materials, layout, and results of all geotechnical tests (SPT, UC, PM, and O-Cell load tests) for the I-225, County Line, Franklin, and Broadway test shafts (see Appendix A for definitions of all terms). CDOT geotechnical standards describe the bedrock for the I-225 and County Line test shafts according to their standard penetration resistance (N in units of bpf) as firm (20<N<30), or medium hard (30<N<50), or hard (50<N<80). CDOT standards describe the bedrock for the Franklin and Broadway test shafts as very hard (N>80). The unconfined compressive (UC) strength of intact core specimens (q_{ui}) ranges from 8 to 16 ksf for the claystone of the I-225 and County Line shafts, 40 to 80 ksf for the Franklin claystone, and 85 to 300 ksf for the Broadway sandstone. Hence, the County Line and I-225 bedrock can be classified as very stiff clays according to AASHTO (1998) and CFEM (1992) manuals, and at the boundary area between stiff clays and cohesive intermediate geomaterials (IGM's) according to the FHWA (1999) manual. Therefore, design methods for stiff clays are appropriate for these weathered rocks. The Franklin and Broadway bedrock is classified as rock according to AASHTO and CFEM. The FHWA manual classifies the Franklin bedrock as cohesive IGM's, and the Broadway bedrock as rock. In this study, the bedrock at the County Line and I-225 sites is referred to as "soil-like claystone," the Franklin bedrock as "very hard sandy claystone bedrock," and the Broadway bedrock as "very hard clayey sandstone bedrock". Overburden in this study is defined as soils having SPT<20 for clay-based soils or SPT<50 for sand-based soils, and rocks otherwise.

This chapter presents:

- ❑ Major findings of the study, including: 1) best correlation equations between results of various simple geotechnical tests, and best-fit design equation to predict the ultimate unit side resistance (f_{max}), ultimate unit base resistance (q_{max}), and the load-settlement curve (Table 5.5) as a function of the results of simple geotechnical tests; 2) assessment of the Colorado SPT-Based (CSB) design method (Table 5.6); and 3) assessment of AASHTO and FHWA design methods (Tables 5.7 and 5.8).

- Recommended design methods with resistance factors (ϕ) and factors of safety (FS) for drilled shafts with conditions close to those at the four load test sites. All the qualifications and limitations for using these design recommendations are also listed. Other design recommendations are also presented.

- Assessment and Recommendations for improvement of the geotechnical subsurface investigation procedures in Colorado (SPT, sampling procedure, UC, and PM tests, definition of adequate investigation).

Table 6.1. Description of the Foundation Bedrock, Construction, Materials, Layout, and Results of all Geotechnical Tests for the I-225 Test Shaft

O-Cell Load Test Date and Location: 1/8/02; Denver, Intersection of I-225 and I-25**.**						
Excavation Method and Time: Auger drilling with no use of water, slurry, or casing; ~3 hours						
Conditions of Shaft Wall Sides and Bottom: Any smear from the sides of the borehole (lower 8 ft) was removed; considered smooth, dry sides and bottom; and cleaned hole.						
Concrete Placement Method and Time: Slowly by tremie pipe; ~ 2 hours						
Concrete Slump: 9"	**f'$_c$:** 3423 psi		**E$_c$:** 530000 ksf			
Top Elevation of Ground 5644 ft	**Depth from Ground to Top of Shaft, GWL, Top of Competent Rock, and Base of Shaft:** 6, 15.5, 12.5, and 28.6 ft.		**D=** 3.5 ft	**L =** 16.1 ft (extends 0.8 ft beneath O-Cell)		
Geotechnical and Geological Description of Weak Rock: soil-like claystone bedrock. This weathered and sedimentary claystone behaves more like very stiff to hard clay than "rock". The site is located within the Denver-Arapahoe Rock Formation.						
Test Results For the Weak Rock around Sides of the Test Socket						
Depth: from to (ft)	f_{max} (ksf)	f_{all} (ksf)	f_d (ksf)	SPT-N (bpf)	q_{ui} (ksf)	E_m (ksf)
15.8 to 21.8	2.6	1.3	1.8	32	8.3	970
21.8 to 27.8	3.6	1.8	2.8	55	12.3	2550
Socket: 12.5 to 27.8	3.1	1.6	2.3	41	10	1513
Test Results For the Weak Rock Beneath the Test Socket						
Depth (ft)	q_{max} (ksf)	q_{all} (ksf)	q_d (ksf)	SPT-N (bpf)	q_{ui} (ksf)	E_m (ksf)
28.6	55	27	27	58	13.1	2550
Test Results For the Test Shaft						
Q_{max}= 1078 kips, Q_d= 662 kips, Q_{all} = 539 kips (70% from side resistance), w_{all} = 0.24"						

Table 6.2. Description of the Foundation Bedrock, Construction, Materials, Layout, and Results of all Geotechnical Tests for the County Line Test Shaft

<table>
<tr><td colspan="7">O-Cell Load Test Date and Location: 1/8/02; Denver, between I-25 and the exit from SB I-25 to County Line Road.</td></tr>
<tr><td colspan="7">Excavation Method and Time: Auger drilling with no use of water, slurry, or casing; ~3 hours</td></tr>
<tr><td colspan="7">Conditions of Shaft Wall Sides and Bottom: Any smear from the sides of the borehole (lower 8 ft) was removed; considered smooth, dry sides and bottom; and cleaned hole.</td></tr>
<tr><td colspan="7"></td></tr>
<tr><td colspan="7">Concrete Placement Method and Time: Slowly by tremie pipe; 2 hours</td></tr>
<tr><td colspan="2">Concrete Slump: 9"</td><td colspan="3">f'c: 3193 psi</td><td colspan="2">E_c: 500000 ksf</td></tr>
<tr><td colspan="7"></td></tr>
<tr><td colspan="2">Top Elevation of Ground
5886 ft</td><td colspan="3">Depth from Ground to Top of Shaft, GWL, Top of Competent Rock, and Base of Shaft:
6, not encountered, 8, and 22 ft.</td><td>D=
4 ft</td><td>L = 14 ft
(extends 0.5 ft beneath O-Cell)</td></tr>
<tr><td colspan="7"></td></tr>
<tr><td colspan="7">Geotechnical and Geological Description of Weak Rock: soil-like claystone bedrock. This weathered and sedimentary claystone behaves more like very stiff to hard clay than “rock”. The site is located at the northern margin of the Dawson Formation.</td></tr>
<tr><td colspan="7">Test Results For the Weak Rock around Sides of the Test Socket</td></tr>
<tr><td>Depth: from to (ft)</td><td>f_{max} (ksf)</td><td>f_{all} (ksf)</td><td>f_d (ksf)</td><td>SPT-N (bpf)</td><td>q_{ui} (ksf)</td><td>E_m (ksf)</td></tr>
<tr><td>Socket: 8 to 21.5</td><td>3.4</td><td>1.7</td><td>3</td><td>38</td><td>10.4</td><td>1800</td></tr>
<tr><td colspan="7">Test Results For the Weak Rock Beneath the Test Socket</td></tr>
<tr><td>Depth below (ft)</td><td>q_{max} (ksf)</td><td>q_{all} (ksf)</td><td>q_d (ksf)</td><td>SPT-N (bpf)</td><td>q_{ui} (ksf)</td><td>E_m (ksf)</td></tr>
<tr><td>22</td><td>53</td><td>27</td><td>22</td><td>61</td><td>16.8</td><td>3200</td></tr>
<tr><td colspan="7">Test Results For the Test Shaft</td></tr>
<tr><td colspan="7">Q_{max}= 1340 kips, Q_d= 876 kips, Q_{all} = 670 kips (76% from side resistance), w_{all} = 0.25”</td></tr>
</table>

Table 6.3. Description of the Foundation Bedrock, Construction, Materials, Layout, and Results of all Geotechnical Tests for the Franklin Test Shaft

<table>
<tr><td colspan="7">O-Cell Load Test Date and Location: 1/11/02; Denver, along 2nd pier column from south abutment beneath the west column of the Franklin Bridge over I-25.</td></tr>
<tr><td colspan="7">Excavation Method and Time: Auger drilling with use of slurry and casing only in the overburden; ~ 5 hours</td></tr>
<tr><td colspan="7">Conditions of Shaft Wall Sides and Bottom: Roughened-sided socket is expected with normal drilling procedure; wet sides as at least 18" of groundwater infiltrated through sides; cleaned hole.</td></tr>
<tr><td colspan="7"></td></tr>
<tr><td colspan="7">Concrete Placement Method and Time: Slowly by tremie pipe; 3 hours</td></tr>
<tr><td colspan="2">Concrete Slump: 8"</td><td colspan="3">f'_c: 3410 psi</td><td colspan="2">E_c: 530000 ksf</td></tr>
<tr><td colspan="7"></td></tr>
<tr><td>Top Elevation of Ground Surface
5296 ft</td><td colspan="3">Depth from Ground to Top of Shaft, GWL, Top of Competent Rock, and Base of Shaft:
0, 4, 4.5, and 25.3 ft.</td><td>D=
3.5 ft</td><td colspan="2">L = 20.8 ft
(extends 1.8 ft beneath O-Cell)</td></tr>
<tr><td colspan="7">Geotechnical and Geological Description of Weak Rock: very hard, mostly thinly bedded, blue and sandy claystone bedrock. The site is located within the Denver-Arapahoe Rock Formation.</td></tr>
<tr><td colspan="7">Test Results For the Weak Rock around Sides of the Test Socket</td></tr>
<tr><td>Depth: from to (ft)</td><td>f_{max} (ksf)</td><td>f_{all} (ksf)</td><td>f_d (ksf)</td><td>SPT-N (bpf)</td><td>q_{ui} (ksf)</td><td>E_m (ksf)</td></tr>
<tr><td>Socket: 4.5 to 23.7</td><td>19</td><td>8.5</td><td>19</td><td>50/4"</td><td>64</td><td>11050</td></tr>
<tr><td colspan="7">Test Results For the Weak Rock Beneath the Test Socket</td></tr>
<tr><td>Depth below (ft)</td><td>q_{max} (ksf)</td><td>q_{all} (ksf)</td><td>q_d (ksf)</td><td>SPT-N (bpf)</td><td>q_{ui} (ksf)</td><td>E_m (ksf)</td></tr>
<tr><td>25.3</td><td>236</td><td>118</td><td>71</td><td>50/5"</td><td>41</td><td>4700*</td></tr>
<tr><td colspan="7">Test Results For the Test Shaft</td></tr>
<tr><td colspan="7">Q_{max}= 6612 kips, Q_d= 5024 kips, Q_{all} = 3306 kips (90 % from side resistance), w_{all} = 0.2".</td></tr>
</table>

Table 6.4. Description of the Foundation Bedrock, Construction, Materials, Layout, and Results of all Geotechnical Tests for the Broadway Test Shaft

<table>
<tr><td colspan="7">O-Cell Load Test Date and Location: 1/10/02; Denver, Broadway Viaduct over I-25, along bent 6 (6th bent from west abutment) beneath the center column.</td></tr>
<tr><td colspan="7">Excavation Method and Time: Auger Drilling use of slurry and casing only in the overburden, ~ 7 hours</td></tr>
<tr><td colspan="7">Conditions of Shaft Wall Sides and Bottom: Roughened-sided socket is expected with normal drilling procedure; dry hole.</td></tr>
<tr><td colspan="7"></td></tr>
<tr><td colspan="7">Concrete Placement Method and Time: Slowly by tremie pipe; 4 hours</td></tr>
<tr><td colspan="2">Concrete Slump: 7.5”</td><td colspan="3">f'_c: 3936 psi</td><td colspan="2">E_c: 580000 ksf</td></tr>
<tr><td colspan="7"></td></tr>
<tr><td colspan="2">Top Elevation of Ground Surface
5255 ft</td><td colspan="3">Depth from Ground to Top of Shaft, GWL, Top of Competent Rock, and Base of Shaft:
6.5, 17.1., 17, and 47.1 ft.</td><td>D=
4.5 ft</td><td>L = 30.1ft
(extends 6.3 ft beneath O-Cell)</td></tr>
<tr><td colspan="7">Geotechnical and Geological Description of Weak Rock: predominately very hard, well-cemented, blue, and clayey sandstone. The site is located within the Denver-Arapahoe Rock Formation.</td></tr>
<tr><td colspan="7">Test Results For the Weak Rock around Sides of the Test Socket</td></tr>
<tr><td>Depth:
From to (ft)</td><td>f_{max}
(ksf)</td><td>f_{all}
(ksf)</td><td>f_d
(ksf)</td><td>SPT-N
(bpf)</td><td>q_{ui} (ksf)</td><td>E_m
(ksf)</td></tr>
<tr><td>20.8-30.8</td><td>17</td><td>8.5</td><td>15.9</td><td>50/3”</td><td>97</td><td>8900</td></tr>
<tr><td>30.8-40.8</td><td>35.1</td><td>17.5</td><td>35.1</td><td>**50/2”</td><td>210</td><td>23448</td></tr>
<tr><td>Socket: 17 to 40.8</td><td>24</td><td>12</td><td>24</td><td>**50/2.5”</td><td>145</td><td>15025</td></tr>
<tr><td colspan="7">Test Results For the Weak Rock Beneath the Test Socket</td></tr>
<tr><td>Depth below
(ft)</td><td>q_{max}
(ksf)</td><td>q_{all}
(ksf)</td><td>q_d
(ksf)</td><td>SPT-N
(bpf)</td><td>q_{ui}
(ksf)</td><td>E_m
(ksf)</td></tr>
<tr><td>47.1 ft</td><td>318</td><td>159</td><td>71</td><td>*83/6”</td><td>219</td><td>21900***</td></tr>
<tr><td colspan="7">Test Results For the Test Shaft</td></tr>
<tr><td colspan="7">Q_{max}= 15276 kips, Q_d= 11362 kips, Q_{all} = 7638 kips (95 % from side resistance), w_{all} = 0.5".</td></tr>
</table>

*SPT terminated in the first 6-inches penetration interval. ** Roughly estimated data based on the results of Table 5.2. *** Estimated based on Results of UC tests.

6.2 Study Findings

The findings listed in the following are valid for drilled shafts with conditions (type of weak rock, layout, construction, and materials of test shafts) close to those described in this study at the four load test sites (Tables 6.1 to 6.4). In addition, these findings are based on a limited amount of data, and therefore should be considered with caution. More data are needed to confirm/refine these findings, and the range, in terms of SPT-N and unconfined compressive strength, for which these findings are valid.

6.2.1 Best-Fit Design Equations

- The best conservative correlation equation between unconfined compressive strength of intact core specimens (q_{ui}, ksf) and the SPT-N value (bpf) for soil-like claystone bedrock is

  q_{ui} (ksf) =0.24 N 6.1

  Note that the FHWA manual (1999) recommends q_{ui} (ksf) =0.27 N.

- The initial modulus measured from the PM test (E) was selected over the reload modulus (E_r) to represent the mass Young's modulus of the rock mass (E_m), for many reasons listed in Section 5.4.

- For indirect estimation of the unconfined compressive strength from PM test results, the most appropriate equation for soil-like claystone is (units in ksf)

  $q_{ui} = 0.5 \ (P_l - P_o)^{0.75}$, ...6.2

and for the hard claystone and sandstone bedrock is (units in ksf)

$(P_l - P_o)/S_u = 1 + \text{Ln}\ (E_m/(2.67S_u))$ 6.3

where $S_u = 0.5\ q_{ui}$ and S_u is the undrained shear strength of the rock.

- It was very difficult to collect reliable core specimens for strength tests in the soil-like claystone and in the fractured rock. The general experience in the Colorado Front Range is that a small percentage of actual UC strength results may approach the true mass strength of the rock formation (i.e., UC test can be unreliable). The SPT and PMT produced more consistent strength data for the soil-like claystone and fractured rock and could be made very reliable if the correlation equations suggested above are refined and made more accurate. Reliable cores were recovered from the very hard claystone and sandstone bedrock, leading to reliable UC strength data in these rocks.

- Use a conservative ratio between the mass stiffness of the rock and unconfined compressive of intact rock (E_m/q_{ui}) equal to 150 for claystone bedrock and 100 for the hard sandstone bedrock. For the soil-like claystone, the ratio (E_m (ksf)/N) can be taken as 40. For all materials, a best fit expression can be taken as E_m (ksf) = 1016 $q_{ui}^{0.5}$ and for design purposes as E_m (ksf) = 600 $q_{ui}^{0.5}$.

- Best-fit design equations to predict ultimate side resistance (f_{max}) and ultimate base resistance (q_{max}) of different types of bedrock as a function of the SPT test results (N-values with units of bpf), and UC test results (q_{ui} with units of ksf) are listed in Table 5.5.

- A universal best-fit equation for different types of weak rocks between f_d side resistance at displacement equal to 0.01 D ($f_d = f_{max}$ for very hard claystone and sandstone bedrocks) and E_m is obtained as: f_d= 0.0018 E_m (Table 5.5). This suggests that the side resistance of rocks is very dependent on the stiffness of the rock. This finding is consistent with the findings of Seidel and Collingwood (2001), who indicated that part of the side resistance comes from at least some roughness at the interface, which in turn produces dilation when the shaft is loaded. Dilation increases effective stresses and therefore the shear strength at the interface, since the rock at the interface essentially behaves as a drained material. The normal effective stresses due to dilation are directly proportional to the normal stiffness of the rock.

- Shown also in Table 5.5 is the procedure to construct an approximate load (kips)-settlement (inches) curve as a function of the SPT-N values (bpf) and the unconfined compressive strength (q_{ui}, with units of ksf) for all weak rocks investigated in this study.

- The settlements of all shafts that correspond to the design loads (w_{all}) ranged from 0.25 inch for the County Line and I-225 shafts to 0.5 inch for the Broadway shaft. These were smaller than the tolerable settlement of 0.65 inch, suggesting that the design loads calculated based on the Strength Limit control the design.

- In the soil-like claystone at County Line and I-225, 70% of the resistance to working loads was provided by side resistance. In the very hard claystone and sandstone at the Franklin and Broadway sites, 95% to 100% of the resistance at working loads was provided by side resistance.

6.2.2 Assessment of Colorado SPT-Based (CSB) Design Method

Table 5.6 lists the measured f_{max} and q_{max} and the predicted values with the Colorado SPT-based (CSB) design method (described in Chapter 2), together with the factor of safety calculated as the measured ultimate resistance from the load test divided by the predicted (from CSB method) allowable resistance. This factor of safety (FS) is called “apparent” because the applied CSB design method by Colorado geotechnical engineers is often more conservative than what is assumed in the calculations of this FS.

- The CSB side resistance design method for the soil-like claystone resulted in “an apparent” factor of safety of less than 2 but above 1, ranging from 1.3 to 1.8. This suggests that this design method worked well with less than normal factor of safety of 2. The main reasons for this apparent low factor of safety not being reflected in the performance of structures that are now in service are:
  - FS for side resistance >1.3.
  - FS around 2 for base resistance.
  - The very small settlements that would occur for soil-like claystone under the design loads (expected to be less than <0.3 inch based on the results of Chapter 4).
  - The applied design loads are often overestimated and include live loads that have yet to be applied.

- There is a large difference between the measured and predicted ultimate resistance values This is because the CSB was intended to be a “safe” ASD design method with embedded factors of safety that are unknown but apparently thought by the developers to assure safety under all circumstances. Colorado engineers assumed the FS in this method to be 3 for purposes of estimating ultimate capacity values from the allowable values computed by the CSB method, which is much than the larger than the apparent factors of safety (Table 5.6). This caused the large difference between measured and predicted ultimate resistance values.

- The predicted allowable base resistance values for the soil-like claystone bedrock from the current CSB method (q_{all} = 0.5 N) are very close to those measured from the load tests (q_{all} = 0.46N).

- On the other hand, the CSB design method is very conservative when drilled pier sockets are constructed in the very hard claystone/sandstone formations (i. e., the “Denver Blue”). This method results in FS ranging from 3.4 to 7, leading to costly design and construction of high-capacity piers embedded in the competent claystone and sandstone bedrock. For these bedrocks, the use of AASHTO and FHWA strength-based design equations are appropriate and will be very cost-effective.

- Results of this study are similar to the findings for upper Cretaceous clay-shales in Texas. Design methods based on TxDOT cone tests (somewhat like the SPT) tend to produce apparent factors of safety of less than 2 in side shear when the shale is soft and soil-like. However, when it is harder and cemented, factors of safety are more like 4.

- Efforts should be undertaken to replace the use of SPT-based design methods for cohesive weak rocks and rocks with strength-based design methods. For soil-like claystone, where it is very difficult to collect reliable samples for strength testing, the UC strength values for use in a strength-based design method can be estimated from an improved correlation between strength data and SPT N-value (i.e., q_{ui} (ksf)=0.24 N). Alternatively, SPT-based design methods can be used directly, as recommended in the UCSB method, to be described later.

6.2.3 Assessment of FHWA and AASHTO Design Methods

For the soil-like claystone, the influence of discontinuities is not important, as these materials will be analyzed as soils. For the very hard claystone and sandstone bedrocks, it was demonstrated that the influence of discontinuities is very small and can be neglected. Therefore, it was assumed that the rock is intact in the side resistance design methods presented below, and the rock is massive with an insignificant number of joints or the joints are closed and spaced more than 10 ft in the base resistance design methods presented below. The same conclusion was reached about clay-shales in the Dallas area, with the exception of one unit of one formation, where soft bentonite seams were identified.

Tables 5.7 and 5.8 list the measured ultimate side and base resistance values and the predicted values from the FHWA and AASHTO design methods.

- For the soil-like claystone, the methods recommended by Turner et al. (1993) and Horvath and Kenny provide excellent predictions of f_{max}, better than the method recommended by AASHTO for stiff clays (Table 5.6).

- A rough-sided rock socket could have three times the side resistance capacity of a smooth-sided rock socket, indicating that side resistance is strongly influenced by socket roughness. *Therefore, it is very important to perform a roughness test in addition to performing SPT, UCT, and PMT tests at the sites of load tests.* Rough sockets could be achieved through normal drilling or artificially by the use of shear rings (Chapter 2). The Carter and Kulhawy method (Eq. 2.10) allows for three levels of roughness: μ=0.92 for smooth socket (close to the Horvath and Kenny method), 2.05 for intermediate roughness level under normal drilling, or 2.7 for high roughness level under normal or artificial drilling. The O'Neill et al. method (1996) allows for smooth or rough socket but the level of roughness is not accounted for in the analysis. The O'Neill method was developed for roughness pattern as observed in auger-cut clay-shales, expected to be close to the intermediate roughness level of Carter and Kulhawy.

- For the very hard claystone and sandstone bedrocks, the Carter and Kulhawy and O'Neill methods for rough sockets provide reasonable predictions of f_{max} (Table 5.6). The results in Table 5.6 suggest an intermediate roughness level for the Broadway sandstone bedrock and a high roughness level for the Franklin claystone bedrock. *This roughness for both shafts was generated under normal drilling procedures.* Indeed, the drillers of the Franklin test shaft indicated that the normal drilling in the blocky bedrock at the Franklin site created a very rough socket. This finding could explain the early match in the f-w curves measured from the load tests on the Franklin and Broadway test shafts (Figure 4.24), although the Broadway site bedrock is almost three times stronger than the Franklin site bedrock. The Carter and Kulhawy design method for side resistance is endorsed by AASHTO LRFD (1998) with a resistance factor of 0.55.

- The predictions for q_{max} vary significantly among different base resistance design methods due to differences in the definition of the ultimate base resistance in these methods (Table 5.8). For example, for the Broadway test shaft, if the one-inch displacement criterion is selected to define ultimate resistance as suggested in the FHWA manual (1999), q_{max} would be 128 ksf. If the 0.05D displacement criterion is selected, q_{max} would be 318 ksf, while the true short-term base resistance for massive cohesive rock could be higher than 900 ksf (as estimated using q_{max}=4.5 q_{ui}).

- Conservative predictions of q_{max} for the soil-like claystone can be obtained from the best-fit design equation, q_{max}=3.8 q_{ui} (Table 5.5). With q_{ui} (ksf)=0.24 N, this equation is equivalent to using q_{max} (ksf) =0.92 N as recommended in the UCSB method. These equations presume that long-term softening of the claystone will not occur.

- Conservative predictions for the maximum base resistance for the very hard claystone bedrock (Franklin site) can be obtained from the Canadian design method (Table 5.8), which is endorsed by AASHTO LRFD (1998) with resistance factor of 0.5.

- The q_{max} for the very hard sandstone bedrock was well predicted with the O'Neill method (1996) and the Zhang and Einstein (1998) method (Table 5.8). The Canadian design method or the fracture-based design equations (q_{max}=2.5 q_{ui}) are not appropriate for very hard rocks with q_{ui}>200 ksf, because ultimate base resistance should be defined based on displacement (0.05 D), not strength, in order to limit the shaft settlements at service loads.

- The O'Neill et. al. (1996) method is an advanced design method that predicts the load transfer curves and load-settlement curve as a function of L, L/D, E_m, (E_c/E_m), q_{ui}, and other parameters. This method was programmed in an Excel worksheet called CDOTSHAFT.

6.3. Recommendations for Requirements for Minimum Length of Rock Socket (L)

Some structural engineers in Colorado require a minimum embedment of shafts into the bedrock (L) of three times the shaft diameter (L=3D). This is not an AASHTO requirement, and is probably excessive from two perspectives:

- From a geological perspective, an absolute value of penetration (10 ft to 15 ft) is adequate to penetrate beyond the highly weathered material, and an absolute penetration makes more sense than requiring a certain number of diameters.

- From a mechanics perspective, if concerned about lateral loads, the use of the LPILE program would suggest that 2-diameter penetration in the competent rock is almost always sufficient.

For example, the geotechnical engineer for an upcoming construction project recommended a minimum penetration length of 10 ft into competent rock and a minimum total shaft length of 20 ft. However, the structural engineer recommended a minimum penetration length of 21 ft in the competent rock for 7-ft diameter shafts, although the bedrock encountered at that site seems to be quite competent. The L= 3D penetration requirement for large diameter shafts would control the design, resulting in excessive cost if a 10 – 15 foot penetration can be proven by load testing to be adequate.

It is recommended to select a minimum L that is based on the weathering condition of the bedrock (10' to 15'), the requirements for supporting lateral loads, and to meet any design structural requirements of the shaft rather than a multiple of the diameter or other somewhat arbitrary criteria.

6.4 Preliminary Recommendations for Design Methods

The design recommendations presented in this section are valid for drilled shafts with conditions (type of weak rock, layout, construction, and materials of test shafts) close to those described in this study at the four load test sites (Tables 6.1 to 6.4). In addition, these "preliminary" recommendations are based on a limited amount of data, and therefore should be considered with caution. More data are needed to confirm/refine the recommendations presented in this section and the range, in terms of SPT-N and unconfined compressive strength, for which these design equations are valid.

6.4.1 Limitations and Qualifications of the Recommended Design Methods

This section provides more specific details of the qualifications and limitations that should be considered before using the "preliminary" design methods (see Tables 6.1 to 6.4).

Geology

The Broadway, Franklin, and I-225 load test sites are located within the Denver-Arapahoe (Undifferentiated) Formation. The County Line Road test site is located at the northern margin of the Dawson Formation. Rock type is probably a better delineator of the engineering behavior than geologic formation, unless the depositional process or age of one formation is very different from others in the study area. The Denver Formation and Dawson formation in the south Denver metro area (Locations of I-225 and County Line) are interfingered at their boundaries, very close in age and in the depositional process, and both are classified as soft claystone with very close strength values. Therefore, it is reasonable to assume one category of weak rocks for the bedrock at County Line and I-225 sites. The design methods presented below are only applicable for the rock formations described above.

Factor of Safety and Adequate Subsurface Geotechnical Investigation

A weighted average load factor of 1.5 was assumed and used to estimate the factor of safety (FS) from the recommended resistance factor ϕ as FS=1.5/ϕ. The structural engineer should calculate the actual design load factors, based on the actual loading conditions experienced by the bridge structure, and use those to estimate FS.

The factors of safety and resistance factors of the design method are related to the extent of subsurface geotechnical investigation as will be discussed in Chapter 7. The factors of safety and resistance factors recommended in this study and in AASHTO could be applied if: 1) adequate subsurface geotechnical investigation is performed (see Section 6.5 for more details), and 2) close and appropriate quality control and quality assurance methods will be followed during construction of production shafts as performed for the test shaft, as described next.

Construction Issues:

Good construction practices for production shafts meeting the requirements of CDOT Standard specifications for drilled caissons are expected. Section 503.04 of CDOT specifications reads, " Holes shall be pumped free of water, cleaned of loose material, and inspected by the engineer." Based on this requirement, it is expected that the contractor will keep the hole dry, scrape any soft cuttings from the sides of the hole, and clean the base of the hole.

For deep shafts cleaned following the standard procedure, but where a clean bottom cannot be verified (as in very deep shafts), reduce the calculated ultimate base resistance by 20%. Alternatively, it is possible to post-grout the base of the drilled shaft. This will minimize the effects of stress relief in boreholes that might have been left open too long and of loose cuttings left on the hole. The q_{max} determined from loading tests are biased unconservatively, because production shafts may not be constructed with the same care as test shafts. Grouting all shaft bases will ensure that the conditions of production shafts are similar to those in the test shafts, which means that the design formula may not change, but the associated reliability will be higher (e.g., higher resistance factors). Post-grouting the base of the drilled shaft will also permit utilization of the full theoretical base capacity, stiffen up the bases, and could alleviate concerns with long-term settlements. This has worked very well in stiff soils in Texas and will be considered in the future for other weak rock formations. Some of the southern state DOT's (like Mississippi and Florida) are beginning to do base grouting for shafts constructed under slurry to stiffen up the bases. This work is performed by a company called "Applied Foundation Technologies."

Rapid drilling and placement of concrete of the shaft holes is also expected as per Section 503.07 of CDOT specifications. The drilling and concreting process should be continuous, with no stoppage of work between the completion of drilling and cleaning the hole and placement of concrete after setting the steel cage. The rate of rise of concrete should be at least 12 m (40 feet) per hour and the 7–8 inches slump is maintained to ensure that ground stresses are re-established. If the concrete is not placed the same day as the drilling of the socket occurs, the contractor shall either "overream" the hole (cut it to a larger diameter) by 2 inches or increase the rock socket length by 1/3 of the specified socket length, prior to placing concrete. This requirement might be waived if directed by the Engineer after consultation with the geotechnical engineer for very large shafts embedded in cemented, very hard clay-shale, or durable rock where this requirement is not economically feasible and the rock strength would not be reduced due to excessive exposure time.

In order to prevent the rock socket of the production shafts from being smooth, it is also expected that the drillers: 1) will not use drilling slurry or casing in the rock socket, 2) will not pour water to make cuttings sticky so they can be picked up by an auger or bucket, 3) will use casing in the overburden when perched water is expected, and 4) remove quickly any water encountered in the rock socket. The requirement for not using slurry in drilling the rock socket could be waived (e.g., in caving sandstone) in writing by the Engineer after consultation with the project geotechnical engineer who might adjust the design side resistance values.

As discussed in Chapter 5, normal auger-drilling in very hard claystone and sandstone bedrock will generate a rough-sided socket that was accounted for in the design methods presented below. To be conservative, an intermediate roughness level is assumed. It should be required that the drillers make a final drilling pass by replacing the outer cutting teeth with a "roughening" tooth that extends about 1.7" from the sides of the auger to roughen the socket at least minimally. This is done routinely in Colorado shafts embedded in soft claystone (as described for the County Line and I-225 shafts in Chapter 3). This requirement should be made universal for all shafts constructed in Colorado. It should be noted that this procedure is much less rigorous than the extensive and more expensive process of roughening with shear rings (see Chapter 3). It is intended to remove any smear from the sides of the borehole in order to allow the natural roughness to be effective.

Scaling to Shafts with Different D and L

The design methods presented next could be used for drilled shafts with diameter (D) or length of rock socket (L) different than those used at the four load test sites. The first requirement is that the construction techniques of these shafts remain the same as those at the load test sites. A good rule to remember that when scaling upward in diameter, the unit base and side resistance values go down (are unconservative), at least in theory. There are no specific extrapolation formulas, but it is well known that if part of the side resistance comes from interface dilation, the larger the diameter of the shaft the smaller the radial effective stresses when the shaft is pushed up or down, and in theory the smaller the unit side resistance. Load testing a shaft with a diameter 50% smaller than that of the production shafts is usually acceptable. In other words, there is not much of a scaling problem, if any, going from results on 4-ft diameter shafts to the design of shafts up to 6 ft or even 8 ft diameter. If the results were used to extrapolate a 4-ft shaft to the design of 10 or 12-ft-diameter shafts, then there could be a problem. To be conservative, the following recommendations for upward scaling are suggested:

- Use the design methods with no modifications for shafts up to 5 ft in diameter.
- For shafts with D>5 ft, reduce the unit base resistance (q_{max}) linearly with D, so q_{max} for a 10-ft-diameter shaft is 50% of that recommended by the design method.
- For shafts with D>5 ft, reduce f_{max} linearly with D, so f_{max} for a 10-ft diameter shaft is 75 % of that recommended by the design method.

If settlement estimates are not to be performed, the recommendations listed above for reducing the resistance values will limit the service settlements of large-diameter shafts (see Section 2.2.5).

The socket length (L) affects the shape of the load transfer curves but does not affect ultimate resistance of weak rocks significantly. The computer program CDOT SHAFT developed in this study can be used to study the influence of L and D on the load transfer curves and ultimate base and side resistance values for shafts with diameters up to 8 ft.

Long-Term Performance of Drilled Shafts:

Time dependent settlements (creep) of shafts embedded in claystone bedrock over their design life (often 75 years) might be important when a high percentage of the load is dead load, and could have influence on the requirement for allowable settlement at design loads (i.e., the serviceability limit design). LOADTEST, Inc. (2002) provided rough estimates of the creep limit loads for the four test shafts (I-225, County Line, Franklin and Broadway). LOADTEST, Inc. believes that significant creep for these shafts will begin at top loads higher than those recommend in this study for the service design loads (Q_{all} in Tables 6.1 to 6.4).

Long-term capacity, which load tests such as those reported in this study do not address, may be less than short-term capacity, which this study does address. There is a possibility of long-term softening of the claystone due to stress relief along the rock fractures (occurs when q_{max} exceeds 2.5 q_{ui}, see Chapter 2) and perhaps water infiltration through fractures. If the weak rock or rock is sensitive to water (has high slake loss as determined from Colorado Testing Procedure 26-90), FHWA (1999) recommends to consider q_{max}= 2.5 q_{ui} (even in massive claystone and sandstone). Using q_{max}= 2.5 q_{ui} would result in base resistance values smaller than those recommended below for soil-like claystone and very hard sandy claystone. It will be required to perform Colorado Testing Procedure 26-90 on the very hard claystone and verify it is rock-like material (durable, not sensitive to water, and very small potential for creep).

For the soil-like claystone, it is recommended to use q_{max}= 3.8 q_{ui} (equivalent to q_{max}= 0.92 N, not q_{max}= 2.5 q_{ui}) and not to consider any creep settlements for the following reasons:

- Softening or creep settlements of drilled shafts embedded in soil-like claystone bedrock were not reported by Colorado geotechnical engineers. The short- and long-term performance of innumerable structures designed in Colorado over the last 40 yrs with the CSB design method on soil-like claystone has been excellent.

- The load tests on test shafts, embedded in wet soil-like claystone rock having moisture content of approximately 20% and saturation levels around 90%, were performed to base resistance larger than 2.5 q_{ui} with no signs of fracturing. The SPT, UCT, and PMT were performed on the same wet soil-like claystone rock. Most of the I-225 claystone bedrock was located under the GWL.
- The short-term settlements of shafts embedded in the claystone (soft to very hard) under service loads were very small (~0.3 inch, smaller than the tolerable settlements).

The discussion presented above suggests that long-term settlement or softening of the claystone, bedrock supporting the drilled shafts is not of any concern. As discussed before, post-grouting the shaft bases could alleviate concerns with long-term settlements, and is therefore recommended as an additional precaution measure. In addition, an accelerated research study or long-term monitoring of the settlements of shafts embedded in claystone bedrock is recommended in the future.

6.4.2 Design Methods for Soil-Like Claystone (I-25 @ I-225 and County Line Sites)

Clay-based geomaterials with SPT-N values (bpf) between 20 and 100 (q_{ui} <24 ksf) are defined as soil-like claystone bedrocks (Tables 6.1 and 6.2).

Use the updated Colorado SPT-Based (UCSB) design method recommended in Chapter 5, where

q_{max}(ksf) = 0.92 N (bpf), ...6.4a

and

f_{max}(ksf) = 0.075 N(bpf) 6.4b

Use a resistance factor of 0.75 or factor of safety of 2 with this method, so q_{all} (ksf) = 0.46N and f_{all} (ksf) =0.037N. Note the measured factor of safety associated with the UCSB method is 2 (Table 5.6).

Neglect the side resistance in elevation ranges where, based on the judgment of the geotechnical engineer, the soil-like claystone will at some time in the design life of the foundation shrink away from the side of the shaft. In general, a depth of five feet below the ground surface is recommended in the FHWA manual for average seasonal wet-dry conditions. In some parts of Colorado, this depth may be deeper than five feet. If the soil-like claystone is expansive and where long dry periods follow long wet periods (expected to be the case for Denver), a minimum exclusion zone of 5 feet and a maximum exclusion zone of 15 feet is recommended. If the exclusion zone lies completely within the overburden, the shaft capacity is not affected.

The UCSB method will produce a factor of safety very close to or larger than 2, which is higher than the FS generated from the CSB design method (1.3- 1.8) currently used in Colorado. Other AASHTO and FHWA design equations for the soil-like claystone employ high factors of safety, ranging from 2.3 to 3. It is recommended to use the UCSB design method with relatively smaller FS than the AASHTO method because: 1) of the excellent short- and long-term performance of innumerable structures designed in Colorado over the last 40 yrs with the CSB design method; 2) it is more cost-effective than the AASHTO/FHWA strength-based design method that employs a higher FS; 3) the use of SPT-based design is commonplace in Colorado; and 4) it is more consistent to obtain SPT data than UC strength data in soil-like claystone geomaterial. FHWA was not intended to be a definitive design guide – only a default in the event that better local and regional information are not available, which with this study will have been developed for the Colorado weak rock formations. But, it should be realized by Colorado geotechnical engineers that the FS of the UCSB is close to 2 (resistance factor of 0.75) not 3 as currently assumed by CDOT engineers for the less conservative CSB method.

A very simple load-settlement curve can be constructed as a function of the SPT N values as shown in Table 5.5. The elastic compression of shafts embedded in the soil-like claystone geomaterial is very small and can be neglected. The settlements that would occur under the design loads are expected to be less than 0.3 inches.

6.4.3 Soil-Like Sandstone

Sandstone-based geomaterials with SPT-N values between 50 and 100 are defined as soil-like sandstone geomaterials. No load tests on such material was performed in the current study or reported in Colorado. This material is equivalent to the cohesionless IGM's described in the FHWA design manual (1999). The FHWA design manual (1999) recommends design equations for this material that carry the assumption that the sandstone is behaving like a very dense sand:

$$q_{max}\,(ksf) = 1.07\sigma_v^{0.2}\,(N)^{0.8}, \quad \text{6.5a}$$

and

$$f_{max} = K_o\sigma_v \tan\varphi \quad \text{6.5b}$$

where N is the average SPT blow count in which 60% of the potential energy of the hammer is transferred to the top of the driving string; σ_v is the vertical effective stress (at the base for base resistance, middle of the layer for side resistance); K_o is the at-rest coefficient of earth pressure, and φ is the angle of internal friction. Expressions for K_o and φ are as a function of N and σ_v are described in the FHWA design manual.

AASHTO (1998) and FHWA (1999) did not recommend factors of safety and resistance factors for cohesionless IGM's, because of shortage of load test data. It is recommended to use a resistance factor of 0.5 (FS=3) for this design method. The factor of safety could possibly be reduced and the resistance factor increased with the acquisition and analysis of additional load test data in the soil-like sandstone geomaterials of Colorado.

6.4.4 Very Hard Sandy Claystone (I-25@Franklin Site)

The Franklin bedrock is a very hard, mostly thinly bedded, bluish gray, and sandy claystone bedrock with q_{ui} ranging from 40 ksf to 90 ksf (average of 65 ksf) around the bedrock socket and around 41 ksf beneath the socket (Table 6.4). In this rock, SPT testing was terminated in the second interval with 50 blows per 4 inches of penetration (50/4") around the shaft and 50/5" beneath the shaft. It is required to perform Colorado Testing Procedure 26-90 on samples of this claystone and verify it is classified as rock-like material. If the material is classified as soil-like, it should be treated in the design as soil-like claystone.

The following design equations are recommended for conditions similar to those at the Franklin test shaft:

Use the Canadian design equation (AASHTO LRFD, 1998) with resistance factor of 0.5 to predict the base resistance:

$$q_{max} = 1.2\,\Lambda(q_{ui}) \qquad (6.6)$$

where Λ is a depth factor = $(1+L/D) \leq 3.4$.

Use the Carter and Kulhawy design method with intermediate roughness level and a resistance factor of 0.55 (AASHTO LRFD 1998) to predict the side resistance as:

$$f_{max}\,(\text{ksf}) = 2.05\, q_{ui}^{\ 0.5} \qquad (6.7)$$

A very simple load-settlement curve can be constructed as a function of the unconfined compressive strength of intact cores (q_{ui}) and is shown in Table 5.5. The elastic compression of shafts embedded in the competent claystone bedrock is significant and should be calculated as described in Chapter 3. The settlements that would occur under the design loads are expected to be less than 0.3 inches.

Follow the recommendations in the next section for collecting accurate SPT data in the very hard sandy claystone.

6.4.5 Very Hard Clayey Sandstone (I-25@Broadway Site)

The Broadway bedrock is very hard, well-cemented, bluish gray and clayey sandstone with claystone interbeds and q_{ui} ranging from 97 ksf to 293 ksf (average of 145 ksf) around the bedrock socket and around 219 ksf beneath the socket (Table 6.4). In the rock around the test shaft, SPT testing was terminated during both the second interval (50/3”) and the first interval (100/5.5”). In the rock beneath the test shaft, the SPT testing was terminated in the first interval (83/6”). The following design equation is recommended for conditions similar to those of the Broadway test shaft.

Use a modified form of the Zhang and Einstein design method (FHWA, 1999) with resistance factor of 0.55 to predict the ultimate base resistance (FS=2.72)

q_{max} (ksf) = 17 $(q_{ui})^{0.5}$ 6.8

For estimation of f_{max} and construction of a simple load-settlement curve (Table 5.5), follow the methods described for the very hard claystone bedrock (Section 6.4.4). The settlements that would occur under the design loads are expected to be less than 0.5 inches.

SPT data in the very hard claystone and sandstone rock will provide a cheap, but could be a very rough, picture of the level of the rock strength. Future research should attempt to correlate accurate SPT data in this hard rock with the corresponding resistance and strength data. A procedure for obtaining accurate SPT data in the very hard claystone and sandstone rocks is described in Section 6.5.2.

6.5 Assessment and Preliminary Recommendations for Improvement of Colorado Geotechnical Subsurface Investigation Procedures

It seems there are no uniform standards in Colorado for assigning design geotechnical test data for rock layers (e.g., SPT-N values or strength data) based on the results of the subsurface geotechnical investigation. There are many differences between the geotechnical testing methods performed at the load test sites by CDOT and those performed by Colorado’s private geotechnical firms, leading in many cases to significant differences in the obtained testing results (SPT and core specimen strength data, see Chapter 3). In addition, two new sampling techniques for weak rock were tried in this study (triple-walled core barrel and the continuous sampler) to obtain reliable core runs for strength testing. The PM test, which is not a routine geotechnical test in Colorado, was heavily used in this study for different types of weak rocks. However, the PM test method used in this study was developed for soils, leading to problems when applied to weak rocks. The errors with various testing techniques are discussed in detail in Chapter 5 of the FHWA LRFD manual (1998). These errors could be minimized by standardizing the equipment, testing procedures, and quality control of the geotechnical tests. The lessons learned in this study for improvement of Colorado subsurface geotechnical investigations are summarized in this section. A new research study to improve geotechnical subsurface investigations in Colorado and place them in accordance with the LRFD method is recommended in Section 7.1.

6.5.1 Adequate Subsurface Geotechnical Investigation

The conventional geotechnical site investigation consists of making borings to obtain core samples for strength testing and/or to perform SPT tests. Table 2.1 of the FHWA Design Manual (1999) provides information on the frequency of borings for drilled shafts for bridges. For example, one boring per shaft is recommended per single-column, single shaft foundations. Other considerations (geologic details, variability of subsurface conditions, and accessibility) might dictate other boring patterns. For rocks with RQD greater than 50%, the depth of the boring is the expected depth of the production shaft plus two base diameters. For each uniform rock layer identified at the site from several borings, it is preferred to perform at least 30 tests (SPT or strength tests). At a minimum 10 tests should be performed per layer. For data analysis, any high-side outlier data may be discounted. The mean value of the measurements could be used in the design if the coefficient of variation of the remaining measurements (COV) is less than 0.35 for strength and SPT data. This value could be modified based on engineering judgment and past experience of the geotechnical engineer. When a site is extremely variable and/or when few tests are performed, the uncertainty of the measured geotechnical data will be high and it will be difficult to meet the requirements for COV listed above. In this case, several options are available to the designer. First, divide the characterization domain into smaller domains each with different design parameters meeting the requirements for COVs listed above. Second, make more borings and increase the number of tests. Third, assign resistance factors smaller than those recommended in AASHTO or in this study (resistance factor considered as a function of a site variability is discussed in Chapter 7). Fourth, use values near the lower bounds of measurements.

For weak layers with varying strength within two diameters below the base of the shaft, consider the layer with the lower SPT N or strength value for estimation of q_{max}.

6.5.2 SPT Test

Different diameter samplers (e.g., SPT with ID = 1 3/8 inches vs. California sampler with ID = 2 inches vs. Ring sampler with ID = 3.0 inches) under the same energy provide different number of blows per 1-foot penetration, particularly for hard to very hard bedrock (Chapter 3). Even when the sampler diameters are the same, it is possible that the measured bpf values will be different (see results at the County Line site in Chapter 3). Many of Colorado geotechnical engineers (from personnel communication with Bill Attwooll, Dennis Hanneman, and Greg Fisher) believe that SPT and California samplers provide very similar results in term of bpf for the softer bedrock (e.g. Soil like claystone). This issue should be investigated in future CDOT research studies (Section 7.1).

The SPT test should be performed in accordance with ASTM D1586 and the following recommendations:

- Use an automatic safety hammer which is the one guided by a tube exterior to the hammer to make sure the operations is safe to workers (i.e., outside tube to protect hands). In the manual hammer, the number of wraps around the cathead has an effect on the energy delivered to the sampler and could lead to the variability in the obtained SPT results. Variations in the sampler drop height were also possible with the manual hammer.
- Perform SPT at levels not previously disturbed by other tests.
- Use the same-sized (weight) rods.
- In the very hard claystone and sandstone bedrocks, it is expected that the maximum blow count of 50 will be reached during the first interval for penetration less than 6 inches (e.g., 50 blows with penetration of 3 inches or 50/3”). If this was the case, the reported penetration depth (e.g., 3 inches) could be higher than it should be due to seating problems early in the test. In such a case, it is recommended to apply 12 blows to seat the tool in the bottom of the borehole. Then, drive the spoon, in a first test interval, with 50 blows or to a penetration of 6

inches, whichever comes first. In the former case, the penetration for 50 blows, in inches, is recorded. In the later case, the number of blows (N) for 6 inches of penetration is recorded. The spoon is then driven again in a second interval as done for the first interval. Because the penetration depth associated with 50 blows could be very small (1/4" to 1"), it is critical to measure this penetration very precisely. Based on the results of the 1st two intervals, record the equivalent SPT-N values for penetration of 12". In some cases, there is a need to penetrate at least 2 inches to recover a sample for lab testing.

- Monitor a significant number of SPTs using a PDA-type device specially marketed by GRL/PDI to monitor energy delivered to the head of the drive string (rod). This is to make sure that around 60% of the potential energy of the hammer is delivered into the rod (to obtain the SPT N_{60} value). Such calibration should be done at several depths and in both soft and hard geomaterial (and in between). The automatic hammer is expected to meet this requirement.

6.5.3 Sampling Procedure

Drill one borehole to accommodate HQ-sized core barrels and sample the weak rock using either the tripe-walled barrel producing 5 ft core runs or the double-walled core barrel (called "wire line" technique" in CDOT) producing 5 ft core runs. The third priority would be to use a push type sampler that could be advanced with a hollow stem auger (no water) producing 2.5 ft core runs as in the continuous sampler. The California sampler is recommended as a last resort.

The triple-walled core barrel is similar to the more common double-walled barrel, but also has a plastic tube insert that fits inside the inner barrel of the double-walled sampler. The inner plastic tube is thinner than the inner barrel of the double-walled core barrel, allowing the sample to be retrieved without breaking apart along joints or seams. Hence, it is expected that the triple-walled core barrel technique will produce enhanced core recovery, less core breakage, and protected storage for cores during handling and transportation of the core samples to the lab**.**

One of the coauthors (Hanneman) does not agree that the triple wall core system is superior to the conventional wireline coring for Denver's weak rock formations based on the following observations. It was very common for condensation to accumulate on the plastic tube between the time that the CDOT Drill Crew collected the samples and the time that the samples were logged in the laboratory (typically less than 24 hours), requiring the core to be removed from the tube for logging. For many of the claystone samples and the clayey to very clayey sandstone samples, significant smearing occurred on the outer portion of the core, also requiring removal of the core from the tube to scrape away the smear to see the actual core. This situation also occurs with double wall coring, but the samples are easier to access. In addition, a few of the claystone samples swelled tight in the plastic tube. The tube does not fit sufficiently tightly to prevent swelling. However, most of these problems can be eliminated by cutting the plastic tube to recover the core runs as recommended later.

The push tube sampler is not a common tool in the Denver area. It is recommended here to be used in cases where severe washout occurs during coring, making the core incapable of being recovered intact. The continuous sampler system was investigated in this study. The specific details of CDOT experience with this technique are not documented in this report, but could be obtained by contacting the CDOT Research Office. It was concluded that the continuous sampler system: 1) is excellent for logging the profile of the test hole because the recovery would be always nearly 100%, 2) could be a competitive alternative in very soft rocks having SPT less than 30 bpf; and 3) requires more research to study the effectiveness of this system and its proper use. If the push tube sampler also does not render acceptable samples, other devices, such as the "California drive sampler" can be used to recover samples for laboratory testing. The California drive sampler, which is commonly used in the Denver area, will usually recover samples, but it produces a very high degree of rock disturbance in those samples. The measured unconfined compression strength of the California sampler specimens at the Broadway site ranged from 3.1 ksf to 29.2 ksf with an average of 17.4 ksf. When specimens were obtained through coring, much higher strength values were obtained by CDOT in this study (ranged from 85 ksf to 312 ksf).

The study's other recommendations for sampling weak rock cores are:

- California drive sampling (for retrieving samples for strength testing) should not be encouraged in Colorado in future practice unless it is absolutely necessary to use this device to obtain samples (as a last resort). However, retrieved core samples from California sampler can be used for other testing purposes (e.g., for the swelling-consolidation test used for estimation of the minimum dead loads needed in the design of lightly-loaded shafts). Some geotechnical engineers (as in the County Line and I-225 sites) alternate the ring and SPT sampler to obtain N-values and retrieve samples.
- Preserve all samples very carefully to prevent them from cracking or expanding before they are returned to the laboratory and tested.
- CDOT needs to use the split core barrel, as often employed by Colorado geotechnical firms, instead of a solid barrel. This will make it easier to retrieve core runs and thus reduce disturbance.
- Cores should be logged as soon as possible (in the field if possible). Describe the color, textural appearance, joint structure (RQD and recovery percentage), and bedding direction, and the elevation of the rock cores. Some of these terms may be hard or inappropriate to define for soil-like claystone and sandstone rocks.
- In order to recover the core run from the plastic tube of the triple-walled core barrel with minimal disturbance, cut very carefully the plastic tube away from the core (plastic tube is not reusable in such cases), especially when the rock is highly jointed.
- Advance the test hole with the hollow-stem auger (OD of the auger is ~7 inches) to the top of the rock. Remove the auger and advance a coring casing string with inside diameter of 4 inches to top of the rock to hold the hole open. Then, drill inside this casing through the rock with the HQ-sized core barrels (O.D =3.5 inches). This approach is faster than advancing the hole through the overburden with the coring string. Additionally, advancing the core string in the overburden of the Broadway site resulted in the destruction of the core bit because of presence of cobbles in the overburden. Alternatively, coring is often conducted in private practice by using the hollow stem auger "screwed" into the top of bedrock as the casing.
- For the very hard claystone and sandstone bedrock (with q_{ui} around 350 ksf), it was noticed that the toothed bit worked better than the hard bit in the collection of HQ cores.
- Colorado private geotechnical firms utilize the NX size cores (ID~ 2 inches), while CDOT utilizes the larger HQ size cores (ID~ 2.5 inches), which could cause differences in the

measured strength data. The HQ core string typically achieves better recovery and more weak rock is recovered. The HQ size core is the standard adopted and recommended in this study.

- If needed, sample overburden soils at 5-foot vertical depth intervals with a tube sampler that is pushed (e.g., Shelby tube), not driven, into the ground for cohesive soil, or with a split spoon sampler during the performance of the SPT for cohesionless soil.

6.5.3 UC Test

Conduct unconfined compression tests in accordance with ASTM D-2166 and measure the unconfined compressive strength (q_{ui}) and initial elastic modulus (E_i) of the intact core specimens. This is the procedure for strength testing of soil specimens, not rock specimens, as per ASTM D2938. ASTM D2938 appears not to be appropriate for Denver's weak bedrock formations. Consider also the following recommendations:

- The test should be performed within 24 hours after samples are collected.
- Based on the results of SPT and strength tests, visual examination of the recovered samples and core runs (see information collected for logging the cores), and drilling information, define the boundaries of different weak rock layers and the elevation of top of rock (see definition of rocks, SPT> 50 for soil-like sandstone, or SPT> 20 for soil-like claystone).
- For each uniform rock layer in the zone of interest, acquire a large number of core specimens at the site for strength testing as discussed before.
- The UC test should be performed on intact core specimens. If the sample failed along an existing fracture, the results should be neglected or given a low weight.
- In the event that samples are friable (soils or weak rocks), confined compression tests (unconsolidated, undrained triaxial compression tests) should be substituted, in which the cell pressure in psi is made equal to the sample depth in feet, and the results are expressed as the confined compression strength, q_c, and used in correlations and design in the same manner as q_{ui}.

6.5.5 Menard PM Test

Care should be exercised to excavate boreholes of uniform diameter and circular cross-section. Two methods are learned in this study for preparation of test pockets for PM tests in weak rocks:

- Dry drilling method for soil-like claystone (County Line and I-225) and very hard claystone with unconfined compressive strength less than or around 60 ksf (Franklin). Advance the test hole with a 7-inch hollow-stem auger to the top of the test pocket. Then prepare the test pocket by drilling with a 3-inch solid flight auger about 5 feet below the base of the hollow stem auger hole. Dry drilling should be considered because it is faster and maintains the rock at its natural moisture content, which is important for soil-like claystone bedrock.
- Wet drilling method for very hard (q_{ui} larger or around 100 ksf, Broadway) claystone and sandstone bedrocks (very hard for solid auger to penetrate). This method is preferred when the bedrock is located under the groundwater level. Advance the test hole to the top of the test pocket using an N casing (ID= 3 inches) string. Alternatively, advance the test hole to the top of the test pocket with the hollow-stem auger in the overburden (or to top of the test pocket). Remove the auger and advance a coring casing string with inside diameter of 4 inches to hold open unstable holes (i.e., caving soils or if water is present in the overburden). No casing is needed for stable holes. Then, prepare the test pocket by rotary drilling with a 2 15/16-inch tricone rock bit and a drilling fluid to a depth of about 5 feet below the casing.

Perform the PM test on the weak rock in general accordance with the recommendations of ASTM D-4719 (note that this PM test is for soils). The PM test should consist of loading the pressuremeter into the linear range, unloading, and reloading until the limit pressure is reached, or until 15% radial strain from the at-rest condition is reached (note that this strain is smaller than for soils), or the maximum pressure or volume capacity of the PM system are reached. Consider the following additional recommendations:

- In each uniform rock layer identified with the results of SPT and UC tests, at least 1 PM test should be performed.

- In this study, the 15% radial strain was not obtained when the solid auger was used to advance the test pocket. This is because the diameter of this auger was relatively large (3 inches) and the volume capacity of the PM test was limited to 500 cc. For a PM test with an N-sized probe (2.76 inches diameter), prepare the test pocket using a smaller (2.9-inch) auger or a 2 15/16-inch (2.93 inches) tricone bit. Alternatively, the volume capacity of the PM system should be increased from 500 cc to 600 cc.
- The unloading phase in the PM test should be performed along several small pressure increments -as performed during the loading phase (not one large increment).
- Raw data should be corrected for membrane stiffness and flexibility of the tubes and fittings, as is standard practice.
- Collect the PM test data through a data acquisition system (if possible).
- Perform more than one unload-reload cycle. One cycle must be performed at the end of the elastic range.
- In very hard rocks with unconfined compressive strength larger than 50 ksf (Franklin and Broadway sites), it is recommended to utilize a special rock-type PM or rock dilatometer instead of the standard soil Menard-type PM.

7. A PLAN TO IMPROVE THE GEOTECHNICAL AXIAL DESIGN METHODS FOR COLORADO DRILLED SHAFTS

The resistance side of the LRFD method for drilled shafts includes: 1) a design method that transforms the geotechnical properties of the geomaterial, collected through a subsurface geotechnical investigation program that involves certain equipments and testing procedures, into resistance values, and 2) a resistance factor, ϕ, to account for uncertainties associated with resistance predictions of the design method. The nominal resistance calculated from the design method can be different from the "true" resistance (often measured from the results of load test). To account for this discrepancy, an overall bias factor, $\lambda_{resistance}$, is defined as the true resistance over the predicted resistance from the design method. The bias factor includes the net effect of several sources of errors (three considered in this study) that contribute to the uncertainty of predicted resistance (FHWA, 1998):

- Measurement errors due to equipment and testing procedures. For example, use of an older drill rig may provide systematically higher blow counts compared to a new rig using automatic and safety hammer.

- Inherent spatial variability of subsurface materials at the site that can be determined from the results of a subsurface exploration and testing program. The uncertainty associated with the value of SPT blow counts at a point is larger than the uncertainty associated with an average value measured over a distance or volume, because of averaging effect. For example, the uncertainty of spatial variations across a site will be reduced with increasing the number of test borings.

- Errors due to an overall bias in the predictive design model (or method) that transforms the geotechnical measurements into resistance values. For example, some design methods tend to under predict the resistance.

The uncertainty associated with the bias of each source of uncertainty listed above can be represented by its mean and coefficient of variation (COV). According to the FHWA (1998), the mean of the overall bias factor ($\underline{\lambda}_{resistance}$) may be written as the product of the mean of individual bias factors of the three sources of uncertainties listed above (assumed to be independent):

$$\underline{\lambda}_{resistance} = \underline{\lambda}_{measurements} \times \underline{\lambda}_{inherent} \times \underline{\lambda}_{model}, \qquad 7.1$$

and the overall coefficient of variation of the overall bias on resistance, $COV_{resistance}$, can be calculated as the square root of the sum of the squares of the individual coefficients of variation of the three sources of uncertainties listed above:

$$COV_{resistance} = \text{Square root of } (COV^2_{measurements} + COV^2_{inherent} + COV^2_{model}) \qquad 7.2$$

Information on $\underline{\lambda}_{model}$ and COV_{model} of various design methods are needed to identify the most accurate design method (the method with $\underline{\lambda}_{model}$ approaches unity and smallest COV_{model}). Information on $\underline{\lambda}_{resistance}$ and $COV_{resistance}$ of any design method are needed to evaluate the resistance factor (ϕ) for that method (see FHWA, 1998). These are needed to fulfill CDOT strategic objectives for improvement of the geotechnical design methodology for drilled shafts (listed in Chapter 1). Numerical values must be developed to calculate these bias factors and coefficients of variation. This Chapter will present a comprehensive plan to obtain these values. This plan has three parts:

- ❑ Improvement of the geotechnical subsurface investigation in Colorado and acquiring the testing and site specific resistance parameters of the LRFD ($\underline{\lambda}_{measurements}$, $\underline{\lambda}_{inherent}$, $COV_{measurements}$, and $COV_{inherent}$). This will be covered in Task 1, which follows.

- ❑ Assembly of a database of full-scale loading tests on drilled shafts embedded in various categories of Colorado's weak rocks. These loading tests will be those performed in connection with this project, selected older load tests, and load tests that will be performed in the future. The database for each such load test should contain sufficient geotechnical test information at the load test site, such as results from SPT, UC test, and PMT; a geologic description of the geomaterial layering at the test site; and construction and materials details of the test shafts. This database will provide the numerical values and information needed to estimate $\underline{\lambda}_{model}$ and COV_{model} for various design methods. Very specific details for obtaining such information are presented Tasks 2 through 4, which follow.

- Analysis of all the information collected in the previous two steps and publication and promotion of the findings (covered in Tasks 5 and 6, which follow).

7.1 Task 1: A New Research Study to Improve Geotechnical Subsurface Investigation Procedures in Colorado and Determine the Testing and Site Specific Resistance Parameters of the LRFD

Study Objectives:

- Develop guidelines for a uniform, accurate, feasible, and "adequate" subsurface exploration and lab-testing program that include standards for SPT and/or strength tests (UC tests) and/or PM tests. All the details of this subsurface investigation should be outlined for all the typical weak rock sites encountered in Colorado, including the extraction of the geotechnical test parameters that will used in the design calculation to estimate the unit base and side resistance values. The requirements of adequate subsurface exploration and lab-testing program (Section 6.5.1) should be reviewed and finalized in this study.

- Obtain Colorado-specific $\underline{\lambda}_{measurements}$ and $COV_{measurements}$ for various equipments and testing methods recommended under item 1.

- Develop a procedure for the geotechnical engineer to estimate $\underline{\lambda}_{inherent}$ and $COV_{inherent}$ of the geotechnical property (e.g., $COV_{inherent}$ of SPT N values for a rock layer) from the results of subsurface exploration and laboratory testing. The $COV_{inherent}$ at any site should be smaller than the COV selected in item 1 to define adequate" subsurface exploration and lab-testing program. The findings of this step should provide the geotechnical engineer with the approach to change resistance factor based on the level of performed subsurface geotechnical investigation.

Consider the findings of the current study for Colorado subsurface geotechnical investigations and the recommendations to improve them (Chapter 3, Section 6.5, and other sections). The findings of this study will establish the basis for the recommended design methods identified in this plan. It is not easy but possible to estimate $\lambda_{mean\ (inherent)}$ and $\lambda_{mean\ (inherent)}$. Review Literature and FHWA (1998), and results of recently completed NCHRP research projects on this issue.

All possible sources of error in the SPT, sampling and subsequent lab tests, and PM tests as discussed in Chapter 5 of the FHWA LRFD manual (1998) should be addressed and minimized in this study to obtain the lowest possible coefficient of variation. These errors could be minimized by standardizing the equipment, procedures, and quality control of the geotechnical tests as recommended in Section 6.5 of this report. This study should investigate the relation between the extent of the subsurface investigation (costs) and factor of safety or resistance factors (benefits) employed in the design. Typical values for COV^s that correspond to adequate level of subsurface investigation should be presented and used to calculate resistance factors for the design methods.

There are many differences in the geotechnical subsurface investigation and lab tests performed in Colorado. It is important to develop uniform standards for this investigation and the lab tests that follow. The proper sampling technique to collect reliable core specimens for strength tests from different soil and weak rock formations should be investigated. The type of core bit that should be used in the coring of different rocks also needs investigation (e.g., hard rock bit, toothed bit). The effectiveness of the continuous sampler and its proper use for different geomaterials should be investigated. The possibility of using other advanced sampling techniques should be studied, and the effectiveness of the use of the California sampler to obtain blow counts and strength data for different soil/rock formations and any correlation with the data obtained from standard tests should be investigated. Some Colorado geotechnical engineers believe that SPT and California samplers provide very similar results in term of bpf for the softer bedrock (i.e. Soil like claystone). Bill Attwoll indicated that the ring sampler blow count should be multiplied by about 0.6 to approximate the SPT-N value, an equation based on energy on the sampler end area. Standards for performing PM tests on weak rocks (available for soils) and data analysis should be outlined in this study.

The study should establish plans to implement/market/publicize the study findings in Colorado, including courses to train Colorado's technicians to perform geotechnical testing work per uniform standards.

7.2 Task 2: Literature Review and Data Collection

The technical literature will be reviewed to acquire rock socket load test data in important weak rock formations similar to those encountered in Colorado (e.g., Pierre Shale and Denver Formations). Such weak formations are found in Colorado, Wyoming, South Dakota, and Texas. For example, the tests quoted by Turner et al. (1993) can be used, and other tests conducted more recently by the CDOT can be added to this database. The FHWA database should be consulted for such load tests. CDOT files (e.g., Snow Mass and SH 82 load tests) should specifically be reviewed. Two firms in Colorado, Ground Engineering and CTL, may have access to load test data (e.g., load tests performed in downtown Denver). A load test was also performed by Woodward Clyde Consultants (now URS) near I-76 and I-270.

Papers in the literature describing drilled pier load tests in similar weak rock formations in other geographic areas may also be consulted and used in developing *an extended database, where such tests appear appropriate.* Records of the following information should be acquired for each load test included in the extended database: geographic location of the load test, type of load test, location of test borings relative to the location of load test, geologic formation of the weak rock, load-movement curve, load transfer curves in the event the test pier was instrumented, <u>measured ultimate unit side shear resistance, measured base resistance at a deflection of at least 1 inch (25 mm), q_{ui} (average and median values) or SPT-N value of the weak rock within and below the socket</u>, RQD of the weak rock within and below the socket, any available pressuremeter test results in the weak rock along and below the socket [location (depth and surface condition) and method of making the test (self-boring or Menard-type test) earth pressure at rest, Young's modulus – initial loading and reloading (if available), limit pressure], f'_c of the concrete at the time of testing, the procedure used to conduct the load test, construction details (time required for drilling, time required for concreting, slump of concrete, base and side cleaning procedures), and borehole roughness conditions. Those items that are underlined are considered critical. If data for these items cannot be found, the test should not be included in the database.

The technical literature will be reviewed for any new/promising/feasible design method for drilled shafts in weak rocks that was not covered in Chapter 3, so that the design methods that are proposed at the conclusion of the project are in context with modern thinking on drilled pier design (e.g., method of Seidel and Collingwood, 2001). The literature review should summarize the findings of newly completed research studies regarding the design of drilled shafts (e.g., NCHRP projects 24-17 and 21-08). The senior author is a panel member of NCHRP study 21-08 titled "Innovative Load Testing Systems." From that position, he will summarize the findings of this study, and determine if tests other than O-Cell load test (e.g., Statnamic test) should be considered by CDOT. Statnamic testing can be done after the shaft has been installed if problems are suspected, whereas the O-Cell test cannot. Statnamic or other high-strain tests are probably more cost-effective than O-Cell load tests, but methods of interpretation of these tests are not finalized yet as they are for the O-Cell load test. The Texas DOT has recently conducted the Statnamic load test. Statnamic does have a 3600-ton test device now and there is only one of these on the North American Continent (to our knowledge).

Neighboring states that have weak rock formations similar to Colorado's should be consulted for their load test data, design methods and geotechnical subsurface investigation. Both Missouri and Texas are working on very similar problems. Texas weak rock formations are similar to those in Denver.

The technical literature will be reviewed to acquire any two or more combinations of PM, UC (or any other strength results), and SPT test results in weak rock formations. These will be utilized to improve the correlation between the results of different testing methods for possible use in design methods.

In all efforts described before, it is important to obtain the geological description of the formations of weak rocks for which the test data are collected. The CDOT Foundation Unit and private geotechnical firms will be consulted to develop a list of the geological formations and types of the weak rocks that are typically encountered during construction of drilled shafts in Colorado, their locations, percentage of drilled shafts constructed in each, and the properties of the weak rocks (strength and SPT-N value) of each geological formation.

This task will also summarize the typical drilling methods of drilled shaft holes employed in Colorado in different types of rocks. The issue of how smooth or rough a hole drills in every rock type is worth investigating. It is expected that roughness will be smaller with the holes drilled with core barrels than with augers. Collection of roughness data will be part of the program for new load tests (Task 4).

7.3 Task 3: Acquisition of New Geotechnical Data at Sites of Existing Load Tests

For those sites identified in Task 2 as having load tests that meet the criterion of failure in side shear and development of base resistance for a deflection of at least 1 inch (25 mm), but in which adequate geotechnical data do not exist**,** additional geotechnical test data from a comprehensive subsurface geotechnical investigation on the weak rocks shall be acquired. If this investigation is performed before construction of the test shaft, drill three test holes as close as possible to the future center-location of the test shaft. If drilling to be performed after construction of test holes, and in order to get a picture of the undisturbed material and at the same time stay close enough to the shaft, it is recommended to drill three test holes at one pier diameter (D) from the edge of test shaft (3D/2 from the center of the shaft). The test holes will be placed 120 degrees from each other. Drill the test holes to a depth of 3 pier diameters below the base of the shaft. Subsurface geotechnical investigation methods at each test hole will include auger drilling with standard penetration testing, coring with subsequent laboratory testing on recovered core specimens, and in-situ pressuremeter testing. A large number of tests will be performed for each weak rock layer to accurately (as much as possible) acquire its geotechnical properties.

This work will be performed as described in the following and as per the recommendation of Section 6.5.

1. In the first test hole, SPT tests shall be made at 2.5-foot vertical intervals in the overburden and in the weak rock, or whenever a sand or friable sandstone layer is encountered. Disturbed soil and rock samples recovered with the split spoon sampler will be identified and classified visually in order to demonstrate correspondence with the core runs that will be taken in the second borehole. The first borehole will remain open for a period sufficient to ascertain whether free ground water seeps into the borehole and, if so, long enough to determine the

final piezometric level of the ground water. The initial and final piezometric level of the ground water will be reported.

2. In the second test hole, the borehole will be drilled to accommodate HQ-sized core barrels and sample the weak rock using one of the methods listed in Section 6.5.
3. An unconfined compression test or other strength test (Section 6.5) will be conducted on each of the samples obtained in Step 2 to obtain the unconfined compression strength (q_{ui}) and initial elastic modulus of the intact rock (E_i).
4. The RQD, natural moisture content and dry unit weight of each sample will be obtained in conjunction with each of the compression tests, and representative samples will be tested for Atterberg limits as described in Chapter 3.
5. Analysis of the load test results might require information on the side resistance in the overburden soil. If this was the case, overburden soil will be sampled at 5-foot vertical depth intervals as described in Section 6.5.3. The side resistance can be estimated from the measured SPT-N values for granular soils and from the measured undrained shear strength for cohesive soils (Section 6.5.3).
6. Based on the results of the SPT and strength tests, visual examination of the recovered samples and core runs, and drilling information, the boundaries of different weak rock layers will be defined.
7. In the third test hole, and for each uniform rock layer, at least one Menard pressuremeter test will be performed (Section 6.5.5). From the pressuremeter test data, the coefficient of lateral earth pressure at rest, the initial, reload, and unload moduli, and the cohesive shear strength of the rock will be determined and reported as described in this report.
8. A test report describing the location and conditions of the load test site, geological setting and formations, results of subsurface exploration, laboratory testing, and subsurface conditions will be prepared.

7.4 Task 4: Acquisition of New Geotechnical Data at Sites of New Axial Load Tests

The previous tasks will identify the amount of load testing information available on Colorado-like weak rock types in different geological formations. Based on this information, a preliminary number of new load tests needed for each typical weak rock formation will be determined. It is important to emphasize that new load tests can be considered also (not only to provide research data for improvement of the design methodology) as proof tests and to reduce the construction costs, as will be discussed later in this section.

Four types of Colorado rocks are suggested in Task 5. At least six load tests should be performed in each type of rock with the same drilling and construction techniques. In these tests, it is important to cover all the typical drilling methods performed in Colorado (summarized in Task 2) and the typical geological settings of these types of weak rocks. Rock type is probably a better delineator of behavior than geologic formation, unless the depositional process or age of one formation is very different from others in the study area.

Dr. Naser Abu-Hejleh from CDOT Research Office (303-757-9522) will be available to provide support for all activities described below. He can help in the design of new load tests and subsurface geotechnical investigations, in providing and reviewing the specifications for load tests, analysis of the load test results, and providing design modifications for production shafts based on the load test results.

At the locations of new axial load tests on drilled shafts, comprehensive subsurface geotechnical investigations should be performed as described in the previous section. This is needed for the proper design of the load test and to acquire accurate research strength data for the bedrock layers that could be correlated with the resistance values measured in the load tests. Therefore, it is necessary to perform the geotechnical subsurface investigation as described in Task 3 before performing the new load test.

7.4.1 Purposes and Promotion of New Load Tests

Axial loading tests are performed for two general purposes:

- ❑ To prove that the test shaft is capable of sustaining a given magnitude of an axial load (“proof test”). In this case, the test shaft is constructed in the same manner as the production shafts, usually under the construction project contract. The test shaft must sustain a load that is twice the working load without excessive settlement.

- To obtain the side load transfer curve (f-w) curve and f_{max} for all rock layers that will be encountered in all the production shafts and the base load transfer curve (q-w curve) and q_{max} for the rock layers that will be encountered beneath the production shafts ("load transfer test"). It is desirable that the load transfer test be conducted during the design phase, under a special contract, or in Phase 1 of a project that involves several phases. The load test data can then be used: 1) to design the production shafts in that project with more confidence (smaller FS and higher ϕ) and higher resistance values that would result in some savings to the project, 2) as research data to improve the design methodology in all future applications

Load tests are desirable where a large number of shafts are to be required. It is recommended that an economic study be performed for these large projects as described in Section 7.4.5 to determine the potential savings resulting from performing load tests. CDOT used the O-Cell load test method on the SH 82 project and later in Snowmass Canyon. The load test worked very well on the SH 82 project. On the Snowmass Canyon project savings were in the millions. As part of the construction requirements for T-REX and I-25/Broadway projects along I-25 in Denver, Colorado, four O-Cell load tests on drilled shafts were performed in early January 2002 (described in this report). The results from these tests were used to improve the geotechnical design of the production shafts in these two projects, and resulted in total saving estimated at $140,000 in the Broadway project (Chapter 2). More savings are expected in the future construction of bridges close to the Broadway and Franklin sites (Santa Fe and Alameda Interchanges) and in bedrock formations with geotechnical properties close to those encountered at the Broadway and Franklin sites.

The use of load tests should be publicized by the Colorado FHWA office, CDOT Geotechnical, Bridge, and Research offices, and by the Colorado private geotechnical firms. Publications (like this report), presentations, and brochures are also needed to promote the use of load tests in Colorado.

7.4.2 Location and Number of the Load Tests

Test locations should be selected following one or more of these criteria:

- At or close to the project site, in a location that represents all of the production shafts on the project.
- At or close to the weakest rock if the design was based on the weak rock (not relevant if uniform rock is encountered at the site)
- In flat areas accessible to large equipment (important with sacrificial shafts constructed before construction is started).
- At or close to shafts with the highest loads.
- At or close to locations where perched ground water will be encountered above rock.

The number of load tests should be determined based on the variability of the site rock layers as indicated in the foundation report prepared for the construction project. If multiple geologic formations exist on the site, load testing within each formation should be considered. For a uniform site, or if the weakest area will be tested and assumed in the design, a minimum of two load tests should be performed. A second load is needed to confirm the first load test (especially if an O-Cell load test) and to provide a sense of consistency. Since capacity of the shafts is influenced so strongly by construction, it is preferred to perform two load tests (or more). If there is consistency between the results of the two tests, resistance factor of 0.8 could be adopted in the design. For research purposes, one might consider a load test in the weakest area and a second in the strongest area to investigate the correlation between rock strength and resistance.

7.4.3 Type of Test Shafts (Production or Sacrificial)

The purpose and type of the load test determine the type of the test shaft. Production test shafts are often selected for proof load test, and sacrificial test shafts are used for load transfer tests. When the exact locations of the production shafts are not finalized, it is recommended to consider a sacrificial test shaft. Testing of a production shaft could be risky in some areas (e.g., under water). Performing a load transfer test on a sacrificial test shaft during the design phase would allow for design modifications of the production shafts based on the load test results and could result in cost savings to the project. If a production shaft is selected, it is best to consider a standard conventional load test. It is recommended that the O-Cell load test be performed only on a sacrificial test shaft, not a production shaft, if possible for the following reasons:

1. Filling the voids at the bottom of the shaft around and within the O-Cell with grout has a questionable effect on the structural integrity of the shaft.
2. With an O-Cell load test on production shaft, the designer needs to add perhaps two feet of extra penetration of the shaft in the competent rock.
3. In production shafts, the maximum upward applied load in the O-Cell load test has to be limited to maintain the functionality of the shafts after test completion. In a conventional load test, the load is applied downward as expected in production shafts under the service compression load.
4. The behavior of Colorado rock socket in side shear after the rock has failed in an O-Cell loading and *the direction of shear stress is reversed* is not well-understood. It is possible that in some cases the performance of the O-Cell loading test on a rock socket could result in lower side resistance in the same socket under service loading conditions.

However, the Broadway and Franklin test shafts were production shafts that were O-Cell load tested. In these test shafts, the ultimate base resistance and large portion of the side resistance were mobilized, resulting in savings to the projects. The performance of these two shafts should be monitored to study the long-term performance of these production shafts under service loads.

CDOT needs to leave the door open to other types of load testing, notably Statnamic tests, which can be done after the shaft has been installed if problems are suspected, whereas the O-Cell test cannot. Statnamic or other high-strain tests are probably more cost-effective than O-Cell load tests but methods of interpretation of these tests are not finalized yet. The second task will determine if other innovative load tests should be considered by CDOT in the future.

7.4.4 Features, Limitations, and Cost of Load Tests

The conventional static axial load test is the most reliable technique to determine the performance of shafts in the competent rock. The main limitation of this test is the high cost associated with set-up, test duration, construction delays, and instrumentation. These limitations are acute when high capacity foundations are involved (cost as high as a million dollars per test is reported in the literature). Alternative methods to standard static load testing, therefore, have been developed; one of these is the Osterberg Cell (O-Cell) load test, which is popular in Colorado.

In the loading of a shaft, the side resistance and base resistance may have an interaction effect (e.g., f-w curve influences the q-w curve). If a conventional load test is not instrumented, it would not be possible to separate the f-w curve from the q-w curve. If the conventional load test is instrumented, it is possible to separate the two load transfer curves and measure any interaction between these two load transfer relations. However, the side resistance and base resistance used in the current design methods for shafts are assumed to be independent (uncoupled) from each other. Therefore, it is recommended in all future load tests to obtain information on both the side resistance and the base resistance.

For the selection of the type of the load test (O-Cell or conventional), consider the following:

- Whether the test pier will be a production or a sacrificial pier, as discussed previously.

- The total capacity of the 34 inches O-Cell employed in the Broadway project is around 6000 tons (in two directions). A world record for a total load of 17000 tons was set in Arizona in 2001. Multiple O-cells can be used and placed in the same plane to increase the available test capacity and/or on two levels to isolate strata of interest. The O-Cell load test allows for obtaining both the base and side resistance values (even with no instrumentation). It can be planned that the O-Cell loading test will be conducted to failure either in side resistance or base resistance (whichever occurs first) or for both failures to occur at the same time. The ultimate base and side resistance were reached (almost) at the same stage for the I-225 and County Line test shafts. However, it is rare and hard to design the O-Cell load test for measuring both q_{max} and f_{max} from one load test. Therefore, there is a need to perform two O-Cell load tests to obtain the complete f-w and q-w curves. Additionally, instrumentation of the O-Cell load test is needed to obtain the f-w curves for different rock layers along the test socket. Finally, the O-cell test will provide side load transfer information for loads applied upward not downward as in actual loading of a shaft. The difference could be significant in granular materials but possibly not in competent bedrock shales. This issue should be finalized in the ongoing NCHRP 21-08 project.

- The highest capacity of a conventional load test is only (in theory) 4000 tons, and the test could be massively expensive. Only Caltrans (in the USA) has a beam for that capacity and it is a huge beam that is enormously expensive to transport and set-up. In Asia, tests sometimes approach 2000 tons using kentlage by having a 4-story pile of concrete blocks on the shaft. To alleviate the capacity problem and reduce the cost of conventional load tests, two conventional load tests are usually performed on smaller diameter shafts (i.e., 2 ft). The first test is to measure only the side resistance (f-w curve) in the bedrock only (no contribution from overburden), and the second test to measure only the q-w curve, both in a properly designed and controlled manner. The side shear test is accomplished by using a form material in the bottom of the hole to eliminate base resistance. The base resistance test is accomplished using an oversized hole or a shear breaker casing to eliminate side resistance. This approach is widely used and reported in the literature because (According to personnel communication from Ground Engineering, Inc. of Denver): 1) the side resistance contribution of overburden soils can be eliminated, 2) good quality data can be obtained from direct measurements, 3) data interpretation involves less assumption and estimation, and 4) the test requires smaller jacks and load frame, and as a result, can be performed at a lower cost. However, a risky extrapolation from a 2-ft-diamater shaft to production shafts more than 4 ft in diameter shafts may be involved in this process.

- Cost of the loading system. The O-Cell load test at the Broadway site using the 34 inches O-Cell (with total capacity of 6000 tons) costs around $70 K in 2002. This cost covers the expertise from LOADTTEST, Inc., and their equipment (O-Cell and instruments) and labor to perform the load test and issue a test report. For a total capacity of 9000 tons, the costs are expected to be doubled. Therefore, performing the O-Cell load test on smaller-diameter shafts (up to 5 ft diameter) and scaling the results to larger diameter shafts should be considered. For a conventional load test, the coauthor has never seen a load test on a full-sized drilled shaft (36 - 48 inches in diameter by 60 - 70 feet deep), that is instrumented, cost less than about $125,000 at an accessible site in Texas, and that is a very good price. This cost would include the installation of two to four reaction piers and provision of a reaction frame capable of resisting 1200 tons of applied load, jack, load cell, reference beams and deflection measurement instruments, technicians, engineer, etc. According to personnel communication from Ground Engineering, Inc. of Denver, costs for conventional tests done on the Big I project in New Mexico and the cable-stayed bridge in downtown Denver have shown conventional load test costs to be comparable to O-Cell tests. Based on a cost comparison developed by John Deland from CDOT for the Trinidad project in April 2003, it appears that cost difference between conventional and O-Cell load tests is not a major concern in the selection of the load test method.

According to the FHWA design manual (1999), the cost of O-Cell test is often in the range of 50% to 60% of the cost of performing a similar small capacity conventional loading test, because there is no need to construct a reaction system. The conditions under which the cost of conventional loading tests may be nearly the same as for O-Cell tests are (a) low capacity (less than 1200 tons, because 1200-ton reaction frames are generally available around the country), and (b) tests on shafts (production or sacrificial) using production shafts as reaction shafts (which cut out the costs for the reactions). For shafts with capacity higher than 1200 tons, the choices are to scale the test shaft downward in size, use the Caltrans frame, or use an expedient innovative load tests as the O-Cell or Statnamic device methods.

Based on the above, it is recommended to consider the O-Cell load test for the high-capacity production drilled shafts (ultimate load larger than 1500 ton), and either the O-Cell load test or the conventional load test for low-capacity production shafts. The possibility to perform Statanmic tests (see Task 2) should remain open. It is also recommended for CDOT to perform side-by-side O-Cell loading test and a top-down (standard) loading test at a couple of sites (e.g., soft claystone bedrock and very hard claystone or sandstone bedrock). This might allay any doubts that some engineers might have that those O-Cell tests give something close to the correct results. This issue and the recommendations for the use of the statnamic tests should all be addressed in NCHRP project 21-08. One of the objectives of this on-going research project is to evaluate innovative load testing methods for deep foundations and recommend interim procedures for use and interpretation of these tests.

The discussion below will assume O-Cell load tests on sacrificial test shafts.

7.4.5 Design of the O-Cell Load Test

The location, layout, construction process, and material of the test shafts should be similar to those planned for the production shafts.

Objectives of Load Tests:

The first task in the design of a load test program is to define the objectives of the load tests, or more specifically, the target unit side resistance (f_{target}) and the target base resistance (q_{target}) that should be expected from the load test. The f_{target} should be as close as possible to the true side resistance of the weak rock. The q_{target} should be as close as possible to the true base resistance for soil-like claystone bedrock, and for the displacement to exceed a settlement of 0.05D (preferred 0.1 D) for very hard claystone and sandstone bedrocks. Based on the results of geotechnical subsurface investigation at the location of load tests, and based on quality previous research data, the CDOT research office (or experienced geotechnical engineers) can provide an estimate for f_{target} and q_{target}..

Two O-Cell loads on two separate test shafts at two different locations is highly recommended. If the project budget allows for just only one test shaft, the designer may consider the option of a load test with two Osterberg cells placed at two levels inside one test shaft. This test can be designed to obtain both the f-w and q-w curves. The discussion below assumes two separate O-Cell load tests on two separate test shafts.

The results of the first load test should be obtained before the test shaft for the second load test is constructed. This is to correct for any problem encountered in the first load test and to necessary modifications to the location of the O-Cell in the second load test.

- ❑ The primary objective of the first load test is to obtain the f-w curve up to f_{target}, and the secondary objective is to obtain a portion of the q-w curve, which will be confirmed from the second load test. It is important for this to be the objective of the first load test because:

1. Almost 95% of the resistance to working loads at the Franklin and Broadway shafts was provided by means of side resistance, and then base resistance picked up the rest of the load resistance. The q-w curve for the Broadway shaft was linear (no yielding) up to the end of the test, suggesting that the true base resistance is higher than the resistance measured at the defined displacement criterion (0.05D, see Chapter 5).
2. For some bedrock formations, it is reported (FHWA, 1999) that side resistance might be lessened past the peak resistance (referred to as brittle behavior). This should be investigated for Colorado weak rocks as it has important consequences for the design.

- ❑ The primary objective of the second load test (unless modified based on the results of the first load test) is to obtain the complete q-w curve up to q_{target}, and the secondary objective is to obtain a portion of the f-w curve, which will be confirmed from the second load test.

Layout of the Test Shafts

Depth of overburden to competent rock should be determined from the subsurface geotechnical investigation. The L and D for the embedment of the production shafts should initially be calculated based on the recommendations of the geotechnical engineer for the construction project and based on the highest axial load expected in the project. Using the f_{target} and q_{target}, and a resistance factor of 0.8 in the LRFD method, L and D should be reevaluated and used for construction of the test shaft and to estimate the potential cost savings in applying the load tests.

If more than one alternative for shaft diameters is recommended in the project, and if the money is available, it would be best to test the larger diameter shaft, then scale those results down, which should be safe theoretically. However, to reduce cost of loading tests, it is recommended to determine f_{max} and q_{max} from tests on the small-diameter drilled shaft and then scale the results to the larger diameter shafts as discussed in Section 6.4.1. Test shafts should not have D less 0.5 the diameter of the prototype shaft, nor should they be less than 2.5 ft. Based on experience, the benefits of load tests are more for the smaller shaft diameters (4 ft not 7 ft). In the design of the load tests, the longest L value should be considered (including the L for the larger diameter shafts) in order to obtain f-w and q-w curves for all rock layers expected in the production shafts.

The influence of different L (e.g., from 12 ft to 21 ft) on the measured resistance values is expected to be small (see Section 6.4.1).

The real problem with scaling seems to come when one discovers that it is far cheaper to drill a six-inch-diameter socket and subject it to a pullout test to measure unit side resistance in order to apply the measured value to the design of larger-diameter shafts. O'Neill et. al. (1996) found that the unit side resistance on such small test sockets was about 2.7 times that on the full-sized drilled shafts-a disaster if this information is applied directly to a larger production shaft.

To ensure that almost all the "push" will be upward and guarantee side shear failure will occur in the test socket above O-Cell before base failure, and to take advantage of most of the stroke of the O-Cell,

q_{target} x $A_b/1.5 \geq (L_2-L_1)$ 3.14D x f_{target} + overburden side resistance, ...…… 7.1

capacity of O-Cell $\geq$ 3.14D f_{target} + overburden side resistance, ...7.2

where L_2 and L_1 are the lengths of shafts in the competent rock above and below the O-Cell. Based on Eqs. 7.1 and 7.2, the proper location of O-Cell can be determined.

For the second load test, the O-Cell needs to be placed at the bottom of the test shaft, and L should be large enough to guarantee a base failure before side failure:

2 $q_{target} A_b \leq L_2$ x 3.14D f_{target} + overburden side resistance, ...7.3

Capacity of O-Cell $\geq q_{target} A_b$. 7.4

The sets (four gages per set) of strain gages (Geokon Model #4911) should be placed at the boundaries between soil and competent rock (if the test shaft will extend through the overburden) and between the different rock layers as determined in the subsurface geotechnical test report. Consider the placement of proper type of strain gages at the bottom of the reaction socket if the

O-Cell will not be placed close to the base of the test shaft. The Geokon sister bars that house the Geokon Model # 4911 strain gage are 4 feet long. This means that the measurement should be taken 2 ft above the bottom and that the reaction socket has to be at least 4 feet long. Other types of vibrating wire strain gages could be made 2 feet long and work. However, non-uniform stress distribution exists over the cross section for 0.5 D to D below the O-Cell. Therefore, end strain gages could help if the reaction socket is long. For direct measurements of base resistance for test shafts with reaction socket, consider a second O-Cell, a commercial flat jack, or Geokon pressure cells placed right on the bottom of the reaction socket.

Construction of the Test Shafts

The construction plans should include language that empowers the Engineer (e.g., "subject to the Engineer's approval"), and that leaves some details (e.g., location of O-Cell and strain gages) open.

Eliminate the contribution of overburden to side resistance by installation of a temporary casing to top of rock and keep it there until the test is complete. The concrete would then be placed to 1 ft below bottom of casing. The contractor needs to ensure that the casing and concrete are not mechanically connected.

The test shafts should be constructed identical to the construction of other production shafts, in accordance with Section 503 of CDOT standard specifications, as recommended by the geotechnical engineers, after consulting the testing company, and as approved by the construction project engineer. The rate of rise of concrete should be at least 12 m (40 feet) per hour and the slump should be 7 – 8 inches in order to ensure that ground stresses have reestablished. For shafts embedded in very hard rocks, the compressive strength of the concrete of the test shaft should be at least 4000 psi or a higher value specified by the load test company to ensure that the concrete will not be crushed during the test. The unconfined compressive strength and stiffness of the concrete at time of load test should be determined in the lab and used to estimate the composite Young modulus of the shaft, E_c. Consider the use of LVWDTs to measure accurately the upward movement of the O-Cell and top of the shaft.

The thickness of the steel plates around the O-Cell (see Figure 3.1) should be 3 inches (2 inches employed for test shafts reported in this study), and the diameter of the bottom plate should be as close as possible to the diameter of the shaft.

Consider the use of four telltales to get compression data of the test shaft and measure any titling. It seems that there is evidence of small tilting of the O-Cell in almost all O-Cell load tests performed in rock. This is a problem that O-Cell vendors need to address, because if the cell tilts, the friction in the socket changes in some unpredictable way. The influence of this small tilting is unknown, but it should be minimized by having a high quality, uniform, and thick concrete pad below the O-Cell. Consider construction of a good base using high strength grout below the O-Cell to minimize tilting.

7.4.6 Data Collection at the Load Test Site

Three types of data should be collected:

1. A report for the subsurface geotechnical investigation at the load test sites as described in Task 2, including strength of all rock layers, and geological formation of the rock.
2. A report on the load test from the testing company. The data in this report should be analyzed very carefully (as described in Chapter 3) to obtain the f-w curve and f_{max} for rock layers around the test shaft and q-w curve and q_{max} for all rocks layer beneath the test shaft, and to construct the top load-settlement curve.
3. Cost, total and net saving of the load tests to the project, and other benefits of the load tests.
4. Location, layout, materials, and construction information of the test shaft, including:

- Map description of the location of the test shaft, including its coordinates (northing, easting, and elevation) if possible;
- Layout of the test shaft: diameter of the shafts (D), length of shaft in the overburden (L_o), and length of the bedrock socket (L), and depths to: groundwater level (GWL), competent bedrock, top and base of the shafts, the O-Cell, and the sets of strain gages.
- Date of construction the test shafts, and times when excavation/drilling started and completed, and times when concreting started and completed. Use these information to

estimate the time the hole was open (beginning of socket excavation up to the time of start of placement of concrete) and rate of concrete placemat (ft/hour).

- Methods/procedure for: excavation/drilling (e.g., auger or core barrel), cleaning the base and sides of the borehole, and placement of the concrete. Was any water tipped into the borehole to aid in removal of cuttings?
- Was the hole wet or dry? Possible sources of this water, if any, and its amount (could be measured from water accumulated at the base of shaft hole at end of the drilling operations).
- Information on the smoothness of the sides of the borehole in the rock including any estimates (even if rough) of the depth, width, and spacing of grooves. If possible, caliber the test pier borehole and obtain its roughness profile using laser devices or mechanical devices. At minimum, have the inspector use some sort of feeler (or visually) and determine the depth of the deepest grooves.
- Slump and rate of placement of the fresh concrete. The unconfined compressive strength and stiffness of the concrete at time of load test should be determined in the lab, and used to estimate the composite Young modulus of the shaft, E_c.

7.5 Task 5: Analysis of the Test Data and Information at the Load Test Sites

Select the best PM moduli (initial or reload) to represent the modulus of elasticity of the rock mass (E_m) for all categories of weak rocks (see Chapter 5). Finalize for each load test the E_m for all rock layers within and below the test socket.

Categories of weak rock should be defined. Each category will use a common design method and a definition of ultimate base and side resistance. As a starting point, four categories of weak rock are suggested based on the current study:

- Soil-like claystone bedrock with SPT between 20 and 100.
- Cohesive weak rock (SPT-N value >100 and q_{ui}<100 ksf).
- Cohesionless weak rock or soil-like sandstone (50<SPT-N value< 100), and
- Weak rock when q_{ui} less than 500 ksf, and SPT-N values >100 for granular-based rock, and q_{ui}>100 ksf for clay-based rock.

The last three categories of weak rocks are close to (but not the same as) those defined in the 1999 FHWA manual (see Chapter 2). The data analysis should determine the proper number of these categories and their descriptions (e.g., cohesive or granular, soil-weak rock-rock, 20<SPT-N value<100, 24 ksf $<q_{ui}<$ 100 ksf) and the proper definitions of ultimate resistance for each category (see Chapters 2 and 3). For each category of weak rock, the analysis should finalize the description of that category (the range of strength or SPT- N values) and the appropriate definitions of ultimate side resistance and base resistance. This definition is controlled by three factors presented in Chapter 3.

For each category of weak rock, the analysis should proceed as described in Chapter 5 (not necessarily in the order listed below):

1. Determine for each load test in the category of weak rocks: f_{max}, f_d, and the geotechnical test data (N, q_{ui}, E_m) for each rock layer within the test socket, and q_{max}, q_d and the geotechnical test data below the test socket. Then, determine the ratios E_c/E_m, E_i/E_m, and E_m/q_{ui} for each weak layer within and below the test socket, and the ratio L/D for the test shaft.
2. Determine the range of E_m/q_{ui} for that category and any expression to predict E_m from q_{ui}. Refine/validate the correlation equation suggested in Chapter 5 for the indirect estimation of q_{ui} from the PM test results.
3. Only for the soil-like claystone category, refine/validate the correlation equation developed between SPT-N value and q_{ui}. For the very hard rock, investigate if a correlation expression could be developed between the SPT-N values and strength data.
4. Define the normal construction of the test shafts in that rock category. Determine how smooth or rough a hole drills in the rock layer. If more than one drilling method is employed for that category of rocks, consider this issue in the analysis. In processing the test data, note deviations from normal construction practice (excessive water in the borehole, excessive time between drilling and concreting, excessively rough borehole due to sloughing of rock blocks, etc.) and treat the resulting test data accordingly (e. g., give the test an appropriately low weight in the development of final design relations).

5. Define the normal materials of the test shafts in that rock category. In processing the data, note also any significant deviation in the material in the test shaft (e.g., E_c) and treat the test data accordingly.
6. Define the typical range of D (e.g., D<5 ft.) and L/D (e.g., between 2 and 6) of test shafts in that category and note significant deviations from the typical range. In processing the data, note also any significant deviation in the D and L/D of the test shaft and treat the test data accordingly.
7. Determine the geological formations of the weak rocks in that rock category, and their influence on the accuracy of the design method. Rock type is probably a better delineator of behavior than geologic formation, unless the depositional process or age of one formation is very different from another for the same rock type. Determine if that category should be divided to more categories based on the geological formation.
8. Develop best-fit design equations between side or base resistance values and rock strength, as described in Chapter 5.
9. Investigate correlation equations between f_d, q_d and the SPT-N values, q_{ui} or E_m as described in Chapter 5. Then, present, for that category, the equations to construct a very approximate load-settlement curve as a function of N values, q_{ui} or E_m (Chapter 5). Provide a range of the service load settlements that should be expected based on the Strength Limit.
10. For the category of soil-like claystone, assess the CSB design method and the UCSB method recommended in this report (Chapters 5 and 6). For this category also, investigate the suitability of the AASHTO/FHWA design methods as described in the next item. Recommend the most accurate and cost effective design methods for weak rocks in that category that is based on the in-situ SPT and/or PM test results (not based on UC strength because it is difficult to collect on a routine basis reliable core specimens as presented in Chapter 6).
11. Explore, if appropriate, a tighter correlation between roughness (or side resistance) with rock type and drilling tool type.
12. Compare the prediction for f_{max} and q_{max} from the FHWA and AASHTO design methods described in Chapter 2 (and from any new design methods identified in Task 2) with f_{max} and q_{max} measured from the load tests (as in Chapter 5) for a number of rock layers. Note

possible influence of construction, E_c, D, L/D, presence of discontinuities, or the type of geological formation on the resistance values.

13. Identify the most accurate geotechnical axial design method for drilled shafts embedded in that category of weak rocks and complete the development of resistance factors for this method, by evaluating $\underline{\lambda}_{model}$ and COV_{model} for several candidate design methods and based on past experience in Colorado. The recommended resistance factor should be based on adequate level of subsurface geotechnical investigation.

7.6 Task 6. Write and Submit Final Technical Report, and Promote the Implementation of the Results

A draft technical report documenting the preceding research tasks will be prepared. The report will provide for each category of weak rocks a description of the process used to develop the new design relations and ϕ from the load test data and the geotechnical test data; the recommended design relations themselves; and a suggested process for the designer to employ in order to use the new design relations. The cost saving and benefits of the new design methods for drilled shafts will be demonstrated. Promote the implementation of these findings in CDOT and Colorado as presented in Section 7.4.1.

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APPENDIX A: NOTATION AND UNITS

A_b = base area of shaft in the rock socket in ft^2.
A_s = side area of shaft in the rock socket in ft^2.
ASD = allowable strength design method.
CFEM = Canadian Foundation Engineering Manual.
COV = coefficient of variation.
D = diameter of shaft in the rock socket (ft).
E = initial stiffness modulus of elasticity of in-situ rock as determined from PM test (ksf).
E_c = stiffness modulus of elasticity of the shaft (ksf or ksf/micro strain).
E_i = initial stiffness modulus of elasticity of intact rock as determined from UC test (ksf).
E_m = stiffness modulus of elasticity of in-situ (mass) rock (ksf).
E_r = reload modulus of elasticity of in-situ rock as determined from PM test (ksf).
E_u = unload modulus of elasticity of in-situ rock as determined from PM test (ksf).
FS = factor of safety.
f'c = concrete unconfined compressive strength (psi)
f, f_{max} = average unit side resistance and ultimate unit side resistance (ksf) pf the rock layer.
f_{all}, f_d = allowable side resistance and side resistance at displacement equal to 0.01 D (ksf).
f_{target} = target unit side resistance in the load test (ksf).
GWL = groundwater level.
IGM = intermediate geomaterial at the transition from soils to rock or weak rocks.
K_o = at-rest coefficient of earth pressure.
L = length of shaft in the rock socket (ft).
L_2, L_1 = lengths of shafts in the competent rock above and below the O-Cell (ft).
L_o = length of the shaft in the overburden (ft).
LRFD = load and resistance factor design method.
N = number of blows measured in the SPT to drive 12 inches or 1 foot (units of blows per = foot or bpf).
O-Cell = Osterberg Cell.
OCR = overconsolidation pressure.
P_f = horizontal yield pressure as measured in the PM test (ksf).
P_l = horizontal limit pressure as measured in the PM test (ksf).
P_o = in-situ horizontal pressure as measured in the PM test (ksf).
PM = pressuremeter.
Q_{all} = allowable resistance load of the shaft (kips).
Q_b, Q_s = base and side resistance load of the shaft (kips).
Q_d = resistance load of the shaft at displacement equal to 0.01 D (kips).
Q_{max} = ultimate resistance load of the shaft (kips).
q, q_{max} = unit base resistance and ultimate unit base resistance (ksf).
q_{all}, q_d = allowable base resistance and base resistance at displacement equal to 0.01 D (ksf).
q_{ui} = unconfined compressive strength of intact rock core specimens (ksf).
q_{um} = UC strength of rock mass as estimated indirectly from PM test (ksf).
q_{target} = target unit base resistance in the load test (ksf).
RF = roughness factor.
S_u = undrained shear strength.

SPT = standard penetration test in which 60% of the potential energy of the hammer is transferred to the top of the driving string.
UC = unconfined compressive.
w = displacement of the shaft relative to neighboring soil or rock.
w_{all} = settlement of the shaft at the service load.
v = Poisson's ratio.
α = factor relating side resistance to UC strength.
Λ = depth factor = $(1+L/D) \leq 3.4$ in the Canadian design method.
σ_n = normal effective stress between the concrete and borehole wall when compression loading is initiated.
σ_v = vertical effective stress.
ϕ = resistance factor required for the LRFD design method.
φ = angle of internal friction.
λ = bias factor.
$\underline{\lambda}$ = mean of the bias factor.
μ = factor such that $f_{max} = \mu\, q_{ui}^{0.5}$.

APPENDIX B: BEDROCK OF COLORADO'S FRONT RANGE URBAN CORRIDOR

Over much of the state, Colorado soils and bedrock are highly variable due to repeated episodes of mountain building, subsidence, igneous intrusion and extrusion, and glaciation. Within many provinces or trends, however, soil and bedrock character vary within definable limits due to similar geologic history, thus allowing for generalizations of their geotechnical properties. Emphasis in this appendix is on bedrock conditions likely to affect structures rather than total geologic aspects.

This study concentrates on bedrock likely to be encountered for drilled shaft foundations along the Urban Front Range Corridor (the Corridor). For our purposes, the Corridor is defined by a combination of geologic/geomorphic and population/transportation factors. From west to east, it covers the far eastern portion of the Rocky Mountains Front Range, the Frontal Hogback, and the valleys and uplands divisions of the Great Plains Western Piedmont Sub-Province. It extends from approximately Fort Collins on the north, including the Greeley area, to Pueblo on the south, thus capturing the State’s dominant population centers along Interstate 25.

Bedrock that exists along the Urban Front Range Corridor varies considerably as a result of the geologic processes that formed them. This section provides a brief overview of the bedrock types often found in the Corridor and discusses engineering properties that may affect drilled shaft foundations. Information presented in this section has been summarized from a document by Maytum and Hanneman prepared for another CDOT research study (laterally loaded drilled shafts for sound barriers). More detailed geologic descriptions are presented in that document.

B1. Bedrock

B1.1 Generalized Distribution

Except for transitional zones where bedrock is very highly weathered, the interface between soil and bedrock is usually fairly well defined along the Corridor. A major unconformity (period of non-deposition and/or erosion) due to uplift along the mountain front has separated younger soil from older bedrock. Bedrock units in the Corridor are distributed into four major settings (arranged as younger to older for the age of their generally included units):

1. Early Tertiary (Paleocene) coarse sandstone and conglomerate units, the youngest bedrock, are primarily limited to the central part of the Corridor forming major exposures in the Monument Highlands.
2. For valleys and uplands of the Western Plains Piedmont (the dominant portion of the Corridor), upper Late Cretaceous sedimentary rocks are intermittently exposed through soil cover throughout the northern and southern parts and comprise most of the bedrock likely to be encountered in foundations.
3. The mountain front belt includes a wide age range (Triassic to Pennsylvanian) of diverse sedimentary rocks that are exposed in a variably wide and locally intermittent band immediately east of the mountains. Jurassic to lower Late Cretaceous age shale and sandstone-dominant, tilted strata are intermittently well exposed along the narrow Frontal Hogback and as flatter lying outcrops in the Arkansas River valley near Pueblo.
4. Pre-Cambrian igneous and metamorphic rocks are exposed pervasively in mountainous areas along the west margin of the Corridor.

B1.2 Common Bedrock Types within the Corridor

Most drilled shafts are likely to be constructed where upper Late Cretaceous sedimentary rocks exist (item 2 above) which includes most of the Denver metro area, Fort Collins, Greeley, Boulder, Colorado Springs, and Pueblo areas. Major bedrock units include the Upper Dawson Arkose, Castle Rock Conglomerate; Denver, Arapahoe, & Lower Dawson Formations and the Laramie Formation, Fox Hills Sandstone, and Pierre Shale. Other bedrock types (items 1, 3, and 4 above) are discussed in the source document (for other CDOT Research) by Maytum and Hanneman.

B1.2.1 Upper Dawson Arkose and Castle Rock Conglomerate

Upper Dawson Arkose (Paleocene): Outcrops of this formation dominate the area along I-25 from the southern suburbs of Denver to northern Colorado Springs. Soil cover is generally limited to thin colluvium/residuum on gentle slopes and thin to moderate alluvium restricted to a few valleys. Younger (Oligocene) Castle Rock Conglomerate is common and highly visible in the area, but is limited to mesas/highlands above most major transportation routes. The Upper Dawson consists of an intricately interfingering, lensing series of members including quartz-feldspar sandstone, sandy and bouldery well cemented conglomerate, friable (weakly cemented) clay-rich sandstone, and claystone-siltstone. Well-cemented zones are very hard. Clayey horizons (including clay matrix sandstones) have high swell potential; less silty or sandy claystone layers may be very plastic when saturated. Other layers are considered stable to very stable.

B1.2.2 Denver, Arapahoe, and Lower Dawson Formations

The Denver, Arapahoe, & Lower Dawson Formations encompass a broad, arc-shaped band sweeping from northern Denver around the Monument Highlands with the general arrangement being Denver Formation dominant to the north (under most of the Denver metropolitan area), Arapahoe Formation in the center, and Lower Dawson Arkose to the south (around Colorado Springs). These units, although sometimes separately mapped, are largely age equivalent and interfinger with each other over long distances.

The Denver Formation mostly consists of claystone/shale, over most of the Denver area, with thinner interbeds of siltstone, weakly to well cemented sandstone, and infrequent conglomerate. Claystone/shale, as well as tuffaceous sandstone, are well noted for having major vertical and horizontal zones with high to very high swell potential; non-sandy claystone is frequently highly plastic when saturated. Claystone clays and ash-derived sandstone clays are montmorillonite rich (frequently termed "bentonitic") often including seams of nearly pure bentonite. Where unweathered, the formation includes a blue-green-gray claystone (and sandstone in some areas) locally known as the "Denver Blue". The "Denver Blue's" upper surface is not a stratigraphic horizon, but rather an irregular weathering/alteration zone that is often transitional. The bluish color has been observed to change to a predominantly grayish color after exposure to air.

The Arapahoe Formation is generally coarser than the Denver Formation. The two are frequently mapped as Denver-Arapahoe Undifferentiated in the Denver area. The formation is generally described as well stratified, interbedded claystone/shale, siltstone, sandstone, and conglomerate. A well-developed lower Arapahoe conglomerate is frequently only weakly cemented and is a significant aquifer. Conglomerate and sandstone units have variable low to moderate swell potential; siltstone and claystone/shale have moderate to high swell potential.

Lower Dawson Arkose also tends to be well interbedded with layers of conglomerate, coarse sandstone, shale, and silty fine sandy shale (termed “mudstone”). The coarser units usually have moderately well graded quartz and feldspar sands with granitic pebbles (“arkose”); local coal beds are noted. Clay rich and clay-dominant zones have moderate to very high swell potential and moderate to high plasticity, particularly in the Austin Bluffs area north of Colorado Springs.

B1.2.3 Laramie Formation, Fox Hills Sandstone, and Pierre Shale

Laramie Formation, Fox Hills Sandstone, and Pierre Shale formations occur in two broad situations: (1) intermittently exposed in moderately dipping beds east of the mountain front (immediately east of the Frontal Hogback) from Ft. Collins to Denver and (2) with thin soil mantles in gently dipping and near flat lying units in the Louisville area and along Interstate 25 between Colorado Springs and Pueblo.

The Laramie Formation is dominated by thinly bedded shale and siltstone with common hard to friable sandstone interbeds, lesser thin hard conglomerate, and lignitic to sub-bituminous coal beds. The formation is sandier in the lower portion. Most Laramie clays are dominantly kaolinitic with usually low to moderate swell potential; the middle third tends to be montmorillonitic with resulting high swell potential. Sandstones vary from weakly to well cemented.

Foxhills Sandstone units are cross-bedded and quartz sand-dominant. Relatively thin interbeds of claystone/shale, mudstone, and coal occur throughout. The sands are generally weakly cemented and friable; they are important aquifers with medium to high permeability, particularly north of Denver.

The Pierre Shale is a very thick, claystone/shale-dominant formation with numerous thin bentonite beds throughout. The bedrock units are almost always suspect for moderate to very high swell potential, medium to high plasticity, and low slope stability nearly everywhere they are encountered along the Corridor. Thin sandstone interbeds occur throughout the formation. Significantly thick sandstone members are present in several areas at different stratigraphic positions. Hard limestone masses (butte formers in outcrop) occur in the middle portion to the south. To the south, the middle portion also contains appreciable gypsum content that may affect sulfate-susceptible cement.

B1.3 Depth to Bedrock

Depths to the most common bedrock units are highly variable and depend on geologic processes that have occurred in an area and sometimes man's activities in the form of cut/fill operations. There is a large area of near surface bedrock in the Monument Highlands between southern Denver and northern Colorado Springs. Bedrock predominates the near surface geomaterials closer to the Rocky Mountain Front Range at the western edge of the Urban Front Range Corridor. In other areas of the Corridor, bedrock may exist near the surface or could be much deeper beneath alluvial deposits, sometimes in the range of 80 to 100 feet, or more. Generally, however, bedrock is likely to be encountered within the upper 50 feet of geomaterials at most sites. Bedrock is intermittently located within the upper few feet in many areas of the overall Corridor.

B1.4 Bedrock Hardness

The most common bedrock types in the Corridor are sedimentary deposits that have been heavily overconsolidated by as much as 1,000 feet of overburden that was subsequently eroded to the present day terrain. The previous overburden pressure, degree of weathering, and amount of cementation of sandstone or conglomerate, are the key factors that largely determine the hardness of the bedrock. Unconsolidated, undrained shear strengths in the Denver Formation range from 3 ksf to 30 ksf, and shear strengths in the Denver Blue range from 8 ksf to more than 30 ksf (Hepworth & Jubenville, 1981). Standard penetration test results generally range from about 30 to 80 for the non-Denver Blue bedrock, although some highly weathered areas may have SPT values in the teens. Denver Blue bedrock normally has SPT blow counts of at least 80. Denver Blue claystone/sandstone bedrock typically has blow count values in the range of 50/8” to 50/2”, and sometimes this is the first 6 inches of a drive that would normally not be recorded for a SPT. SPT refusal also occurs. Bedrock hardness varies from very low strength to moderate strength according to International Society of Rock Mechanics classification criteria. The weaker bedrock is better described in terms of soil consistency terminology in the range of very stiff to hard and tends to behave similar to heavily overconsolidated clay.