

**Report No. CDOT-DTD-R-2005-4
Final Report**

**COLORADO'S AXIAL LOAD TESTS ON DRILLED
SHAFTS SOCKETED IN WEAK ROCKS: SYNTHESIS
AND FUTURE NEEDS**

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September 2005

**COLORADO DEPARTMENT OF TRANSPORTATION
RESEARCH BRANCH**

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<p>16. Abstract: Drilled shaft foundations embedded in weak sedimentary rock formations (shale bedrocks) support a significant portion of bridges in Colorado. Since the 1960s, empirical design methods based on the blow counts of the standard penetration test (SPT) have been used to design drilled shafts in Colorado that deviate from the AASHTO LRFD design methods. The most accurate design method is to conduct load tests on test shafts, which are very expensive to perform. CDOT's strategic objective is to identify the most appropriate LRFD geotechnical axial design methods for Colorado's drilled shafts socketed in weak rocks that use test data obtained from cheaper and simpler geotechnical tests (e.g., SPT and unconfined compression test). To fulfill this objective, the measured resistance and settlement results of an adequate number of load tests on drilled shafts socketed in Colorado's shale bedrocks should be obtained and compared with predictions from design methods that use data of simpler geotechnical tests on the same bedrocks. In this report, Colorado's typical geological formations and construction methods for drilled shaft foundations are documented and discussed. Available information on Colorado's past axial load tests performed in the last 35 years on drilled shafts socketed in shale bedrocks are documented (e.g., test results from the load tests and from the simpler geotechnical tests, construction, materials, and layout of the test shafts). The load test results are analyzed and evaluated using Colorado SPT based design methods and methods recommended in CDOT Research Report 2003-6 and AASHTO/FHWA. The influence of conditions of the test shaft hole during construction (roughness and presence of water) on the measured resistances in the load tests is investigated. Based on the lessons learned from the work described above and the recommendations of CDOT Research Report 2003-6, Colorado's future needs for axial load tests on drilled shafts were established.</p> <p>Implementation: This report provides detailed recommendations on <i>When, Where, and How to perform future Colorado axial load tests on drilled shafts socketed in shale bedrocks</i>. The recommended axial load testing program would generate net <i>savings</i> to the construction project in addition to providing research data for improvement of the design methodology for drilled shafts. As a demonstrated example for applying these recommendations, the report provides all the details for the planning, design, construction, testing and analysis of the two Trinidad load tests. This report should serve as an important resource to CDOT engineers for both consideration and conducting of axial load tests on drilled shafts in CDOT's future construction projects.</p>					
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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
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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)

Fahrenheit (°F)	5(F-32)/9 (F-32)/1.8	Celcius (° C)	° C	1.8C+32	° F
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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

COLORADO'S AXIAL LOAD TESTS ON DRILLED SHAFTS
SOCKETED IN WEAK ROCKS:
SYNTHESIS AND FUTURE NEEDS

by

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Report No. CDOT-DTD-R-2005-4

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EXECUTIVE SUMMARY

Drilled shaft foundations embedded in weak sedimentary rock formations (e.g., Denver blue claystone shale bedrock) support a significant portion of bridges in Colorado. Since the 1960s, empirical design methods and “rules of thumb” have been used to design drilled shafts in Colorado that deviate from the AASHTO LRFD design methods. The most accurate design method for these shafts is to conduct load tests on test shafts, which are very expensive to perform. CDOT’s strategic objective is to identify the most appropriate LRFD geotechnical axial design methods for Colorado’s drilled shafts socketed in weak rocks that use test data obtained from cheaper and simpler geotechnical tests (e.g., SPT-N value from the standard penetration test, unconfined compressive strength, q_u from the unconfined compression test or UCT). To fulfill this objective, the measured resistance and settlement results of an adequate number of load tests on drilled shafts socketed in Colorado’s shale bedrocks (same as weak rocks) should be obtained and compared with predictions from design methods that use data of simpler geotechnical tests on the same shale bedrocks. CDOT Research Report 2003-6 thoroughly documented and analyzed the results of four Osterberg (O-Cell) load tests performed on soft to hard to very hard and massive shale bedrocks, and outlined a long-term plan of six tasks to fulfill the strategic objective listed above. This study was initiated to execute the following tasks in this plan:

- ❑ Compile and evaluate Colorado’s past axial load test information on drilled shafts.
- ❑ Determine CDOT’s future needs for performing new axial load tests on drilled shafts.

All the acquired Colorado load test information is presented, discussed, and evaluated in Chapters 3, 5, and Appendices A, D, and E. The following information (if available) is presented for the load tests: construction, materials, and layout of the test shafts; geological and geotechnical description of the foundation bedrock around and below the test shafts including the results of SPT, UCT, and pressuremeter tests; and results of the load tests. The compiled load tests (Table 3.1) are named after their location as: Fort Carson, 23rd Street Viaduct in Denver, I-270/I-76, SH82 Shale Bluffs in Pitkin County, T-REX along I-25 in Denver (I-225, County Line, and Franklin), Broadway Viaduct along I-25 in Denver, and Trinidad. The analyzed load test results support the use of the design methods recommended in CDOT Research Report 2003-6, and indicate that the Colorado SPT based design method for very hard shale bedrocks is very

conservative (high factor of safety) and leads to factors of safety lower than 2 for the soft claystone shale bedrocks.

The type and general locations of Colorado's bridges are discussed in Section 2.2 of this report. The geology of Colorado's highways is presented in Section 2.3 and the impact of geology on highway structure foundations is presented in Section 2.4. Tables 2.1 and 2.2 summarize the geological formations along Interstates I-25, I-70, and I-76, and along State Highway 50. Table 3.1 lists the compiled Colorado axial load sites on drilled shafts and the names of their geological bedrock formations. Tables 2.1 and 2.2 suggest that many of Colorado's highways alignments and all locations of loads tests are in the Sedimentary Cretaceous and Tertiary Formations.

Section 2.5.1 presents an overview of CDOT construction specifications and Colorado's methods for construction of drilled shafts. Section 2.5.3 presents recommendations to improve CDOT construction practices for drilled shafts (cleaning, drilling and concrete placement, use of water, slurry, and casing, wet holes, and shaft roughness). The construction methods for the test shafts employed in the T-REX and Broadway projects are described in detail in Section 2.5.2, and for test shafts in other locations are presented in Chapters 3 and 5, and Appendix A. Different levels of roughness and dry and wet shaft hole conditions were encountered in the load tests.

Implementation Statement: CDOT's Future Needs for Axial Load Tests on Drilled Shafts.

Based on the lessons learned from the work executed in this study (see Chapter 6) and the recommendations of CDOT Research Report 2003-6, CDOT's future needs for axial load tests on drilled shafts were established in Chapter 6: *Where, When, and How to perform future axial load tests on drilled shafts*. The recommended axial load testing program would generate net savings to the construction project (higher resistance values and lower factor of safety) in addition to providing research data for improvement of the design methodology for drilled shafts. Therefore, it is important to consider the following details in performing load tests on drilled shafts in CDOT's future construction projects.

1. Type, Locations, and Number of Future Load Tests. The Osterberg Cell (O-Cell) method should be considered in Colorado's future drilled shaft load tests until more cost-effective and

innovative load test methods become available. Examples of revisions to Section 503 of CDOT Construction Specifications when O-Cell load tests are employed are presented in Appendices B and C. It is also recommended to consider conventional load tests for low-capacity 1000 tons production shafts. Colorado's future load tests should be performed on shafts embedded in weak sedimentary rocks with unconfined compressive strength (q_u) up to 500 ksf. The load tests should not be limited to any particular sedimentary geological formations because it is believed that drilled shaft load test results in one sedimentary formation can readily be extrapolated to another sedimentary formation of similar in situ strength and type. Future load tests in Colorado *should be* drilled with augers having cutting teeth. This is the appropriate drilling method for the weak sedimentary rocks recommended for future Colorado load tests.

No future load tests are recommended for the soil-like sandstone shale bedrocks ($50 < \text{SPT-N} < 100$). No future load tests are recommended for the typical soft claystone shale bedrocks ($20 < \text{SPT-N} < 100$) with smooth shaft holes because they will not generate any savings to the construction project and will not lead to significant improvement in the accuracy of the recommended design methods (see Chapter 6 for justification). Future Colorado load tests should be considered in the following three categories of sedimentary weak rocks:

- I.** The firmer claystone shale bedrock ($50 < \text{SPT-N} < 100$) *when* shear rings are employed during construction for artificial roughening of the shaft hole sides. Shear rings in this kind of shale bedrock generate a measurable improvement in side shear capacity (leading to savings). A minimum of 7 (2 for wet and 5 for dry shaft holes) new load tests are recommended for determination of only the side resistance (not the base resistance).
- II.** Very hard claystone shale bedrock with SPT-N value > 120 bpf (or $> 50/5''$) and $q_u < 100$ ksf, and classified as rock-like material per Colorado Testing Procedure 26-90. A minimum of 7 load test sites (2 per site) are recommended (2 for wet and 5 for dry shaft hole conditions).
- III.** Very hard and massive shale bedrock with q_u less than 500 ksf, and SPT-N values > 100 for granular-based rock, and $q_u > 100$ ksf for clay-based rock, and classified as rock-like material per Colorado Testing Procedure 26-90. A minimum of 5 load test sites (two tests per site) are recommended (1 for wet and 4 for dry shaft hole conditions).

Two load tests are recommended per site for Categories II and III: one mainly to obtain base resistance data and one mainly to obtain side resistance data. Recommendations for inspection of roughening of shaft holes generated under normal drilling (expected for Categories II and III of rocks) and under artificial roughening (for Category I) are furnished.

2. When to Perform a Load Test? Section 4.2 provides step-by-step procedures on when it is cost-effective to consider load tests as part of the subsurface geotechnical investigation during different stages of the project development. Four conditions should be met:

- A large number of drilled shafts are required to support large bridges and with total construction costs for all phases of the project exceeding \$10,000,000 (e.g., corridor projects).
- Penetration depth of the drilled shafts is controlled by the axial loads, not the lateral loads.
- The type of weak rock should be one of the three categories previously listed.
- Net savings are expected based on cost-benefit analysis (described in Chapter 4).

3. How to Perform a New Load Test? Example? Chapter 4 presents comprehensive guidelines for planning, design, and construction of new load tests, especially O-Cell load tests, and analysis of the O-Cell load test results. These guidelines were applied in the Trinidad project. Chapter 5 provides specific details of all the steps employed for the planning, design, construction and analysis of the two Trinidad load tests.

TABLE OF CONTENTS

1.0	INTRODUCTION	1-1
1.1	Background and Study Objectives	1-1
1.2	Overview and Organization of the Report	1-3
2.0	TYPICAL GEOLOGICAL FORMATIONS AND CONSTRUCTION METHODS FOR DRILLED SHAFT FOUNDATIONS	2-1
2.1	Introduction	2-1
2.2	Types and General Locations of Colorado's Bridges.....	2-1
2.3	Geology of Colorado's Highways	2-3
2.4	Impact of Geology on Bridge Foundations.....	2-5
2.5	Colorado's Drilled Shaft Construction Methods	2-8
2.5.1	General.....	2-8
2.5.2	Examples of Construction Methods from the T-REX and Broadway Projects	2-10
2.5.3	Recommendations for Construction of Future Dilled Shafts	2-11
3.0	RECORDS OF COLORADO'S AXIAL LOAD TESTS ON DRILLED SHAFTS SOCKETED IN WEAK ROCKS	3-1
3.1	Overview	3-1
3.2	Trinidad Load Tests	3-3
3.3	T-REX and Broadway Load Tests.....	3-3
3.3.1	Assessment of Colorado SPT Based Design Method	3-5
3.3.2	Recommended Design Methods	3-6
3.4	Fort Carson Load Tests	3-7
3.5	The 23 rd Street Viaduct Load Tests.....	3-8
3.5.1	Subsurface Conditions and Strength Characteristics of the Bedrock	3-9
3.5.2	Test Program and Construction.....	3-9
3.5.3	Test and Analysis Results	3-10
3.6	The I-270/I-76 Load Tests	3-11
3.6.1	Subsurface Conditions and Strength Characteristics of the Claystone	3-12
3.6.2	Construction of Test Shafts	3-13
3.6.3	Testing and Analysis Results	3-14

3.7	SH 82 O-Cell Load Tests (Pitkin County)	3-15
3.7.1	Test 1 (Caisson 47 A, Shale Bluffs).....	3-15
3.7.2	Test 2 (at Pier No. 2)	3-16
4.0	GUIDELINES FOR CONDUCTING COLORADO'S NEW AXIAL LOAD TESTS	
	ON DRILLED SHAFTS	4-1
4.1	Introduction	4-1
4.2	Requirements for Cost-Effective Load Tests	4-3
4.2.1	Meeting the 1 st and 2 nd Requirements	4-4
4.2.2	Meeting the 3 rd Requirement.....	4-4
4.2.3	Meeting the 4 th Requirement.....	4-5
4.2.4	Finalize the Scope of Work for the Geotechnical Investigation and Design Work	4-6
4.3	Guidelines for Planning, Design, and Construction of New Load Tests on Drilled Shafts (Level 3 Design)	4-7
4.3.1	Purpose and Promotion of Load Tests	4-7
4.3.2	Location and Number of Load Tests.....	4-7
4.3.3	Type of Test Shafts (Production or Sacrificial)	4-8
4.3.4	Type, Features, and Costs of Load Tests	4-9
4.3.5	Geotechnical Investigation around the Test Shaft	4-12
4.3.6	Design of the O-Cell Load Test	4-13
4.3.7	Instrumentation of the Test Shafts	4-16
4.3.8	Construction of the Test Shafts	4-17
4.3.9	Data Collection at the Load Test Site	4-19
4.4	Analysis of Osterberg Cell (O-Cell) Load Test Results	4-20
4.4.1	Determination of the Load Transfer Curves	4-22
4.4.2	Definitions of Tolerable Settlement, Ultimate Unit Base Resistance, and Ultimate Unit Side Resistance	4-24
4.4.3	Construction of the Equivalent Top Load-Settlement Curve from the Results of the O-Cell Load Test	4-25
4.4.3	Construction of a Simple Equivalent Top Load-Settlement Curve from the Results of Simple Geotechnical Tests	4-26

5.0	TRINIDAD LOAD TESTS: EXAMPLE OF APPLICATION OF THE PROPOSED GUIDELINES FOR CONDUCTING NEW AXIAL LOAD TESTS	5-1
5.1	Overview	5-1
5.2	Subsurface Conditions and Strength Characteristics of the Bedrock	5-1
5.3	Recommendations for Design and Construction of the O-Cell Load Tests.....	5-2
5.3.1	Selecting L and D for the Test Shafts	5-3
5.3.2	Selecting the Capacity and Location of the O-Cell.....	5-4
5.3.3	Recommendations for the Construction and Instrumentation of the O-Cell Load Tests	5-6
5.4	Construction of Test Shafts.....	5-7
5.5	Load Testing Results and Analysis	5-8
5.5.1	Side Resistance.....	5-8
5.3.2	Base Resistance.....	5-9
5.6	Design Changes and Benefits Based on Load Test Results.....	5-12
6.	SUMMARY AND CONCLUSIONS.....	6-1
6.1	Overview	6-1
6.2	Summary and Evaluation of Colorado’s Records of Axial Load Tests on Drilled Shafts	6-3
6.2.1	Types of Load Tests.....	6-3
6.2.2	Discussion of Colorado's Past Load Tests	6-4
6.3	Lessons Learned for Planning Future Load Tests.....	6-7
6.3.1	Lessons Learned from Colorado's Past Load Tests	6-7
6.3.2	Lessons Learned from the Investigation on Construction of Drilled Shafts.....	6-10
6.3.4	Lessons Learned from the Investigation on Geology of Colorado’s Bedrocks	6-14
6.4	CDOT's Future Needs for Axial Load Tests on Drilled Shafts.....	6-16
6.4.1	Type, Location, and Number of Future Load Tests	6-16
6.4.2	Guidelines for Conducting and Analyzing Colorado's Future Axial Load Tests	6-20

REFERENCES.....7-1

**APPENDIX A: SUMMARY INFORMATION OF COLORADO'S LOAD TESTS ON
DRILLED SHAFTS..... A-1**

**APPENDIX B: SAMPLE GUIDE SPECIFICATIONS FOR OSTERBERG CELL LOAD
TESTING OF DRILLED SHAFTS (By LOADTEST, Inc.)..... B-1**

**APPENDIX C: REVISIONS OF SECTION 503, OSTERBERG CELL LOAD TEST (from
Broadway's Construction Plans and Specifications Project)..... C-1**

**APPENDIX D: INFORMATION FROM PAST COLORADO LOAD TESTS (from Load
Test Reports listed in the References)..... D-1**

**APPENDIX E: RESULTS OF LOAD TEST INVESTIGATION IN THE TRINIDAD
PROJECT E-1**

LIST OF FIGURES

Figure 4.1	Photo of the O-Cell Placed in the Broadway Test Shaft	4-27
Figure 4.2	Results of O-Cell Load Test at the County Line Test Shaft.....	4-28
Figure 4.3	Unit Side Resistance vs. Upward Movement for the Broadway Test Shaft.....	4-28
Figure 4.4	Unit Side Resistance vs. Upward Movement in the Entire Bedrock Socket: Franklin and Broadway Test Shafts.	4-29
Figure 4.5	Unit Base Resistance vs. Settlement: Franklin and Broadway Test Shafts.....	4-29
Figure 4.6	Extracted Load-Settlement Curves: Franklin and Broadway Test Shafts	4-30
Figure 4.7	Extracted Load-Settlement Curves: I-225 and County Line Shafts	4-30
Figure 5.1	Unit Side Resistance vs. Upward Movement in the Bedrock Socket of the Trinidad Test Shafts.....	5-11
Figure 5.2	Unit Base Resistance vs. Settlement for the Trinidad Test Shafts	5-11

LIST OF TABLES

Table 2.1	Rural Interstate Geology (I-25, I-70, and I-76)	2-6
Table 2.2	SH-50 Geology	2-7
Table 3.1	Summary of Colorado's Available Load Tests on Drilled Shafts.....	3-2
Table 4.1	Typical Construction, Materials, and Layout Data for the Test Shafts	4-21
Table 5.1	Results of the Trinidad Load Tests as Reported by Loadtest, Inc.(2003)	5-9
Table 5.2	Geotechnical Design Parameters for the Trinidad's 4 feet Diameter Drilled Shafts before and after the O-Cell Load	5-12
Table 6.1	Available Load Test Information for the Types of Shale Bedrocks Recommended in Future Load Testing.....	6-20

1 INTRODUCTION

1.1 Background and Study Objectives

Drilled shaft foundations embedded in weak rock formations (e.g., Denver blue claystone and sandstone) support a significant portion of bridges in Colorado. Drilled shafts derive support by embedment in these weak rocks, typically found at relatively shallow depth in Colorado. The contribution of overburden to the drilled shaft axial capacity is often ignored. Thorough geotechnical design of a drilled shaft requires determination of a top load-settlement curve, q_{\max} (ultimate unit base resistance) of the rock layer beneath the shaft, f_{\max} (ultimate unit side resistance) of the rock layers around the shaft, the load factor and resistance factor (ϕ) in the LRFD (Load and Resistance Factor Design) method, and the factor of safety (FS) in the allowable stress design (ASD) method.

The most accurate design method to estimate q_{\max} , f_{\max} , and settlements of drilled shafts is to conduct load tests on test shafts constructed as planned in the construction project. The load tests are expensive and therefore only considered in large projects. However, the very accurate design information obtained from the load tests could be used: 1) to design production shafts with more confidence (lowest FS or highest ϕ) and accuracy (leading to less conservative estimates of q_{\max} , f_{\max} , and settlements in most cases), resulting in significant savings to the project, and 2) as research data to improve the accuracy of simpler analytical design methods for drilled shafts that use data of simpler geotechnical tests, mainly SPT (Standard Penetration Test), UCT (Unconfined Compression Test), and the pressuremeter test or PMT. The in situ SPT provides information on the driving resistance of the weak rock in term of blow counts per foot (bpf), or N-value. The laboratory UCT is employed to determine both the unconfined compressive strength (q_u) and Young's modulus (E_i) of intact rock cores. Due to the presence of discontinuities (soft seams and/ or joints) in the rock mass, intact core strength and stiffness as measured in the UCT could be larger than the rock mass strength and rock mass stiffness (E_m). Information on the RQD (Rock Quality Designation) and conditions and structures of joints are utilized to develop reduction factors for strength and stiffness values obtained from laboratory testing on intact cores. From the in situ pressuremeter tests on weak rocks, the stress-strain curve

and E_m can be measured directly and the unconfined compressive strength can be estimated indirectly (see Abu-Hejleh et. al., 2003 for complete details).

Since January 1, 2000, it has been the policy of Colorado Department of Transportation (CDOT) to incorporate the new and more rational AASHTO LRFD method for the design of its structures, including drilled shafts. 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 Standard Penetration Test (SPT) and deviate from the AASHTO LRFD Design Methods. The margin of safety (or ϕ) and expected shaft settlement are unknown in these methods, both needed to implement the AASHTO LRFD design methods. AASHTO offers design methods that are based on the results of UCT for rocks not the results of SPT as in CDOT method. However, AASHTO methods are developed for conditions that may be different from those encountered in Colorado (i.e., not for the weak sedimentary rocks often encountered in Colorado). To address all these needs and shortcomings, the CDOT’s strategic objectives for Colorado’s drilled shafts socketed in weak rocks were identified (Abu-Hejleh et. al., 2003) as to

- Identify the most appropriate and accurate geotechnical design methods to predict the ultimate axial resistance and settlements of Colorado’s drilled shafts socketed in weak rocks that are based on simple and routine geotechnical tests (SPT, UCT, and PMT).
- Identify the most appropriate resistance factors (ϕ) needed per the LRFD for the design methods identified in the 1st Objective.

To fulfill these objectives, the measured resistance and settlement results of an adequate number of load tests on drilled shafts socketed in Colorado’s shale bedrocks should be obtained and compared with predictions from design methods that use test data of simpler and more common geotechnical tests (SPT, UCT, and PMT) on the same shale bedrocks. CDOT Research Report 2003-6 (Abu-Hejleh et. al., 2003), titled “Improvement of the Geotechnical Axial Design Methodology for Colorado’s Drilled Shafts Socketed in Weak Rocks,” thoroughly documented and analyzed the results of four O-Cell load tests performed in 2002 as part of the T-REX and Broadway Viaduct projects. 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 subsurface geotechnical investigation was performed on the weak rock at the load test sites. This included 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 common geotechnical tests (SPT, UCT, and PMT), 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 common geotechnical tests, and 3) assessment of the CDOT and AASHTO design methods.

CDOT Research Report 2003-6 also outlined a long-term plan with six tasks to fulfill the strategic objectives listed above. This study was initiated to execute the following tasks in this plan:

- ❑ Compile and evaluate all available Colorado's past and reliable axial load test information.
- ❑ Determine CDOT's future needs for performing new axial load tests on drilled shafts in CDOT future construction projects.

1.2 Overview and Organization of the Report

The types and general locations of Colorado's bridges are discussed in Section 2.2. Colorado's typical geological formations and construction methods for drilled shaft foundations are presented in Chapter 2. The Geology of Colorado's highways is presented in Section 2.3 and the impact of geology on highway structure foundations is presented in Section 2.4. Tables 2.1 and 2.2 summarize the geological formations along Interstates I-25, I-70, and I-76, and along State Highway 50. Section 2.5.1 presents an overview of CDOT construction specifications and Colorado's methods for construction of drilled shafts. Section 2.5.3 presents recommendations to improve this practice. The Construction methods for the test shafts employed in the T-REX and Broadway projects are described in detail in Section 2.5.2.

All the acquired Colorado load test information is presented, discussed, and evaluated in Chapters 3, 5, and Appendices A, D, and E. The following information (if available) is presented

for the load tests: construction, materials, and layout of the test shafts; geological and geotechnical description of the foundation bedrock around and below the test shafts including the results of SPT, UCT, and PMT; and results of the load tests. The compiled load tests (Table 3.1) are named after their location as: Fort Carson, 23rd Street Viaduct in Denver, I-270/I-76, SH82 Shale Bluffs in Pitkin County, T-REX along I-25 in Denver (I-225, County Line, and Franklin), Broadway Viaduct along I-25 in Denver, and Trinidad. Some reported information (e.g., results of load tests and of the geotechnical investigation) in the Testing Reports are furnished in Appendix D for the 23rd Street, I-270/I-76 and SH82 Shale load tests, and in Appendix E for the Trinidad load test. Load tests that have most of the information needed for analysis and evaluation are summarized in seven tables in Appendix A.

Chapter 4 presents comprehensive guidelines for planning, design, and construction of new load tests, and analysis of the Osterberg Cell (O-Cell) load test results. Sample Guide Specifications for Osterberg Cell Load Testing of Drilled Shafts are presented in Appendix B. Revision of Section 503 of CDOT Standard Specifications to incorporate the Osterberg Cell Load Test in the Broadway construction project is presented in Appendix C. Section 4.2 provide step by step procedures on when it is cost-effective to consider load tests as part of the subsurface geotechnical investigation during different stages of the project development. The recommended guidelines were applied in the Trinidad project. Chapter 5 provides specific details of all the steps employed for the planning, design, construction and analysis of the two Trinidad two load tests.

Chapter 6 provides a brief summary of all work performed in this study and the lessons learned for future planning of Colorado's axial load tests shale socketed in weak rocks from:

- Colorado's past load tests.
- Investigation on the construction methods and observations for Colorado's load test shafts.
- Investigation on the geology of Colorado's bedrock formations.

Based on these lessons and recommendations of CDOT Research Report 2003-6, CDOT's future

needs for axial load tests on drilled shafts are also presented in Chapter 6: *Where, When, and How to perform future axial load tests on drilled shafts*. Future Colorado load tests should be considered in three categories of sedimentary weak rocks that are presented in Chapter 6. Available load tests information on these three categories of weak rocks are identified and ranked. All details required to conduct future load tests in these three types of weak rocks are presented, including the minimum number of load tests.

2. TYPICAL GEOLOGICAL FORMATIONS AND CONSTRUCTION METHODS FOR DRILLED SHAFT FOUNDATIONS

2.1 Introduction

The following information, needed for planning of future load tests in Colorado, is summarized in this chapter:

1. Types and general locations of Colorado's Bridges.
2. Geology of Colorado's Highways and impact of geology on bridge foundations.
3. An overview of Colorado's methods for construction of drilled shafts and recommendations to improve this practice. Examples of construction procedures employed in the T-REX and Broadway projects are also presented.

The geological bedrock formations and construction methods for the compiled Colorado's axial load tests on drilled shafts are presented in subsequent chapters and appendices. The study findings and recommendations for consideration of the geology and construction factors in the planning of future axial load tests on drilled shafts are presented in Chapter 6.

2.2 Types and General Locations of Colorado's Bridges

Bridges are found on virtually all Colorado State Highways. Most of the bridges on 2-lane rural highways are at drainage crossings, with most having relatively small span lengths. Larger bridges are found at railroad and river crossings and on divided highways. The most numerous and largest bridges are associated with limited access highways (interstates/freeways), especially at interchanges. Thus, in terms of Colorado highways, the greatest number of large bridges will be associated with limited-access highway corridor improvement projects.

Drilled shafts extending into soft to firm to very hard claystone/sandstone bedrock often provide bridge support. Drilled shafts are used to support bridge piers and abutments. When a suitable bearing layer is at modest depth, drilled shafts are usually the most economical deep foundation for support of bridge piers. Drilled shafts can also be used for abutment support. However,

driven H-Piles are often used for abutment support, especially for integral decks and abutments where abutment flexibility is desirable.

Drilled shafts are also used as earth retention structures. However, in these applications, drilled shafts are used predominantly for their lateral support capability. Significant axial loads are usually not associated with earth retention structure applications.

The main Colorado highway corridors connect the principal cities/population centers, and tend to follow geographic features such as rivers and mountain passes. Examples of the rural Interstate Highways are as follows:

- I-25 extends from New Mexico to Wyoming at the foot of the Front Range of the Rocky Mountains. Much of the way, the highway alignment is within a few miles of the boundary between the Great Plains and the Rocky Mountain Physiographic Provinces. Along the way, portions of the I-25 alignment are controlled by rivers and streams such as Fountain Creek, Monument Creek, Plum Creek, and The South Platte River, as well as Monument Pass.
- The alignment of I-70 from Denver to Grand Junction is controlled by mountain passes, rivers and streams. The mountain passes are Loveland Pass (the Eisenhower Tunnel) and Vail Pass. East of the Eisenhower Tunnel, which is at the Continental Divide, the highway parallels Clear Creek. Between the Eisenhower Tunnel and Vail Pass, the highway parallels Short Creek and Ten Mile Creek. West of Vail Pass the highway parallels the Colorado River and its tributaries, Vail Creek and the Eagle River all the way to the Utah border.
- I-76 from Denver to Ft. Morgan to Julesburg by the South Platte River.

Other major highways are also controlled by geographic features. Examples include:

- SH-50 from Monarch Pass to Holly by the Arkansas River and its tributaries.
- Virtually all highways through the Front Range, Sangre de Cristo Mountains, the Gore Range, West Elk Mountains, the San Juan Mountains, etc. are controlled by mountain passes and stream/river valleys.

2.3 Geology of Colorado's Highways

Drilled shafts are used for support of highway bridge structures throughout Colorado. Geologic conditions in which drilled shafts are used vary from alluvium and weak formational materials to very hard sedimentary, metamorphic and igneous rocks. Table 2.1 and 2.2 summarize the geological formations along Interstates I-25, I-70, and I-76, and along State Highway 50. SH-50 was selected because it is representative of many Colorado Highways as it crosses the center of the State from Utah to Kansas, including valleys in the west, the Rocky Mountains, and the Eastern Plains. The feasibility of using drilled shafts in these formations is also presented in these tables. Hard rock is typically found in the igneous/metamorphic cores of the principal ranges. However, even these rocks can be highly variable, especially in the volcanic rock in the San Juan Mountains.

As highways extend away from mountain passes, they usually parallel mountain streams and rivers. If the highway follows the valley floor, the alignment is likely to be underlain by alluvium extending to bedrock. Where the highways bypass the valley floors, they often are cut into the generally hard rock. On the flanks of the mountains and in broad intermountain valleys, the highways leave the hard mountain cores and extend over softer sedimentary bedrock. East of the Front Range virtually all of Eastern Colorado is underlain by sedimentary bedrock. Sedimentary bedrock is not a unique material. It can range from very hard, cliff-forming sandstones and conglomerates to very soft shale.

While highways must traverse whatever rock type is along the way, highways tend to follow locations with gentle slopes. These locations are often underlain by softer sedimentary geologic formations such as shale. Examples are the Pierre Shale that underlies I-25 in Trinidad, from south of Pueblo to Colorado Springs, and north of Denver; and the Mancos Shale that underlies SH-82 in the Roaring Fork Valley, I-70 west of Glenwood Springs, and SH-160 west of Durango. Some of the major rivers have deeply incised into the underlying bedrock, with the incised channels backfilled with alluvium. These conditions exist along the South Platte River northeast of Denver and have impacted structures along SH-85, I-76, and where other state

highways cross the river. In addition, the Colorado River is locally deeply incised, which has influenced highway structure foundations in Glenwood Springs, for example.

Based on the above and information listed in Tables 2.1 and 2.2., it can be concluded that most of Colorado's shafts (existing and future) are underlain by Late Cretaceous age sedimentary rock formations that in many locations have engineering properties of "weak rock." Drilled shafts derive support by embedment in these weak rocks, typically found at relatively shallow depth. These sedimentary 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. Three prevalent geologic formations for the weak rocks in Colorado are the Pierre and Denver Formations (Turner et al., 1993) and the Mancos formations. Abu-Hejleh et. al. (2003) provided a geotechnical and geological description of bedrock formations likely to be encountered in the Denver metropolitan area (e.g., Pierre and Denver Formations) and other populated areas along the Front Range Urban Corridor in Colorado. The Mancos Shale (not described by Abu-Hejleh et. al., 2003) underlies large portions of Western Colorado, especially in the broad river valleys. The Mancos Shale is a very thick, claystone/shale dominant formation. The bedrock units are usually dark grey to black, and almost always suspect for moderately to very high swell potential, medium to high plasticity, and low slope stability. There are numerous landslides in Western Colorado on slopes underlain by the Mancos Formation. There are occasional sandstone beds within the clay shale, and one significant sandstone member, the Ferrin Sandstone, within the Formation. Where the Mancos Formation is exposed at the surface, such as immediately north of Grand Junction, and in the Gunnison River Valley between Montrose and Grand Junction, there is often very little vegetation and the ground has a "bad lands" appearance. The Mancos formation is generally equivalent to the Pierre Formation found in the eastern part of Colorado. Both were deposited in the Cretaceous Sea. However, the base of the Mancos rests directly on the Dakota Formation, whereas the Pierre is separated from the Dakota by the Colorado Group and the Niobrara Formations. Both are overlain by sandstones; the Mancos by the Mesaverde Formation, and the Pierre by the Fox Hills Sandstone.

2.4 Impact of Geology on Bridge Foundations

The local geology at a bridge site will largely determine the bridge foundation type. For example, where sound rock is found at foundation level, deep foundations may not be needed. Conversely, where roadways parallel mountain streams, bridges are often supported on deep foundations extending through alluvium to underlying bedrock.

Where bridges are founded over deep alluvium, the foundation type will depend on local conditions and structural design requirements. Several different bridges are illustrative of these conditions.

- The new bridge over the Colorado River at West Glenwood is founded on piles driven into very dense bouldery alluvium. It would not have been practical to attempt to drill shafts into the very hard cobbles and boulders.
- Where SH-39 crosses the South Platte River near Goodrich, there was almost 100 feet of alluvium over bedrock. Because of the nature of the alluvium and the bridge structural requirements, the bridge was supported on drilled shafts. Experience during bridge construction suggests that this depth to bedrock may be a practical limit to drilled shaft construction using normally available construction equipment.
- Where SH-71 crosses the South Platte River near Snyder, about 200 feet of alluvium overlies bedrock. Drilled shafts were not practical at this location. Rather, the bridge was supported on driven piles.

The variability of sedimentary rock requires that site-specific investigations be performed to assess local conditions and appropriate foundation types. For example, high capacity drilled shafts can be used to support bridges in the harder Denver Blue Formation. Even deep foundations would not be necessary if shallow sound sedimentary rock exists at foundation level. However, elsewhere the Denver Formation may be highly weathered and not much better than hard clay, limiting drilled shaft capacities. 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 shear strengths are higher in the “blue” claystone that underlies downtown Denver. Abu-Hejleh (3) reported unconfined compressive

strengths greater than 300 ksf for rocks in the Denver formation and greater than 400 ksf for rocks in the Pierre formations. Significant variability was also noticed in the Mancos Shale as will be discussed next chapter based on the reported load test results for the SH-82 project.

Table 2.1: Rural Interstate Geology (I-25, I-70 and I-76)

Notes:	1) The tabulation of Geology in this table is for general reference only.			
	It is not intended to be precise or definitive.			
	2) Geologic Age: PreC - PreCambrian, P - Paleozoic, K - Cretaceous, T - Tertiary			
	Q – Quaternary			
	3) Predominant rock type: S - Sedimentary, I - Igneous, M - Metamorphic			
Highway	Mileposts	Geologic Formation(s) and Age	Rock Type	Drilled Shaft Feasibility
I-25	0-14	K - Raton Formation	S	Yes
	14 - 67	K - Pierre Formation	S	Yes
	67 - 87	K - Niobrara, Dakota and Purgatoire Formations	S	Yes
	87 - 145	K - Pierre Formation	S	Yes
	145 - 148	K - Laramie Formation	S	Yes
	148 - 190	T - Dawson Formation	S	Yes
	190 - 225	TK - Denver Formation	S	Yes
	225 - 243	K - Laramie, Fox Hills Formations	S	Yes
	243 - 288	K - Pierre Formation	S	Yes
	288 - 295	K - Laramie and Fox Hills Formations	S	Yes
	295 - 300	T - White River and Ogallala Formations	S	Yes
I-70	0 - 10	P - Kayenta and Wingate Formations	S	Yes
	10 - 44	K - Mancos Formation	S	Yes
	44 - 60	K - Mesaverde Formation	S	Yes
	60 - 100	T - Wasatch and Ohio Creek Formations	S	Yes
	100 - 109	K - Mesaverde Formation	S	Yes
	109 - 115	P - Maroon Formation	S	Yes
	115 - 130	PreC, P - Older, generally hard, sedimentary formations, and Basement rocks	S,I,M	Unlikely
	130 - 155	P - Eagle Valley Evaporite	S	Yes
	155 - 160	K - Mancos and Dakota Formations	S	Yes
	160 - 170	P - Eagle Valley and Evaporite Formations	S	Yes
	170 - 185	P - Minturn and Belden Formations	S	Yes
	185 - 200	PreC - Basement Rocks	I,M	Unlikely
	200 - 208	TK - Pierre and other Formations	S	Yes
	208 - 255	PreC - Basement Rocks	I,M	Unlikely

	255 - 260	K - Dakota and Arapahoe Formations	S	Yes
	260 - 295	TK - Denver Formation	S	Yes
	295 - 325	TK - Lower Dawson Formation	S	Yes
	325 - 355	K - Laramie Formation	S	Yes
	355 - 370	K - Pierre Formation	S	Yes
	370 - 450	T - Ogallala Formation	S	Yes
I-76	0 - 40	TK - Denver Formation	S	Yes
	40 - 60	K - Laramie Formation	S	Yes
	60-64	K - Fox Hills Formation	S	Yes
	64 - 80	K - Pierre Formation	S	Yes
	80 - 90	Q/K - Alluvium over Pierre Formation	S	Yes*
	90 - 140	Q/K - Eolian sand and alluvium over Pierre Formation	S	Yes*
	140 - 162	Q/T - Eolian sand over White River Formation	S	Yes*
	162 - 167	T - White River Formation	S	Yes
	167 - 180	T - Ogallala Formation	S	Yes
	180 - 185	T - White River Formation	S	Yes
		* Other foundation types may be appropriate in locally deep alluvium		

Table 2.2: SH-50 Geology

Segment	Geologic Formation(s) and Age	Rock Type	Drilled Shaft Feasibility
Utah Border to Grand Junction	P - Kayenta and Wingate Formations	S	Yes
Grand Junction to Big Blue Creek	K - Mancos Formation	S	Yes
Big Blue Creek to Sapinero	PreC, T - Tertiary Volcanics and Basement Rocks	I,M	Locally Variable
Sapinero to Steuben Creek	K - Various shales and sandstones	S	Probably
Steuben Creek to Gunnison	Q/TK - Alluvium over Cretaceous Shales and Tertiary Volcanics	S,I	Probably
Gunnison to Doyleville	Q/TK PreC - Alluvium over Tertiary Volcanics, Cretaceous Shales and Basement Rocks	S,I,M	Locally Variable
Doyleville to Needle Creek	K - Mancos Formation	S	Yes
Needle Creek to 3 mi E of Monarch Pass	T PreC - Basement Rocks and Tertiary Volcanics	I,M	Locally Variable
3 mi E of Monarch Pass to 6 mi E of Monarch Pass	P - Paleozoic Sedimentary Rocks, some very hard	S	Unlikely
6 mi E of Monarch Pass to Maysville	PreC - Basement Rocks	I,M	Unlikely

Maysville to Salida	Q/T - Alluvium over Dry Union Formation	S	Probably
Salida to 2 mi W of Parkdale	Q/PreC - Alluvium over Basement Rocks	S,I	Locally Variable
2 mi W of Parkdale to Parkdale	K - Cretaceous Shale	S	Probably
Parkdale to 4 mi W of Canon City	PreC - Basement Rocks	I,M	Unlikely
4 mi W of Canon City to Canon City	PK - Steeply dipping hogback rocks	S	Locally Variable
Canon City to 3 mi W of Penrose	K - Pierre Formation	S	Yes
3 mi W of Penrose to Pueblo West	K - Niobrara Formation	S	Yes
Pueblo West to Boone	K - Pierre Formation	S	Yes
Boone to Fowler	Q/K - Alluvium over Pierre Formation	S	Yes
Fowler to La Junta	Q/K - Alluvium over Niobrara Formation	S	Yes
La Junta to Lamar	K - Carlisle Shale, Greenhorn Limestone and Graneros Shale	S	Yes
Lamar to Kansas Border	Q/K - Alluvium over Carlisle, Greenhorn and Graneros Fms	S	Yes

2.5 Colorado's Drilled Shaft Construction Methods

The influence of construction method of the drilled shafts on the vertical and lateral capacity of the drilled shafts is discussed in the FHWA design manual (O'Neill and Reese, 1999) and by Abu-Hejleh et. al. (2003). Developed design methods for drilled shafts based on load test results are applicable to other production shafts only if the construction methods employed in the production shafts are similar or better than those applied in the load test shafts. In Colorado, the axial resistance of the overburden soil is neglected in the design and the resistance is assumed to be entirely derived from the bedrock. Therefore, the emphasis should be on construction methods in bedrock sockets.

2.5.1 General

Section 503 of CDOT Standard Specifications presents CDOT standard requirements for good construction practices of Colorado's drilled shafts. It can be accessed online at <http://www.dot.state.co.us/DesignSupport/Construction/1999book/specb500.pdf>. 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. Any deviation from CDOT Standard should be documented and accounted for either through changes in the geotechnical design of drilled shafts or by the Project Engineer in the field (i.e., increase of bedrock socket length).

Drilled shafts in Colorado are usually installed with auger drills. These can be mounted directly on the supporting vehicle, or can be mounted on cranes. Hydraulic power is typically used to rotate a central steel bar, known as a Kelly bar. Augers with one to three flights are typically installed at the bottom of the Kelly bar. Cutting teeth mounted at the bottom of the augers assist in advancing the augers. The cutting teeth depend on the material being excavated, with blades for softer materials, and hardened points for harder materials. Other less common drilling tools include buckets with cutting teeth, core barrels and breaker bars to extend shafts through very hard layers or boulders, etc.

The methods of advancing the augers depend on subsurface conditions. Where conditions allow, the drill hole is advanced into the supporting materials in the dry. To accomplish this, casing is often extended through water bearing and/or sloughing overlying materials. Groundwater often seeps into drill holes. The CDOT Standard Specifications for Road and Bridge Construction address this issue. If there are 2-inches or less water in the bottom of the shaft, it is considered a dry hole. With more than 2-inches of water, it is defined as a wet hole requiring underwater placement and concrete richer in cement. Slurry drilling is sometimes used as an alternative to casing in wet and caving ground. Mineral or polymer slurry can be used to maintain open holes. Slurry supported holes are wet holes per the CDOT Standard Specifications.

Concrete placement method depends on the condition of the drill hole. Dry holes are usually concreted using tremies, or free-fall of concrete directed into the center of the hole so as not to strike the reinforcing bars. Concrete is usually placed in wet holes using the tremmie method.

2.5.2 Examples of Construction Methods from the T-REX and Broadway Projects.

Four Osterberg (O-Cell) load tests on drilled shafts were performed in the T-REX and Broadway that will be described in the next chapter. The load test sites along I-25 are called: County Line, I-225, Franklin, and Broadway. The construction of the test shafts in these two projects is representative of the typical construction procedure for production shafts employed in the Denver area and in the T-REX and Broadway construction projects. The construction of these shafts is described by Abu-hejleh et. al. (2003) and is summarized below.

Excavation Methods: 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 (soft) 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 (groundwater level) 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. In addition, 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. During drilling, the shaft sides were dry all the way to the bottom of the Broadway shaft. 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. After concrete placement is completed, the temporary casing was pulled out and additional concrete was added to maintain the targeted elevation for top of concrete.

2.5.3 Recommendations for Construction of Future Drilled Shafts

CDOT Research report 2003-6 provides recommendations to improve CDOT Standard Construction Specifications for construction of drilled shafts. These recommendations were

reviewed very carefully after the report was published and a new set of recommendations were developed. They are presented next.

Shaft Cleaning: Good construction practices for production shafts meeting the requirements of CDOT Standard Specifications for Drilled Caissons are expected as discussed before. The inspection process should, and generally does, result in a hole with minimal slough remaining. It is very important to have qualified, experienced inspectors with sufficient authority as part of this process. The inspectors must know they have the full support and backing of the Project and Resident Engineers. If the contractors know up front that they will be held to proper standards they can and generally will do a good job. Shaft drilling tends to be a repetitive process. Therefore, if proper procedures are established in the first few holes, they will generally be followed throughout the project.

For deep shafts cleaned following the standard procedure, but where a clean bottom cannot be verified (as in very deep shafts, in jointed and blocky rock or in cases there is some sloughing and spalling is expected), one of the following measures should be undertaken:

- Reduce the recommended ultimate base resistance by 20%; or
- Deepen the bedrock socket length by 20%. This is not restrictive for deep shafts because most of the carrying capacity of deep shafts will be in side shear. It is not uncommon for about 80 percent of the capacity of deep shafts to be in side shear. Thus, reducing the end bearing by 20 percent only reduces the overall capacity by about 4 percent. For most shafts, this load carrying deficiency can be overcome by the side shear of a few feet of additional penetration; or
- Post-grout the bases of the drilled shaft through pipes inserted in the reinforcing cage. 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} (maximum base resistance) 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.

Drilling and Concrete Placement: Rapid drilling and placement of concrete of the shaft holes is 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 inch 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.

Use of Water, Slurry and Casing: 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 used for load resistance, 2) will not pour excessive 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. In dry holes, a small amount of added water (a few buckets at most) may be desirable and allowed to moisten the cuttings of soft soil-like bedrock. Making these cuttings “sticky” does aid in cleaning the hole. Adding water is not necessary and is not appropriate with very hard and blocky bedrock that is not made “sticky” by the addition of a little water.

Casing and slurry should not be used in rock sockets. The rock penetration should be measured below the bottom of the casing. Slurry is generally defined as a mixture of water and clayey overburden soils and is used to advance a hole until casing can be set. After the casing is set the slurry is then pumped out to create a dry hole when the shaft is extended into bedrock. Thus, once the hole extends into bedrock there is no remaining “slurry.” There are some occasions with holes that cave in bedrock, sometimes from poorly cemented or caving sandstone layers. With

respect to using slurry in the rock socket for caving sandstone, etc., this should be avoided if possible. If this is not possible, 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. If the caving is accompanied or caused by groundwater inflow, the caving can usually be controlled by filling the shaft with water above the hydrostatic water level in the caving zone. This should not be called “slurry”, just water.

Wet Shaft Holes: In some other cases, caving will not occur but the caisson hole is also called “*Wet*” because water is infiltrating into the bedrock hole (due to presence to fractures and joints in blocky rock) at a rate higher than it can be pumped out. Exposure of certain shale to water for long periods could weaken its side resistance. In this case, time of drilling is an important and the contractor should be prepared to drill the hole to the required depth plus a few feet in a continuous drilling operation. The hole should then be quickly cleaned and the reinforcing cage should be installed quickly. The concrete trucks should be positioned to start concrete placement immediately and continuously. The depth of extra drilling should be determined in consultation with the Geotechnical Engineer based on the estimated degradation of the sidewall materials on the exposure to the water.

Very hard shale bedrocks should meet the requirements of Colorado Testing Procedure 26-90 for rock-like material (durable, sound, not sensitive to water, and has very small potential for creep). Water is not expected to degrade this type of sound and rock-like material. Based on field observations and results of load tests (Broadway and Trinidad), large water infiltration in these types of rocks is due to presence to fractures and joints in blocky rocks and will not cause caving and degradation to the rock, so that deepening the hole may not be needed. At any rate, it is probably better to have that be a field decision depending on conditions as opposed to a specification requirement. The decision to deepen the hole with wet shafts should be left to CDOT Project Engineer in the field after consultation with the Project Geotechnical Engineer.

If the shaft hole is wet, the cement ratio of the concrete mix is often increased by 25% and the tremie method must be used to place concrete. The general contractor should have the proper

equipment for the tremie method on site when the shaft is partially or completely full of water during the concreting (wet). If this is not performed, problems could occur during concreting, resulting in poor quality concrete. The general contractor and the drilled shaft contractor must be familiar with standard concreting procedures for wet shafts and should plan to work together on this issue. The general contractor should submit a detailed plan describing his intended procedure to deal with wet shafts.

When a shaft is constructed wet, a solid steel pipe (tremie) should be used, not the flexible drain pipes used in concreting dry holes. The concrete can be pumped through a 5 or 6-inch pipe or delivered by gravity through a 12-inch pipe. FHWA has very clear guidance on this point. Most procedures for placement of concrete in dry shafts, such as using "elephant trunks" or free falling, are not acceptable. In addition, concreting of wet shafts through a tremie must require the use of a seal or "rabbit" type device so that concrete and water do not mix in the tremie. Often a pipe is not sealed but a sponge or "rabbit" is placed at the top of water level so that as the concrete moves down the water filled tremie, it does not mix with the water. This issue might be worth further investigation and a look at other states' specifications for suggestions on their procedures.

Shaft Roughening: Minimal artificial roughening for all CDOT drilled shafts socketed in weak rocks is recommended if roughening under normal drilling is not observed. Medium to rough holes were obtained in the very hard claystone and sandstone shales at the Franklin and Broadway sites with normal drilling procedures. The procedure suggested below for minimal roughening is much less rigorous than the extensive and more expensive process of roughening with shear rings.

Minimal roughening can be achieved by asking the drillers to make a final drilling pass by replacing the outer cutting teeth with a "roughening" tooth that extends about 1.7" (or any other dimension used by the drillers) from the sides of the auger to roughen the socket at least minimally. The tooth dimensions depend on the driller. The 1.7" tooth used on the TREX test shafts was what the contractor had on hand. Others use roughening teeth up to 3" long.

Regardless, the tooth should be able to score the side of the hole. The best procedure is to spin the auger with the tooth attached and simultaneously move the Kelly up and down to create diagonal scoring on the sides of the hole. The roughening does not need to completely score the side of the hole, but should make numerous, visible grooves. It takes only few minutes to be performed on 10 ft bedrock socket. As the artificial roughening described above is simple, quick, and easy to do, it is recommended to consider this roughening “if in doubt of roughening under normal drilling” No inspection of the dimensions of grooves is needed herein (no measurements of depth and width of grooves), just witnessing that the contractor performs the work described herein. The proposed roughening method shall be approved by the Project Engineer. If the roughening operation described below results in excess degradation of the bedrock (e.g., caving), or otherwise adversely affects the final product, alternate procedures shall be proposed to the Engineer. The roughening requirement may be waived at the Engineer’s discretion and after consultation with the Geotechnical Engineer.

The minimal roughening is most appropriate for soft, soil-like claystone, as its main purpose is to knock the “shine” off the side of the hole in order to allow the natural roughness to be effective. In harder shale bedrock there are likely to be some asperities. On such formations, the need for roughening can usually be determined by observations during normal drilling (no roughening is needed if it is observed during the normal drilling).

For holes drilled without casing, the roughening procedures presented above should be the standard operating procedure if roughening under normal drilling is not observed. A grooving tooth cannot be used in a cased hole. In cased holes, and if roughening sides cannot be observed during normal drilling, the contractor should decide the best tool (perhaps a simple version of shear rings) to minimally and quickly (in matter of few minutes) roughen the hole side as with a grooving tooth.

Artificial roughening with shear rings during drilling was employed at some load test sites as will be discussed in the next chapter.

3. RECORDS OF COLORADO'S AXIAL LOAD TESTS ON DRILLED SHAFTS SOCKETED IN WEAK ROCKS

3.1 Overview

This Chapter and Chapter 5 document and evaluate the available records of Colorado's axial load tests performed on drilled shafts socketed in weak rocks in the last 35 years. Because of the significant expense of drilled shaft load tests, there have been relatively few load tests performed in Colorado. A summary of the identified load tests is presented in Table 3.1. For each of the load tests, the study attempted to obtain the following information (see Table 3.1 and Appendices A, D, and E):

- Geographic location.
- Number, date, and type of load test.
- Geological information of the bedrock.
- Construction information of the shaft hole. This includes information on the method and timing (e.g., how many hours or slow vs. fast) of the drilling operations and information on the conditions of the shaft wall sides and bottom (wet or dry; rough or smooth sides; cleaned or not).
- Construction and material information of the test shafts. This includes method and timing (e.g., how many hours or slow vs. fast) for placement of the concrete and information on the strength and slump of the concrete.
- Layout information of the shafts. This includes diameter (D) and length (L) of the rock socket, and other information that describe the location of the test shaft with respect to the bedrock and groundwater table (GWT).
- Geotechnical description of the shale bedrocks as obtained from results of subsurface geotechnical investigation, such as the N-value (# of blows per foot or bpf) obtained from the standard penetration test (SPT), and/or the unconfined compressive strength (q_u) obtained from the unconfined compression test (UCT), and any reported strength and stiffness results obtained from the pressuremeter test (PMT).

- Results of the load tests. These include information on the side resistance (f) vs. movement (w) curve up to the ultimate side resistance (f_{max}), base resistance (q) vs. settlement (w) up to the ultimate base resistance (q_{max}), and the load-settlement curve.

Table 3.1. Summary of Colorado’s Available Load Tests on Drilled Shafts

Name of Load Test	Date	# and Type of Load Test	D (ft)	L (ft)	Geological Formation	Available Information	Comments	
Fort Carson	1970	4, Conventional 2-End Bearing 2-Side	1’	1.5 5.0	Pierre Shale Claystone	Some construction details, UUT	D is small, low capacity piers.	
23 rd Street Viaduct, Denver	1992	3, conventional 1. End-Bearing 2. Side (smooth) 3. Side with shear rings.	2.6’	3.6 9.4 9.5	Denver Blue Claystone Formation	Some construction details, SPT, RQD, UCT, and UUT,	There is significant range in the strength data.	
I-270/I-76, Denver	1992	2, Conventional 1. End Bearing 2. Side Resistance & End bearing, upper 5 ft roughened with shear rings.	2.5	1’ 9’	Denver Claystone Formation	Detailed construction information and geotechnical tests		
SH 82 Shale Bluffs Load Tests	1998	2 O-Cell Load Tests, combination of side shear and base resistance.	# 1	2.5	15’	Weathered Mancos Claystone	Only low RQD	Not recommended.
			# 2	3.0	29.7	competent Mancos Claystone	Only very high RQD	Did not fail in base and side resistance
I-25 @ TREX Project & Broadway project	2002	4, O-Cell Load Tests, combination of side shear and base resistance a) I-225 b) County Line c) Franklin d) Broadway	3.5 4 3.5 4.5	19 16 21 26	Weak Denver Fm Weak Dawson Fm Denver Blue Denver Blue		Comprehensive Investigation and analysis	
I-25 @ Trinidad	2003	2, O-Cell load Tests	4		Pierre Shale			

The compiled load tests (Table 3.1) are named after their location as: Fort Carson, 23rd Street Viaduct in Denver, I-270/I-76, SH82 Shale Bluffs in Pitkin County, T-REX along I-25 in Denver (I-225, County Line, and Franklin), Broadway Viaduct along I-25 in Denver, and Trinidad. For all these tests except the Fort Carson and SH 82 Shale Bluffs load tests, Tables A.1 to A.7 (Appendix A) present the available testing results from the load tests and from the simpler and routine geotechnical tests (e.g., SPT, UCT, and PMT), and information on the materials, construction, and layout of the tests shafts.

3.2 Trinidad Load Tests

As a part of the I-25 Trinidad project located in Trinidad, Colorado, two O-Cell load tests were performed on 48” diameter test shafts. The purpose of these two load tests was to check the recommended geotechnical design parameters for the drilled shafts in the construction project and to provide research data that would improve the accuracy of CDOT future design methodology for drilled shafts. Dr. Naser Abu-Hejleh from CDOT Research Office administrated a comprehensive subsurface exploration and laboratory testing program around the two test shafts, and then provided construction and testing information for the two load tests. The Trinidad load tests are discussed thoroughly in Chapter 5. Summary information on the layout, materials, and construction of the test shafts and the results from the load tests and the subsurface geotechnical investigation program are presented in Table A.5. Reported results from the load tests and the subsurface geotechnical investigation are listed in Appendix E.

3.3 T-REX and Broadway Load Tests

These tests are thoroughly documented in a recently published CDOT Research Report 2003-6 (Abu-Hejleh et. al., 2003) titled *“Improvements of the Geotechnical Axial Design Methodology for Colorado’s Drilled Shafts Socketed in Weak Rocks.”* Tables A.1 to A.4 provides a summary of all the information on the layout, materials, and construction of the test shafts and the testing results from the load tests and the subsurface geotechnical investigations. The results from this study that are relevant to this study are presented in this section.

As a part of the construction requirements for the T-REX and Broadway Viaduct project along I-25 in Denver, four Osterberg axial load tests were performed in 2002 on drilled shafts embedded in the typical range of weak rocks encountered in Denver: soil-like (or soft to firm) claystone (I-225 and County Line sites) to very hard sandy claystone (Franklin site) to even much harder and more massive clayey sandstone (Broadway site). To maximize the benefits of this work, the Osterberg 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 at the load test sites. This included SPT, UCT, and PMT. Analysis of all test data, information, and experience gained in this study were employed to provide: 1) best-fit equations to predict the unconfined strength of weak rocks from SPT, and PMT data; 2) assessment of the CDOT design method (see Section 3.3.1) and AASHTO/FHWA design methods; and 3) recommended design equations to predict the shaft ultimate unit base resistance (q_{max}), side resistance (f_{max}), and an approximate load-settlement curve as a function of the results of simple geotechnical tests for the types of weak rock and conditions investigated in this study (see Section 3.3.2). Other products are developed to help CDOT with implementation of accurate and feasible LRFD methods for the design of drilled shafts.

A relationship between the SPT-N value and the unconfined compressive strength (q_u), is recommended as q_u (ksf) = 0.24 N.

Two failure criteria to define ultimate f_{max} and q_{max} for claystone bedrocks were employed in the analysis:

- For soil-like or soft claystone (SPT N-value < 100 bpf, or q_u < 25 ksf), f_{max} and q_{max} should correspond to the true base and side resistance values that correspond to the full mobilization of the resistance in the plastic range. If needed, 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 a displacement of 5% of the shaft diameter, but not to exceed 3 inches, and f_{max} to a displacement of 1% of the shaft diameter (0.3”).

3.3.1 Assessment of Colorado SPT Based Design 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) with no information on expected settlement of drilled shafts. In the Colorado SPT-Based design (CSB) method, the allowable base resistance in kips per square foot (ksf) is taken as $q_{all} \text{ (ksf)} = q_{max}/FS = 0.5 N$, and the allowable side resistance is taken as $f_{all} \text{ (ksf)} = f_{max}/FS = N/20$. With the lack of information on the proper factor of safety embedded in the CSB design method, a factor of safety (FS) of 3 (resistance factor, ϕ , of 0.5) is often assumed by CDOT engineers and is used to recommend q_{max} and f_{max} values. The same CSB design method, based SPT-N values, is uniformly applied to both cohesive and cohesionless weak rocks and to stronger rocks. 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). Assessment of the CSB method concluded (see CDOT Research Report 2003-6 for more details) that

- ❑ There is a large difference between the measured and predicted ultimate resistance values because the true FS associated with the CSB method is smaller than the assumed value of 3.
- ❑ 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 ($q_{all} = 0.46N$) using a factor of safety of 2.
- ❑ The CSB side resistance design method for the soil-like claystone resulted in a factor of safety of less than 2 but above 1, ranging from 1.3 to 1.8.
- ❑ 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.

3.2.2 Recommended Design Methods.

These recommendations are valid for drilled shafts with conditions (type of weak rocks, adequate subsurface geotechnical investigation, shafts: materials, construction, and layout) close to those of the four load test sites described in the study.

I. Soft claystone bedrock shales (called also soil-like or weathered). Clay-based geomaterials with SPT-N values (bpf) between 20 and 100 ($q_u < 24$ ksf) as those encountered at the I-225 and County Line sites. Updated Colorado SPT-Based (UCSB) Design Method is recommended for soil-like claystone as:

$$q_{\max}(\text{ksf}) = 0.92 N \dots\dots\dots 3.1$$
$$f_{\max}(\text{ksf}) = 0.075 N \dots\dots\dots 3.2$$

And with a recommended factor of safety of 2, the allowable unit base and side resistances are obtained as:

$$q_{\text{all}}(\text{ksf}) = 0.46N \dots\dots\dots 3.3$$
$$f_{\text{all}}(\text{ksf}) = 0.037N \dots\dots\dots 3.4$$

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 soft claystone geomaterial.

II. Very hard sandy claystone bedrock shale with SPT-N value >120 bpf (or > 50/5”) and unconfined compressive strength (q_u) is less than 100 ksf. This is a very typical material in

Colorado. The Franklin bedrock is a very hard, mostly thinly bedded, bluish gray, and sandy claystone bedrock with q_u ranging from 40 ksf to 90 ksf (average of 65 ksf) around the bedrock socket and around 41 ksf beneath the socket. 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. The following design equation is recommended for conditions similar to those of the Franklin test shaft. Use the Canadian design equation to predict the base resistance with a factor of safety of 3

$$q_{max} = (1.2+0.48 L/D) q_u \text{ and not to exceed } 4.08 q_u \text{ when } L/D > 6 \dots\dots\dots 3.5$$

and

$$f_{max} \text{ (ksf)} = 2.05 q_u^{0.5} \dots\dots\dots 3.6$$

with FS of 2.7.

III. Very hard and Massive Bedrock Shale with q_u less than 500 ksf, and SPT-N values >100 for granular-based rock, and $q_u > 100$ ksf for clay-based rock.

The Broadway bedrock is very hard, well-cemented, bluish gray and clayey sandstone with claystone interbeds and q_u ranging from 97 ksf to 293 ksf (average of 145 ksf) around the bedrock socket and around 219 ksf beneath the socket. 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.

$$q_{max} \text{ (ksf)} = 17 (q_u)^{0.5} \dots\dots\dots 3.7$$

The recommended FS is 2.7. Use Eq. 3.6 and FS of 2.7 for the side resistance analysis.

3.4 Fort Carson Load Tests

Load tests on four drilled shafts near Fort Carson, Colorado, were conducted in 1970 for the U.S Army Corps of Engineers, as described by Jubenville and Hepworth (1981) and later analyzed by Turner et al. (1993). The tests were performed on 1’ diameter shafts founded in Pierre shale.

Two of the shafts were side shear tests with 5 linear feet of formational material tested. Cardboard (possibly void form) was used to isolate the tip. For the end bearing tests, the sides of the shafts were isolated. Both side shear tests were successful. One of the end bearing tests was successful. Unconsolidated, undrained triaxial shear tests (UUT) were performed on seven core and drive samples, not the unconfined compression test (UCT) recommended by Abu-Hejleh et al (2003). For illustration and analysis purposes, it will be assumed that the results from the UUT and UCT are similar. The shear strength of the samples from the UUT varied from 8.3 ksf (N= 35 bpf) to 17.3 ksf (72 bpf) ksf with an average of 11.3 ksf (47 bpf). The N-values were estimated from the equation recommended by Abu-Hejleh et al. (2003) as q_u (ksf) = 0.24 N.

The ultimate end bearing was measured at settlement that corresponds to 10% of the shaft diameter as 75 ksf. The measured ultimate side resistance at a settlement of around 0.27" averaged about 4 ksf. These data suggest that the encountered bedrock shale in this site fit to the description of soft claystone bedrock shale. The prediction of q_{max} from Eq. 3.1 is $= 0.92 * 72 = 66.3$ ksf and for f_{max} is $= 0.075 * 47 = 3.53$ ksf. These predictions are close to the measured resistance values and are conservative.

3.5 The 23rd Street Viaduct Load Tests

At the 23rd Street Viaduct, three conventional load tests were performed on 31" diameter shafts (one end bearing with L= 3.6' and two to evaluate side resistance with and without shear rings, L=9.5'). The purpose of the load tests was to confirm the recommended design values presented in the Geotechnical Investigation for the 23rd Street Viaduct Replacement project. The source for information on these tests is a report prepared by Ground Engineering Consultants (1992). The title of this report is "Drilled Straight-Shaft Piers, Pier Load Test, 23rd Street Viaduct, Denver, Colorado." The test shaft locations presently near the NE corner of Wazee Street and Park Avenue (used to be called 23rd street), inside the Coors Field parking lot. The test site is located where Denver Formation bedrock is at a shallow sub crop. Figures D.1 to D.7 in Appendix D summarize the locations (Figure D.1), elevations, and as-built details of the test shafts (Figure D.2), and geotechnical test data (Figure D.3 and D.4) and load test data (Figures D.5 to D.7). Table A.6 provides summary information on the layout, materials, and construction of the test shafts and the testing results from the load tests and the subsurface geotechnical investigation.

3.5.1 Subsurface Conditions and Strength Characteristics of the Bedrock

Standard Denver area geotechnical data are also presented, including blow counts on a California Sampler, and unconfined compression test and triaxial shear test data (Figures D.3 and D.4). The load test site consists of fill and natural clay overlying claystone (Denver Blue Formation) with occasional interbeds of sandstone and siltstone. RQD from two boreholes in the test pockets was in the range of 93 to 95. The design methods (Eqs 3.1 to 3.7) are either based on SPT N values, not the N-values from California Sampler, or unconfined compressive strength of the rock, not the compressive strength obtained from the UU triaxial tests. N-values obtained with California Sampler range from 76 (in the upper range) to 120 (in the lower more competent zone) with average of 94 (Turner et. al, 1993). The unconfined compressive strength of the bedrock, q_u , ranged from 5 ksf (in the upper range) to 25.2 ksf (in the lower competent zone). In the test pocket for end bearing test (lower zone, see Figure D.3), appropriate N- value and unconfined strength should be equal, around, or even greater than, respectively, 120 bpf or 25.2 ksf. To solve uncertainties with the strength data obtained around and below the test shafts, it is recommended to:

1. Obtain more reliable test data from SPT or unconfined compressive strength tests.
2. Rank the test data as fair (not good) because they are based on judgment, not only testing.

3.5.2 Test Program and Construction

The two side resistances load tests were socketed 9.4 ft and 9.5 ft in the bedrock socket with 1 foot of collapsible void placed at the base to isolate the bearing resistance (Figure D.2). One shaft was drilled with straight sides. The other shaft had five 2" by 3" shear rings at 1.5' spacing. The rock socket length for the end-bearing test was 3.6 ft. Void was maintained between the test shaft and the geomaterial around the test shaft to isolate the side resistance. Loads were applied in accordance with ASTM 1143-81, Quick Load Test Methods for Individual Piles. The shafts were instrumented to measure the settlements of the shaft during the test. The load vs. settlement curves for the three tests are shown in Figures D.5 to D.7.

3.5.3 Test and Analysis Results

The end-bearing test resulted in an end-bearing pressure of about 118 ksf at 1-inch settlement. Ground Engineering felt this was less than ultimate as the load deflection curve suggested the bottom of the hole was not clean (Figure D.5). Nevertheless, with a FS of 2 to 3, an allowable design load of 40 ksf to 50 ksf is indicated, which is in line with typical Denver area design values (CSB Design Method). Given Ground's concern that the hole was not fully clean, and that the tip of the shaft was quite shallow, the results may be conservative.

According to the definitions of q_{\max} and f_{\max} established by Abu-Hejleh et. al. (2003),

- If the claystone bedrock is soft, ultimate true q_{\max} in the 1st load test is 198 ksf (settlement of 3"), f_{\max} in the 2nd load test is 5.9 ksf (2"), and f_{\max} in the 3rd load test is 21.6 ksf (0.85").
- If the claystone is very hard, q_{\max} in the first load test is 145 ksf measured at settlement equals to 0.05 D or of 1.5", f_{\max} in the 2nd test is 3.3 ksf (at settlement that corresponds to 0.01 D= 0.031"), and in the 3rd load f_{\max} is 18.2 ksf (see Appendix D).

Based on the measured SPT N values, it seems that the rock around the test shafts fits to soil-like (or soft) claystone, so f_{\max} values of 5.9 ksf can used for the load test with smooth sides and 21.6 ksf for the load tests with sides roughened with artificial shear rings. Using Eq. 3.2, f_{\max} can be predicted as $94 * 0.075 = 7$ ksf. This is close to the measured resistance value from the load test with smooth sides (5.9 ksf). Ground Engineering suggested seepage of perched groundwater might have influenced the concrete to bedrock bond. Groundwater seepage was observed from bedrock fractures during drilling. In the test shafts analyzed by Abu-Hejleh on the soft claystone (at County Line and I-225), the lower 8 ft of the side shafts were cleaned with minimal roughening, and Eq. 3.4 should be used with N-values obtained from the SPT not obtained from the California Sampler. These two factors might contribute to the small difference between the measured f_{\max} and those predicted with the equations suggested by Abu-Hejleh et. al. (2003). Also, the penetration driving resistance to be

The capacity of the 23rd shaft with shear rings (3rd load tests) was substantially greater than the smooth-sided shaft 21.6 ksf compared to 5.9 ksf). With a FS of 2.0 to 2.5, an allowable side

shear of about 11 ksf is indicated. A rough-sided rock socket could have three times the side resistance capacity of a smooth-sided rock socket (Abu-Hejleh et. al., 2003). The performed two load tests at the 23rd Street suggest a ratio close to 3.5.

The shale bedrock beneath the tip of the end-bearing test shaft is at the boundaries between the soft claystone and the very hard claystone (N-value of 120 bpf from the California Sampler or unconfined compressive strength of 25.2 ksf). Whatever design method is employed to predict q_{max} , the measured q_{max} value will suggest that the rock beneath the test shaft is stronger than 25.2 ksf, as suggested before.

Based on the load test data and the overall geotechnical data for the overall project, Ground presented a design end-bearing of 75 ksf, and side shear of 3 ksf in the first 10' of bedrock with 8.5 ksf below 10 feet when shear rings are installed.

3.6 The I-270/I-76 Load Tests

Two conventional load tests were performed on 2.5' diameter shafts (one end bearing with $L=1'$ and the 2nd to evaluate side resistance, $L=9'$). Woodward-Clyde Consultants (WCC) (now part of URS) performed axial and lateral load tests on driven H-Piles and drilled shafts. The study was performed under contract to CDOT under Project Number IM-IR(CX) 25-3(107). The purpose of the study was to confirm design recommendations for then proposed improvements to the I-70/I-270 interchange and the nearby I-25/I-270/SH-36 (Boulder Turnpike) Interchange. The source for information on these tests documented in this study is a report prepared by Woodward-Clyde Consultants (1992). The title of this report is "Pier and Pile Foundation Load Tests at Interchanges 270 and 76, Adams County, Colorado."

Appendix D provides the following information for this test: location (Figure D.8), a summary log of the test borings adjacent to the location of the test shafts (Figure D.9), load test results (Figures D.10 and D.11), and layout and properties of the test shafts (Figures D.12 and D.13). The test site is located between SH 224 (south side) and Clear Creek, just north of the then existing I-270/I-76 interchange and on line with the anticipated I-270 extension (Figure D.8).

The test site is located in a gravel parking lot adjacent to Clear Creek, which provides access to a pedestrian and bike path along the Creek. The bedrock strength and consistency were evaluated by field and laboratory testing. Field-testing included Standard Penetration Tests (SPT), NX coring with measurements of recovery and RQD, and pressuremeter tests. Laboratory UU Triaxial Tests were also performed. Table A.7 provides summary information on the layout, materials, and construction of the test shafts and the testing results from the load tests and the subsurface geotechnical investigation.

3.6.1 Subsurface conditions and Strength Characteristics of the Claystone

Subsurface conditions at the test site consisted of about 4 feet of poor quality fill and 15 feet of sandy alluvium over sandstone and claystone bedrock (Figure D.9). The claystone bedrock was encountered at a depth of about 19 ft. The upper surface of the claystone was weathered with the degree of weathering reducing with depth. This layer is called the weathered claystone layer and extends from depth of 19 ft to 25 ft. The unweathered or more competent claystone layer extends below a depth of 25 ft.

In the weathered claystone layer, the SPT N-values range from 32 to 76 (bpf) with an average of about 50 blows per foot (bpf). No undrained shear strength values from the triaxial tests were reported in this layer. The unconfined compressive strength for this layer was estimated indirectly from the results of PMT and SPT (Abu-Hejleh et. al., 2003) as, respectively, 9.4 ksf and 12 ksf. For analysis purposes, it is seems reasonable to assume $q_u = 10$ ksf and SPT- N value of 50 for the weathered claystone layer.

In the unweathered (or competent) claystone bedrock layer, the SPT N-values ranges from 62 to over 200 (bpf) with an average of 100 bpf. After reviewing the boring logs, it seems that an average SPT-N value of 100 bpf is reasonable and could be employed in the analysis. Results of triaxial tests suggest that the unconfined compressive strength in this layer range from 12.8 ksf to 23.6 ksf. Two pressuremeter tests were performed in this layer,

- One at depth of 26 ft (very close to the boundary between the weathered and unweathered claystone layers), resulting in q_u of 23.8 ksf.

- One at depth of 31 ft, resulting in q_u of 22.2 ksf.

The strength data from the PMT are more consistent and reliable than results from laboratory strength tests as reported by Abu-Hejleh et. al. (2003). Note that the SPT-N value of 100 corresponds to unconfined compressive strength is 24 ksf. For analysis purposes, it is seems reasonable to assume $q_u = 23$ ksf and SPT N-value of 100 bpf for the competent (unweathered) claystone layer.

Both the weathered and unweathered claystone layers with SPT N-value fall under the definition of soil-like claystone as proposed by Abu-Hejleh et. al. (2003). Thus, the ultimate resistance values, q_{max} and f_{max} for both claystone layers should correspond to the full mobilization of the resistance in the plastic range.

3.6.2 Construction of Test Shafts

Two 2.5 foot diameter drilled shafts were constructed and then tested axially to failure. One test was in end-bearing (Test 1), the other was a combination of side shear and end-bearing (Test 2). Excavation for both shafts was performed using 30" helical auger. After drilling through the overburden to the upper surface of the claystone using slurry mixed in the hole, a temporary casing was installed and sealed into the claystone. Then, the slurry was removed and the remaining penetration of the drilled shafts in the claystone was drilled essentially dry. A trace of water entered most of the shafts during drilling, with a maximum rate of 2 to 5 gpm in some shafts. Water depth was 2" or less in all shafts before placement of concrete. The reinforcing steel was inserted before concrete placement for most of the production shafts, except for those shafts with higher inflows of water. In that case, the reinforcing steel was inserted thought he fresh concrete. The concrete was placed to about 2 feet below grade by directing a freefall down the center of the shaft, without striking the steel.

The end-bearing test shaft had minimal penetration into claystone bedrock (one foot below the bottom of the casing). The combination test shaft was socketed 9-feet into bedrock below the bottom of the temporary casing with 5 ft in the weathered claystone and 4 ft in the unweathered

claystone. One shear ring (3" high, 2" deep) was installed near the upper surface of the unweathered claystone and the remaining shear rings were installed in the weathered claystone.

3.6.3 Testing and Analysis Results:

The axial loads were applied through a reaction beam restrained by drilled shafts acting in uplift. Axial loads were measured by calibrated hydraulic jacks and an electronic load cell. The load distribution along the shaft was evaluated by tell tales and strain gages in the shaft at the top of bedrock and near the bottom of the shaft. The test procedure generally followed the "quick" method set under ASTM 1143 Standard Test Method for Piles under Static Axial Compressive Load.

The end-bearing test essentially provided a direct measure of end bearing in the upper, weathered claystone. The end-bearing that correspond to the full mobilization of the resistance in the plastic range is measured as 47 ksf. The ultimate end-bearing pressure was consistent with the Denver Method prediction based on the SPT. It perfectly matches the equations suggested by Abu-Hejleh et. al. (2003), Eq. 3.1, as $q_{max} = 0.92 N$, with prediction of 46 ksf for a claystone with SPT-N value of 50 bpf.

With respect to the combined test, the strain gages and telltales were used to separate the total applied test load into end-bearing and side shear. Interpretation was required because the lower strain-gage was above the bottom of the shaft. Based on the strength ratio between the unweathered claystone (23 ksf), and the weathered claystone (10 ksf), and the measured q_{max} for the weathered claystone (47 ksf), a q_{max} of 105 ksf is estimated for the unweathered claystone. Using this value the average f_{max} in the rock socket (5 ft weathered and roughened claystone and 4 ft of unweathered claystone) is 12.4 ksf. For the entire claystone layer, the average weighted SPT-N value is 72 bpf and q_u is 16 ksf. Using Eq. 3.1, f_{max} can be predicted as $0.075 * 72 = 5.4$ ksf. The shear rings appear to have increased the side shear resistance by a ratio of 2.5 (12.4/5.4). This ratio is lower than the ratio of 3.5 measured at the 23rd Street load test and the ratio of 3 reported in the literature, most likely because the shear rings were not applied in the entire

claystone layer. It seems to be reasonable to conclude that the shear rings increased the f_{\max} in the unweathered claystone by a ratio of 3 as reported in the literature.

3.7 SH 82 O-Cell Load Tests (Pitkin County)

Two O-Cell load tests were performed along SH 82 in 1998. Each test provided information on side resistance and base resistance but no geotechnical test data (e.g., q_u) were provided other than RQD. The results of these two load tests are summarized in two test reports prepared by LOADTEST, Inc. (1998) to the Colorado Department of Transportation, titled as

- Test # 1: Caisson 47 A, Shale Bluffs, Pitkin County
- Test # 2: Test Shaft at Pier No. 2, Highway 82 Glenwood/Aspen-Aspen, CO.

According to Mr. Shan-Tei Yeh, the two load tests were intended to evaluate the resistance of softer portion of the bedrock (highly weathered Mancos claystone formation, Load Test No. 1) and the harder portion (very competent Mancos claystone formation, Test No. 2) so the data could be used for bridge design.

3.7.1 Test 1 (Caisson 47A, Shale Bluffs)

This was an O-Cell test on Highway 82 in Pitkin County. The shaft was drilled into a steeply sloping bedrock surface, with about 10' difference in rock elevation across the shaft. Drilling by dry hole technique began on the morning of 5/19/98. Due to problems with caving of the overburden, drilling was continued the following morning after a temporary casing was installed. A 5-foot of grout was pumped below the level of the O-Cell before the cage was inserted. The remainder of the shaft was filled with concrete by gravity feed (dropped) from top of the hole. The test shaft was 2.5 feet in diameter, but had a relatively small rock socket of about 10 feet above the O-Cell. There is little geotechnical information available. The boring was cored, but only recovery and RQD data (83% to 6%) were presented on the log (Figure D.12).

The load test failed in end bearing and side shear at the same load increment indicating the O-Cell was at the balance point for the shaft. Because of limited penetration in poor quality claystone, the ultimate side shear in the bedrock was between 3 and 4 ksf (movement around 4

inches). There was an approximate 5' concrete (or grout) plug below the O-Cell, so considerable assumed side shear had to be subtracted to obtain the end-bearing capacity. The resulting ultimate end-bearing capacity thus calculated was 40 ksf.

This test shaft is of little value for research purposes and additional geotechnical investigation at this test shaft location is not suggested because:

- The radically sloping bedrock surface around the test shaft making it hard to analyze the data.
- The presence of 5 ft of grout below the O-Cell make it hard to analyze the data and separate the base resistance from the side resistance in that zone.
- Problems encountered during construction.
- There is little geotechnical information available, other than RQD that appears to decrease with depth in rock over the length of the shaft from an RQD of 83 near the top of the shaft, to less than 10 near the tip. The bedrock was described as weathered, much fractured with joints (sometimes vertical) and/or with clay zone (see Figure D.12).
- Region 2 was contacted and according to them it is very difficult to access this site.

The reported side and base resistance values suggest that low resistance values should be expected with highly weathered and fractured claystone shale. By comparing the reported load testing results for this test with the four load tests analyzed by Abu-Hejleh et. al. (2003), it seems that the bedrock at this site is a little bit weaker than the soil-like claystone encountered at the I-225 site. For such bedrock, the RQD is not needed and the rock should be treated as very stiff clay or soft claystone.

3.7.2 Test 2 (at Pier No. 2)

The second load test on the Shale Bluffs project was on a 3 foot diameter shaft that extended 29.7 feet into hard shale. The shaft was constructed dry with a total length of 39.7 ft. The tip of the test shaft was located 1.3 ft below the O-Cell. Log information and Load test data are presented in Figures D.13 to D.16.

The sub-surface stratigraphy at the test shaft location (Figure D.13) consists of sandy gravel overburden down to a depth of 8 ft where weathered shale was encountered in the next two feet. At depth of 10 ft to undetermined depth, very hard shale was present (Figure D.13). As with load Test No. 1, the only geotechnical data on the logs is recovery and RQD data (Figure D.16). Compared to Test 1, the shale in this test appears to be much sounder, with RQD values of 70% to 90% (Figure D.13).

As seen in Figure D.16, the shaft did not fail at the maximum O-Cell load of 1,239 tons applied up (movement of 0.07") and down (0.09"). Movement to that deflection was linear, so the testing company did not attempt to extrapolate the data to failure values. The measured maximum unit side resistances at the end of the test were 4 ksf in the upper 13.4 ft of the bedrock socket, and 14.8 ksf in the lower 15 feet of bedrock socket, with an average value of 9.2 ksf in the entire bedrock socket. The trend of unit side resistance vs. side movement data over 0.07" of movement seems to be close to similar results reported by Abu-Hejleh et. al. (2003) for the Franklin and Broadway load tests and similar to the results obtained for the Trinidad load tests that will be presented later in this report.

The maximum measured unit base resistance was 325 ksf (at settlement of 0.09"). This is a very high value at a settlement of 0.09" when compared to all other load tests presented in this study, suggesting the rock strength in this site is much stronger/stiffer than the very massive bedrock shales encountered in the Broadway (q_u as high as 200 ksf) and Trinidad (q_u as high as 500 ksf) sites. It is possible that the rock strength in this site is close or even exceeds the strength of the concrete. It would be worthwhile to drill an additional boring at this location so that strength tests can be performed on recovered cores. *Until then, we should be careful with accepting the measured base resistance value in this test.*

The report does not state what the design values were, but given the deflections under the maximum load, it would not be unreasonable in the sounder bedrock to use the highest measured values as conservative design values. These values are well above typical design values.

4. GUIDELINES FOR CONDUCTING COLORADO’S NEW AXIAL LOAD TESTS ON DRILLED SHAFTS

4.1 Introduction

This chapter presents:

- Step-by-step procedure on when it is cost-effective to consider load tests as part of the subsurface geotechnical investigation in CDOT future bridge construction projects during different stages of the design phase. The objective of new load tests is not just to obtain research data for improvement of the accuracy of the design methods for drilled shafts, but also to generate significant net savings. After Colorado’s design methods are improved based on sufficient number of load tests, additional loads tests may be performed in the future for purely economical reasons.
- Guidelines for planning, design, and construction of new load tests on drilled test shafts. Sample Guide Specifications for Osterberg Cell Load Testing of Drilled Shafts are presented in Appendix B. Revision of Section 503 of CDOT Standard Specifications to incorporate the Osterberg Cell Load Test in the Broadway construction project is presented in Appendix C.
- Analysis Procedure of Osterberg Cell (O-Cell) Load Test Results.

The comprehensive guidelines suggested in this chapter for conducting new load tests were applied in the Trinidad project. Chapter 5 provides specific details of all the steps employed for the planning, design, construction, and analysis of the Trinidad two load tests.

The CDOT Region Office administrates all the design activities of project development. They are:

1. Environmental Assessment (EA) phase to assess the impacts of the project on the environment. This is usually performed in relatively large corridor projects with multiple structures under what is called the “Corridor Study.” A certain amount of Basic Engineering (BE) needs to be performed as part of this work. Geologic hazards assessment, possibly geologic mapping, and perhaps a few geotechnical borings are performed during the EA/BE study.

2. Development of a Scope of Work for the design of the structure (Scoping Stage). This scope is executed during the FIR and FOR phases of the project, either by various CDOT design offices, or by consultants who have contracts and task orders with CDOT. On many of the Region's projects, a consultant does the bridge plans. The Region needs to know the scope of work as soon as possible to enter into contract with the consultant.
3. FIR or the preliminary design phase. Often, the geotechnical investigations and recommendations are completed in this phase, and 30% of the design of the bridge superstructure is completed.
4. FOR or the final design stage. Design is completed in this stage.

There are three levels of geotechnical axial design of drilled shafts installed in Colorado's bedrock shales:

1. Level I Design based only on the results of the standard penetration test (N-values) which is the most common practice in Colorado. Abu-Hejleh et. al. (2003) described improved SPT-based design methods to estimate the ultimate unit base resistance, unit side resistance, and settlement of shafts installed in soft claystone (SPT N-value less than 100 blows per foot or bpf). This level of design could also be employed for the harder rock formations, if the advanced levels of design suggested below are not appropriate or cost-effective (see below for more details).
2. Level II Design based on the unconfined compressive strength results on recovered rock core specimens in addition to the SPT test. This design is only appropriate when very hard shale bedrocks exist with SPT N-value greater than 100 bpf or 50 for 6" penetration). For shafts installed in bedrock formations as those encountered at the Franklin and Broadway sites, Abu-Hejleh et. al. (2003) described improved strength-based design methods to estimate their ultimate unit base resistance, unit side resistance, and settlement. In this case, it is recommended to perform SPT on 70% (this can be adjusted by the geotechnical engineers) of the test holes. On the other 30% of test holes, to perform SPT until the rock is very hard and then switch to coring with either triple-walled or double-walled core barrel to recover rock specimens for the lab strength tests. If reliable rock core specimens cannot be collected because of the thinly bedded or jointed nature of the weak rock, it is recommended to

conduct the in situ pressure meter test (PMT) to estimate the strength of the rock. This is a more advanced and reliable design procedure than Level 1 and can lead to significant savings to the project.

3. Level III Design based on full-scale load testing of test drilled shafts similar to the production shafts at certain locations and geotechnical investigation as in Level II on the remainder of the project. This is the most accurate and advanced level of geotechnical design and provides all the information the designer needs (unit base and side resistance values, and settlement at working loads). This level of design is only cost-effective for drilled shafts embedded in very hard shale bedrocks as those described under Level II Design.

4.2 Requirements for Cost-Effective Load Tests

Consideration of load tests on drilled shafts in the subsurface geotechnical investigation is cost-effective if all the following conditions are met:

- 1) Projects with large number of drilled shafts required to support large bridges and when total construction costs for all phases of the project exceeding \$10,000,000. The greatest number of large bridges that will be supported by drilled shafts will be associated with limited-access highway corridor improvement projects.
- 2) Penetration depth of the drilled shafts is controlled by axial load, not lateral load.
- 3) Presence of:
 - a. Very hard claystone and/or sandstone shale bedrocks with SPT N value larger than 50/6" and confirmed to be rock-like geomaterial per Colorado Testing Procedure 26-90.
 - b. Soft Claystone with SPT-N value larger than 50 and when artificial shear rings will be employed for roughening the sides of the shaft holes.
- 4) Net savings are expected based on cost-benefit analysis.

If one of these conditions is not met, then the current SPT-based design methods (Level 1) should be employed for the design of the drilled shafts because savings will not offset the additional testing costs required in Level 2 and 3 Designs.

4.2.1 Meeting the 1st and 2nd Requirements

The structural engineer, geotechnical engineer, resident engineer and project engineer for the project should meet to decide if the 1st and 2nd conditions are met.

As load test costs on large projects will be less than one percent of project costs, there would not be a significant impact on total project costs even if there were no immediate benefits, which is an unlikely outcome on most projects.

The load tests performed in the Snowmass Canyon project have the potential to generate savings in the millions according to Dr. Liu from CDOT Geotechnical Office. The results of the T-REX and Broadway load tests presented in this report were used to improve the geotechnical design of the production shafts in these two projects. Broadway's production shafts were redesigned based on the load test results. The ultimate side resistance was increased from as low as 4.8 ksf to 15 ksf and for both the side and base resistance the resistance factor, needed in LRFD, was increased from 0.55 to 0.8. The total savings are estimated at \$140,000. 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. Based on the Trinidad load test (Chapter 5), it was recommended to reduce the rock socket length for the 4' diameter shafts from 59 ft to 8 ft. Savings in the Trinidad project from the load tests were \$113,000.

4.2.2. Meeting the 3rd Requirement

Preliminary subsurface geotechnical investigation (Phase 1) is recommended to address the 3rd condition before conducting the complete subsurface geotechnical investigation in Phase 2.

Phase I (Preliminary) Geotechnical Investigation (before or during the scoping stage of the design) The objective of the 1st phase is to get a general idea of the strength, geology, characteristics, and the variability of the rock that will support the bridge drilled shafts. Few test holes should be drilled covering the entire area of the project site. On each test hole, it is

recommended to perform SPT until the rock is very hard (N-value are much higher than 50/6”) and then to switch to coring with either triple-walled or double-walled core barrel to recover rock core samples for the lab strength tests (unconfined compression tests or UCT). The extent of the geotechnical investigation should be finalized by the Geotechnical Engineer based on the variability of the rock and the previous geotechnical work in the area.

The presence of very hard and competent claystone and/or sandstone bedrock formation (like at the Broadway and Trinidad sites) should be verified based on the results of: 1) SPT-N values (larger than 50/6”), 2) UCT (unconfined strength larger than 40 ksf), and 3) results of Colorado Testing Procedure 26-90 (to verify it is rock-like geomaterial that is durable, not sensitive to water, and has very small potential for creep).

It is highly recommended to perform the Phase I investigation before working on the scope of work for the design of the structure. It could be performed during the Environmental Assessment Phase of large corridor projects. For bridges that will be replaced, information needed for Phase I could be obtained from previous geotechnical investigations performed at the old bridge. As a last resort, it can be performed during the design scoping stage. The cost of a Phase I investigation is approximately \$5000 to \$10,000 for several scattered holes along the entire project site. Additional minimal costs to the project will come from performing two mobilizations instead of one for the subsurface geotechnical investigation (for Phase I and Phase II). These costs may be compensated if advanced level of geotechnical investigation found very cost-effective or from savings in other projects.

4.2.3 Meeting the 4th Requirement

General guidelines to determine the number and type of load test tests are presented in the next section. Then, additional costs of the load test program and the geotechnical investigation around the test shafts can be determined. The potential benefits of load tests in increasing the design unit base and side resistance values and reducing the factor of safety are also discussed in the next section. Based on the cost and benefit information, the structural engineer will determine the potential net savings of load tests.

4.2.4 Finalize the Scope of Work for the Geotechnical Investigation and Design Work

Based on the results of step 3, one of the three levels of geotechnical design and subsurface geotechnical investigation presented before should be selected and included in the scope of geotechnical design work. The structural engineer, geotechnical engineer, resident engineer and project engineer for the project should meet to decide if load tests should or should not be included in the scope of design work. If the net savings of the load tests were not significant, the option of Level 2 Design should be considered. It is roughly estimated that the cost of Level 2 geotechnical design and its geotechnical investigation is twice the cost of the 1st level of design. If the net savings are expected to be minimal or costs are more than savings, Level 1 Geotechnical Design should be recommended.

Guidelines for planning and performing new load tests are described in the next section. An important decision should be made if the load test can be performed in a timely manner to develop or change the design recommendations for drilled shafts:

- Conducted before the FIR meeting. In this case, the scope of work for Level III design should be referenced in the scope of design and field work (Scoping stage).
- Performed in the FOR design phase where last minute design changes to drilled shafts could be made. A new contract to perform the load tests may be needed in this case.
- Performed per a separate construction project prior to the main construction project, or in the early stages of the construction project. In these two cases, a caisson design change in a CMO will be needed. In the construction plans, there should be an option to adjust the shaft penetration lengths upon completion of load tests. The project planners should be on the lookout for a situation where a separate project or Phase 1 construction project could be done for conducting the load test so that the foundation resistance values could be available to designers in a timely manner to incorporate into the Phase 2 of the project.

4.3 Guidelines for Planning, Design, and Construction of New Load Tests on Drilled Test Shafts (Level 3 Design)

Sample Guide Specifications for Osterberg Cell Load Testing of Drilled Shafts are presented in Appendix B. Revision of Section 503 of CDOT Standard Specifications to incorporate the Osterberg Cell Load Test in the Broadway construction project is presented in Appendix C.

Level 2 Geotechnical subsurface investigation should be performed along the entire project site. Based on the results of this investigation, load tests can then performed as presented in the following subsections.

4.3.1. 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”). The load test data can then be used: 1) to design the production shafts with more confidence (smaller FS around 2 and higher resistance factor ϕ around 0.8) and using higher ultimate unit base and side resistance values that may result in significant savings to the project, 2) as research data to improve the future design methodology of Colorado’s drilled shafts installed in weak rocks.

4.3.2 Location and Number of the Load Tests

Test locations should be selected following one or more of the following 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 (not relevant if uniform rock is encountered at the site).
- ❑ In flat areas accessible to large equipments (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 the rock.

The number of load tests should be determined based on the variability of the site rock layers as identified in the results of Level 2 geotechnical subsurface investigation. If multiple geologic formations exist on the site, load testing within each formation should be considered. For a uniform site, or if the weakest rock 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 and to provide a sense of consistency, especially because capacity of the shafts is influenced so strongly by construction. 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, it is a good idea to consider a load test in the weakest rock area and a second in the strongest rock area to investigate the correlation between rock strength and resistance.

4.3.3. 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:

- ❑ 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.
- ❑ 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.
- ❑ 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.
- ❑ 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.

4.3.4. Types, Features, and Costs 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.

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 separate information on both the side resistance and the base resistance. If a conventional load test is not instrumented, it would not be possible to separate the side resistance

load transfer curve (f-w curve) from the base resistance load transfer curve (q-w curve). If the conventional load test is instrumented, it is possible to separate the two load transfer curves and even to measure any interaction between them. 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).

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

- ❑ Whether the test shaft 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 for uniform type of rock (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.
- ❑ 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 kentledge 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-diameter 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 on a full-sized drilled shaft (36 - 48 inches in diameter by 60 - 70 feet deep), that is instrumented, the cost are typically larger than about \$125,000. 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 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. Statnamic load tests can be done after the shaft has been installed whereas the O-Cell test cannot. Statnamic or other high-strain load tests are probably more cost-effective than O-Cell load tests but methods of interpretation of these tests are not finalized yet.

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 Statnamic tests 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.

4.3.5. Geotechnical Investigation around the Test Shaft

If the geotechnical subsurface investigation at the location of load test is performed before construction of the test shaft, drill test holes as close as possible to the future center-location of the test shaft. If drilling will be performed after construction of test shafts, 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 the test holes at one pier diameter (D) from the edge of test shaft ($3D/2$ from the center of the shaft). Subsurface geotechnical investigation methods at the test holes will include auger drilling with standard penetration testing, coring with subsequent laboratory testing on recovered core specimens, and, *if needed*, 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. Note that if the claystone or sandstone shale layer is very hard, two test holes will be needed as there will be no need to perform the pressuremeter tests.

Three test holes are recommended to be drilled around the test shaft:

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- or NX-sized core barrels and collect rock core runs. An unconfined compression test to determine the rock unconfined compressive strength and other lab tests will be conducted on each rock sample as described by Abu-Hejleh et. al. (2003).
3. Based on the test results, visual examination of the recovered samples and core runs, and drilling information, the boundaries of different weak rock layers will be defined.
4. In the third test hole, and for each uniform rock layer, at least one Menard pressuremeter test will be performed. From the pressuremeter test (PMT) 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. CDOT Research Report 2003-8 (Abu-Hejleh et. al., 2003) provides guidelines for preparation of the test holes for the PMT and conducting the test in accordance with ASTM D-4719 and for analysis of the test data.
5. 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.

4.3.6. Design of the O-Cell Load Test

The study will recommend the use of the O-Cell load test in Colorado's future drilled shaft load tests until more cost-effective and innovative load test methods become available.

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

Specific Objectives of Load Tests: The first task in the design of a load test program is to predict the ranges of unit resistance values that should be expected from the load test (that will be performed) based on the results of subsurface geotechnical tests and results of previous load tests in similar rock formations. That is to establish a lower limit of f_{\max} ($f_{\max L}$) and upper limit ($f_{\max U}$), and a lower limit of q_{\max} ($q_{\max L}$) and upper limit ($q_{\max U}$) and the measured maximum unit resistance values are expected to fall between these ranges. The $f_{\max U}$ should be as close as possible to the highest possible true unit side resistance of the weak rock. The $q_{\max U}$ should be as close as possible to the highest possible true unit base resistance for soil-like claystone bedrock, and to exceed the unit base resistance value that corresponds to a settlement larger than 0.05 D (preferred 0.1 D) for the very hard claystone and sandstone bedrock shales.

Two O-Cell load tests on two separate test shafts at two different locations are 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 make 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 side resistance load transfer curve (f-w curve) up to $f_{\max U}$, and the secondary objective is to obtain a portion of the base resistance load transfer curve (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:
 - Almost 90 % 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).

- 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 $q_{\max U}$, and the secondary objective is to obtain a portion of the f-w curve, which will confirm the results of the 1st load test.

Layout of the Test Shafts: Depth of overburden to competent rock should be determined from the subsurface geotechnical investigation. The length (L) and diameter (D) for the embedment of all the production shafts in the rock should initially be computed based on the recommendations of the geotechnical engineer for the construction project. Using the $f_{\max L}$ and $q_{\max L}$, and a resistance factor of 0.8 (or FS of 2), L and D should be reevaluated for all production shafts to estimate the potential cost savings from the load tests. These savings could be finalized after the load test results are obtained.

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 the load test result 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 by Abu-Hejleh et. al. (2003). 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 smaller shaft diameters (4 ft not 7 ft). 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.

In the design of the load tests, the longest L value should be considered (including the L expected 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 values (e.g., from 12 ft to 21 ft) on the measured resistance values is expected to be small.

1st O-Cell load test: 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,

$$q_{\max L} \times A_b \geq (L_2 - L_1) 3.14D \times f_{\max U} + \text{overburden side resistance, 4.1}$$

$$\text{Capacity of O-Cell} \geq (L_2 - L_1) 3.14D f_{\max U} + \text{overburden side resistance,4.2}$$

where L_2 and L_1 are the lengths of shafts in the competent rock above and below the O-Cell. Based on Eqs. 4.1 and 4.2, the proper location and capacity of the 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_2 should be large enough to guarantee a base failure before side failure:

$$q_{\max U} A_b \leq (L_2 - L_1) \times 3.14D f_{\max L} + \text{overburden side resistance,4.3}$$

$$\text{Capacity of O-Cell} \geq q_{\max U} A_b + L_1 \times 3.14D f_{\max U} \text{4.4}$$

4.3.7. Instrumentation of the Test Shafts

Consider the use of LVDTs to measure accurately the upward movement of the O-Cell and top of the shaft. 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.

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 tilting. It seems that there is evidence of small tilting of the O-Cell in almost all O-Cell load tests performed in the T-Rex and Broadway projects. 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. Consider the use of two PVC pipes that extend past the O-Cell to place this grout. Post-grouting the base of the shaft is needed when the hole is “wet” as will be discussed in the next subsection.

4.3.8. Construction of the Test Shafts

Use augers with cutting teeth for drilling for rapid and continuous drilling of the shaft hole with minimal use of water during drilling, and no use of slurry and casing in the portion of rock socket used for load resistance.

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. The project engineer should approve the plans for the O-Cell load tests and should be able to make changes to the locations of the O-Cell and strain gages. The design engineer should provide Revision of Section 503, Osterberg Cell Load Test (see Examples in Appendices B and C). Other details are described in the following. The test shafts should be constructed in accordance with Section 503 of CDOT standard specifications and EXCATLY/IDENTICAL as

expected in the production shafts (the same materials, drilling method, cleaning procedure, and placement of concrete). The rate of rise of concrete in the production shaft should be at least 12 m (40 feet) per hour and the slump is 7 - 8 inches in order to ensure that ground stresses have reestablished. For shafts embedded in 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 compressive strength and stiffness of the concrete at time of load test must be determined in the lab and used to estimate the composite Young modulus of the shaft.

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.

Contingent grouting procedure should be specified that would be employed if the test shafts turned to be wet during construction. If the shaft is dry then no grouting is required. A wet shaft is defined as a shaft filled with water- or shaft that even if pumped out, has a water intrusion rate higher than the pumping out rate. In this case, CDOT placement procedure for the concrete might lead to low quality concrete above or below the O-Cell. Basically, it will be hard to get a tremie pipe past the O-Cell. Placing a concrete plug/pad may be work with little water. But with significant water, grouting is the best procedure to ensure uniform high quality concrete/grout around and below the cell. The grout should be placed below the O-Cell (1 ft) and above the O-Cell (around 2 ft). The grout strength will be very close to the strength of the concrete of the production shafts (around 4000 psi is OK). The grout should be a standard mix and be installed according to CDOT placement standards for drilled shafts under water. Make cylinders of the fresh grout mix as you do for fresh concrete and make sure that CDOT strength requirements for the grout are met. After placement of grouting for two feet or so above O-Cell, the construction and materials of the remaining of the test shafts will follow CDOT requirements for placement of concrete under water (Section 601.12f), and as approved by the engineer.

4.3.9. Data Collection at the Load Test Site

Three types of data should be collected:

- ❑ A report for the subsurface geotechnical investigation for the entire project (Level 2 Geotechnical Investigation) and at the load test site.
- ❑ A report on the load test from the testing company. These data will be analyzed very carefully as described in the next section 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.
- ❑ Cost, total and net saving of the load tests to the project, and other benefits of the load tests.
- ❑ 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).
 - Any available 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 .

Examples of the construction, materials, and layout information collected for the Broadway and T-REX test shafts are shown in Table 4.1. 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 4.1 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). Complete information on the excavation and concrete placement methods of the drilled shafts are presented in Chapter 2.

4.4 Analysis of Osterberg Cell (O-Cell) Load Test Results

The recommended analysis of the O-Cell load test results is similar to the analysis employed by Abu-Hejleh et. al. (2003) for the analysis of the O-Cell load tests performed in the T-REX and Broadway projects, as presented in the following sections.

A photograph of the Osterberg Cell is shown in Figure 4.1. The O-Cell test is performed by applying hydraulic pressure to the O-Cell which acts equally 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. The load increments were applied using the Quick Load Test Method (ASTM D1143). The test shafts were also instrumented to record the upward deflection of the shaft head and upward and downward movement of the O-Cell as the load is applied in increments (Figure

6.2). The I-225 and County Line tests were continued until the ultimate side shear, the ultimate end bearing, or the capacity of the O-Cell was reached. Unfortunately, this was not the case for the production test shafts at Franklin and Broadway, where the maximum applied load was limited to maintain the functionality of these production shafts for supporting the bridge loads after test completion. However, the applied load exceeded two times the design loads as often recommended for proof load tests.

Table 4.1. Typical 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		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

Examples of the measured and analyzed results from the O-Cell load Tests are presented in Figures 4.2 to 4.7. Loadtest, Inc. performed the O-Cell test and provided the following test

results: gross O-Cell load versus upward and downward movement of the O-Cell (Figure 4.2), and, from results of strain gages, unit side resistance for different zones across the test shafts vs. the upward movement of the O-Cell (Figure 4.3), and the equivalent top load vs. settlement curve. It is assumed in the Loadtest, Inc. analysis and 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.

4.4.1 Determination of the Load Transfer Curves

For research and design needs, it was important to extract from the O-Cell data the most accurate load transfer curves for the weak rocks: settlement (w) versus base unit resistance (q) until the maximum unit base resistance q_{\max} is reached for the weak rock layer encountered beneath the test shaft, and side movement (w) vs. unit side resistance (f) until the maximum unit side resistance f_{\max} is reached for all weak rock layers encountered across the test shaft.

f-w Curves: There are many sources for errors in estimating the side resistance from only the strain gages (Abu-Hejleh et. al, 2003). In addition to the results from strain gages, 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 (Figure 4.4). 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.

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 . This includes curves of average side resistance versus average side movement in the entire bedrock socket (Figure 6.4) and similar

curves extracted based on results of strain gages (Figure 4.3). In order to estimate the side resistance in the entire 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. For correlation purposes, the average weighted SPT-N values and rock strength was estimated in different zones where f-w curves were generated using the measured results from the geotechnical testing program performed around the test shafts.

q-w curves: 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 provided by the testing company. 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:

- ❑ Use the side resistance vs. movement curve measured from O-Cell to the next level of 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 in strain gages results as discussed before.
- ❑ Use the side resistance vs. movement curve obtained for the entire bedrock socket above the O-Cell. This is conservative approach for estimating the side resistance and possibly 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) for any unit base resistance q (Figure 4.5).

4.4.2 Definitions of Tolerable Settlement, Ultimate Unit Base Resistance, and Ultimate Unit Side Resistance

The tolerable limit for settlement should be finalized by the structural engineer. AASHTO (2002) guidelines provide recommendations on the permissible differential settlement for bridges related to the length of the span and type of the span (simple or continuous support). A tolerable settlement limit of 0.65" was selected for the T-Rex project.

The proper selection of the definition for ultimate resistance values is controlled by the availability of load test data taken to large displacements and the need to limit the shafts settlement at service loads. The adopted definitions of ultimate resistance in this study (could be adjusted in the future when more data become available) are:

- ❑ For the soil-like claystone (County Line and I-225 sites), f_{\max} and q_{\max} that correspond to the full mobilization of the resistance in the plastic resistance (true resistance).
- ❑ For the very hard claystone and sandstone encountered at the Franklin and Broadway sites, q_{\max} to correspond to displacement of 5% of the shaft diameter, but not to exceed 3 inches, and f_{\max} to correspond to a displacement of 1% of the shaft diameter, but not exceed 0.6 inches. The 3 and 0.6 inches values are suggested to limit excessive settlement of large diameter shafts at service loads. Once the ultimate side resistance was obtained, it was assumed to remain constant until a movement of 5% of the shaft diameter occurred.

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, if needed, up to a settlement of 0.01D 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 unit base resistance.

4.4.3 Construction of the Equivalent Top Load-Settlement Curve from the Results of the O-Cell Test.

Applying the definitions for ultimate resistance as presented before, q_{\max} and f_{\max} (f_{\max} for the entire bedrock socket) were obtained and utilized to calculate the ultimate resistance load of the shaft, Q_{\max} , as $A_b q_{\max} + A_s f_{\max}$ where A_b and A_s are, respectively, the base and side areas of the shaft in the rock. The allowable design base and side resistance values and loads as determined from the O-Cell load test results can be determined using FS (factor of safety) of 2 as $q_{\text{all}} = q_{\max}/2$, $f_{\text{all}} = f_{\max}/2$, and $Q_{\text{all}} = Q_{\max}/2$.

Using the obtained q-w and f-w curves, 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.

A simple procedure is also recommended to construct a simple load-settlement curve as follows. Assume initially that the shaft behaves as a rigid shaft and therefore settlement (w) at base and head of the shaft are the same. For arbitrary settlement, w , the corresponding unit side resistance, f , and base resistance, q , are estimated from the extracted q vs. w and f vs. w curves. (f for the entire bedrock socket). Note that uniform unit side resistance distribution is assumed across the entire bedrock socket, estimated for any movement w from the f - w curve. The shaft side resistance in the overburden is neglected. Then, the shaft resistance load or the equivalent top load, Q , for the arbitrary settlement, w , can be estimated by adding the base resistance load ($Q_b = A_b q$) to the side resistance load ($Q_s = A_s f$) as $Q = Q_b + Q_s = A_b q + A_s f$. This should be repeated for several arbitrary values of settlements to obtain several (Q , w) points up to the ultimate resistance load or Q_{\max} (Figure 4.6).

The shaft head load-settlement curve should then be modified to take into account the compressibility of the shaft, in which the settlement of the shaft head is $w + \delta$, where δ is the elastic shortening of the shaft, and settlement of the shaft base is w . The elastic compression is only important in high-capacity drilled shafts embedded in hard claystone and sandstone bedrock

(e.g., Franklin and Broadway shafts). For any arbitrary settlement of w , calculate Q , Q_s , and Q_b as for rigid shafts. The axial load in the shaft in the overburden is Q (no change from top load because side resistance in the overburden is assumed negligible). The shaft axial load at the base is Q_b . The average shaft axial load in the bedrock socket is $(Q+Q_b)/2$. The additional elastic settlement can now be calculated as $\delta = (Q/A_b E_c) L_o + L/(A_b E_c) (Q+Q_b)/2$, where L_o is the length of the shaft in the overburden and L is the length of the shaft in the rock. Now a new point of $(Q, w+\delta)$ is obtained. This should be repeated for several arbitrary values of settlements, w , to obtain several $(Q, w+\delta)$ points up to the ultimate resistance load or Q_{max} (Figure 4.6). To simplify the analysis and to be conservative, similar Q is assumed for both rigid and compressible cases (Figure 4.6, difference only in settlements). In reality, the additional elastic compression of the shaft generates more side movement (more than w) leading to additional side resistance. This requires adjustment of the resistance load of the shaft (Q) in an iterative procedure until the change in load between two successive iterations become negligible.

4.4.4. Construction of A simple Equivalent Top Load-Settlement Curve from the Results of Simple Geotechnical Tests

It would be of interest to get an approximate estimate for the load-settlement curve, especially settlement 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 $(Q_{max}, 0.05D)$ as shown in Figure 6.7. The ultimate shaft resistance load (Q_{max}) corresponds to a settlement of $0.05D$. At a settlement of $0.05D$, most of the base and side resistance for shafts embedded in 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. Analysis of adequate number of load tests in the future should attempt to identify correlation relations between q_{max} , f_{max} , q_d , f_d and the simple test results of SPT, UC, and PM test data as was performed by Abu-Hejleh et. al. (2003).



Figure 4.1. Photo of the O-Cell Placed in the Broadway Test Shaft.

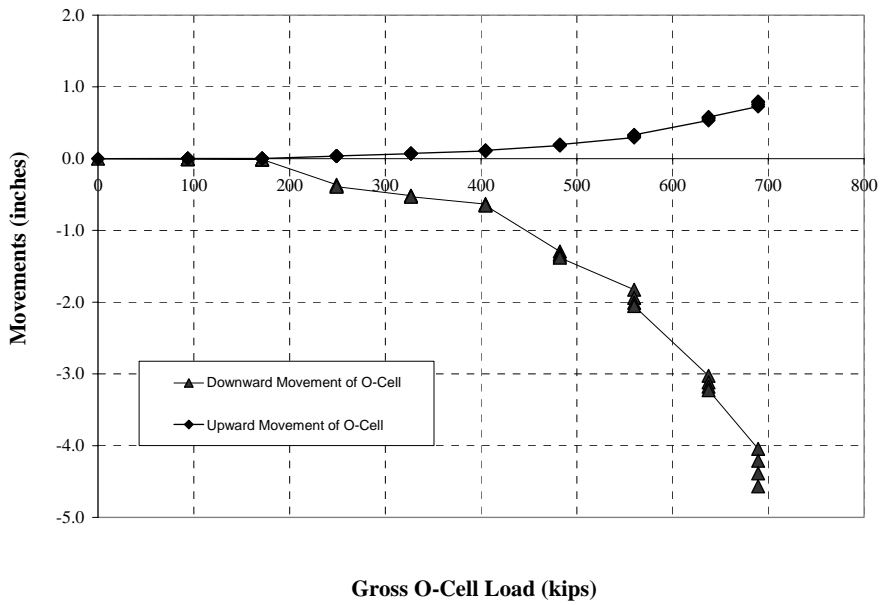


Figure 4.2. Results of O-Cell Load Test at the County Line Test Shaft.

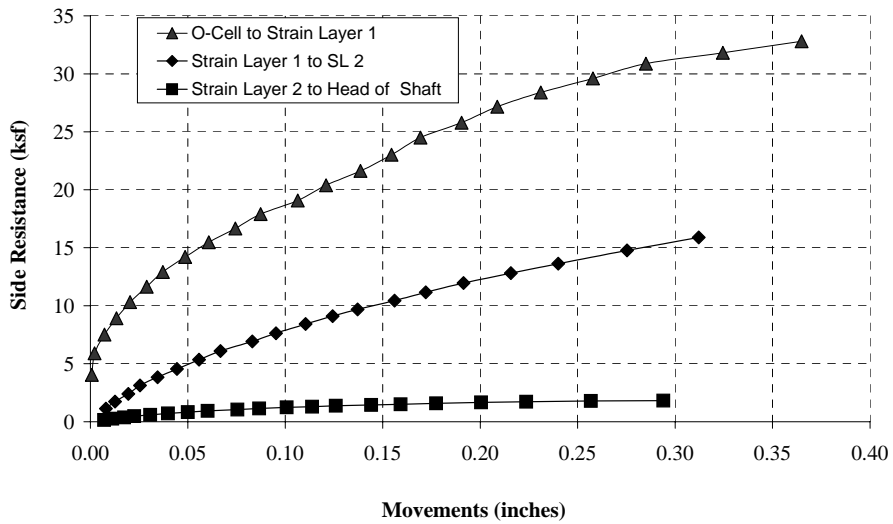


Figure 4.3. Unit Side Resistance vs. Upward Movement for the Broadway Test Shaft

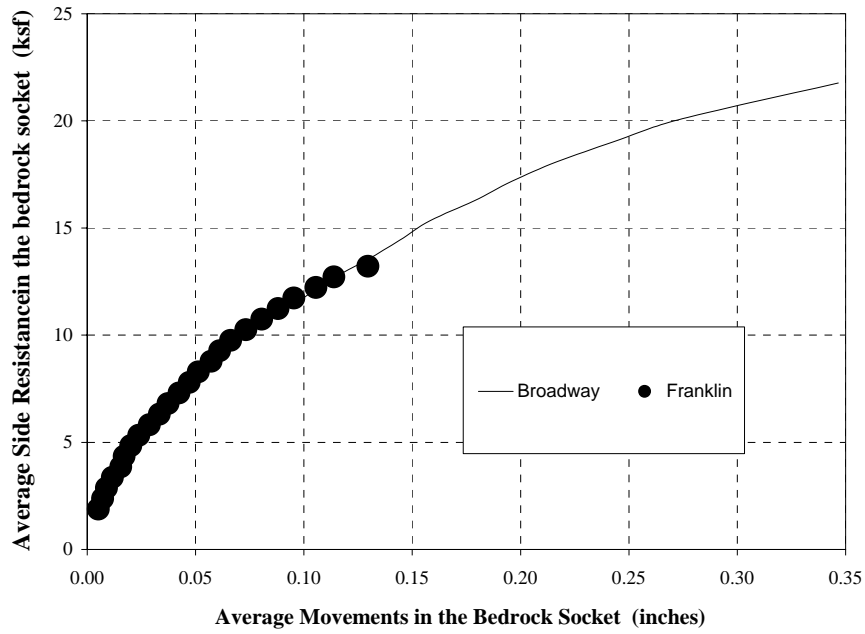


Figure 4.4. Unit Side Resistance vs. Upward Movement in the Entire Bedrock Socket: Franklin and Broadway Test Shafts.

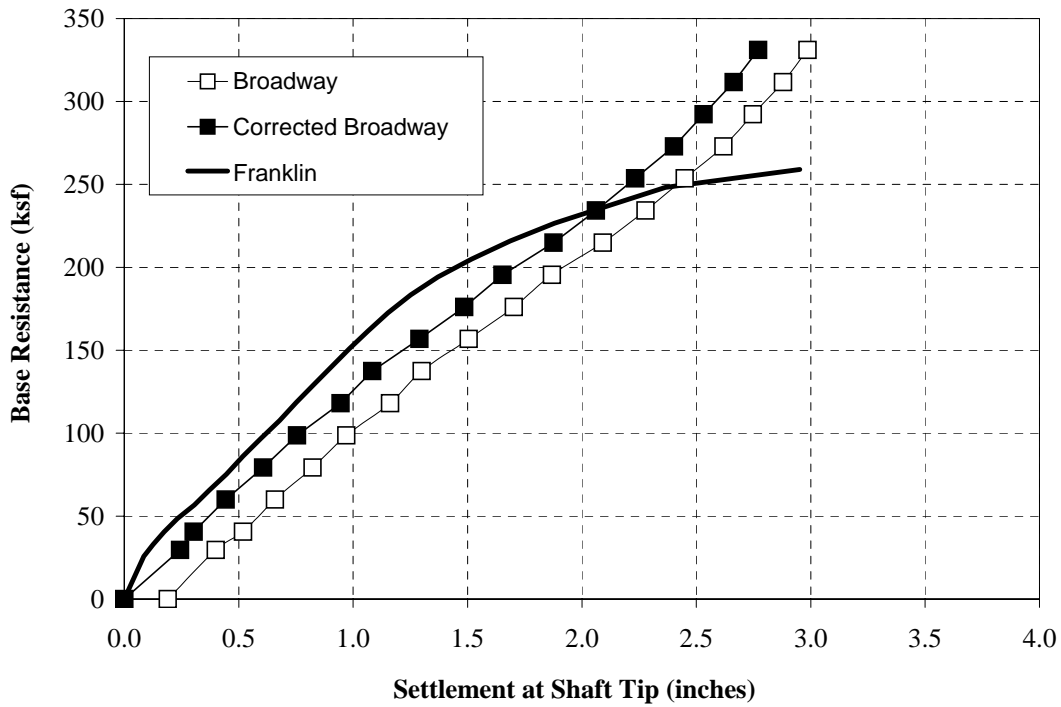


Figure 4.5. Unit Base Resistance vs. Settlement: Franklin and Broadway Test Shafts.

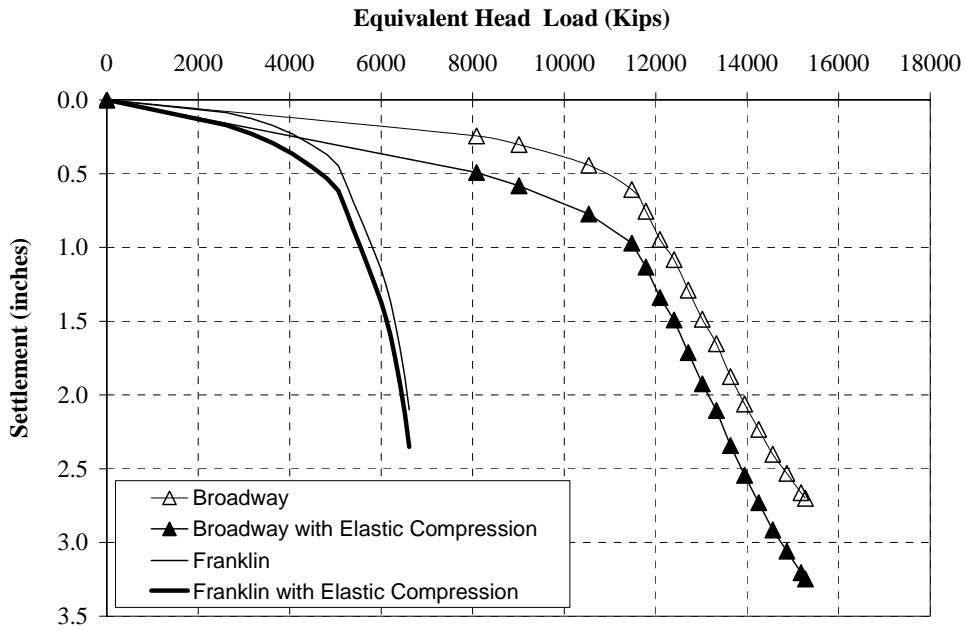


Figure 4.6. Extracted Load-Settlement Curve: Franklin and Broadway Test Shafts.

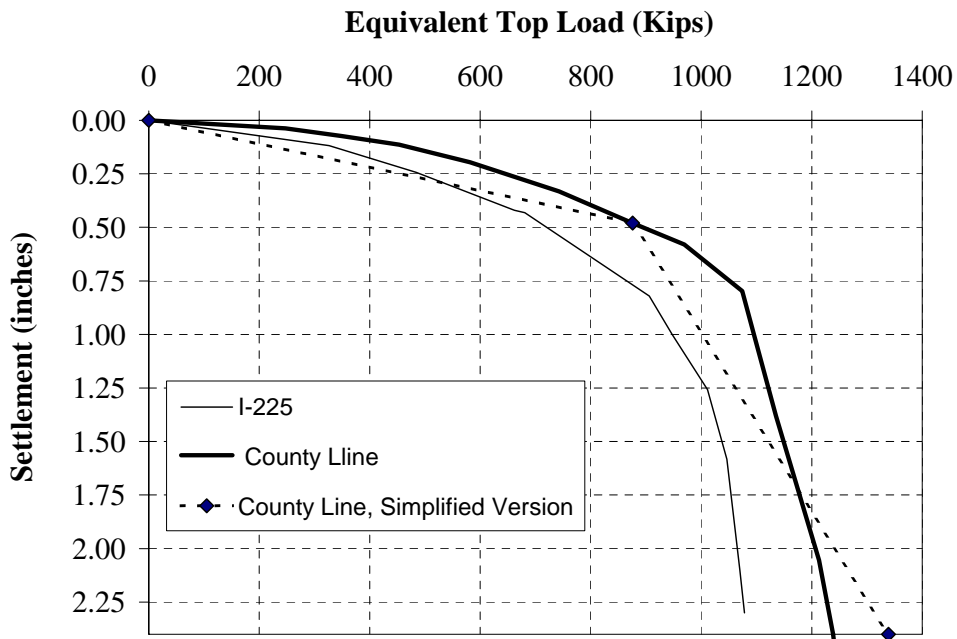


Figure 4.7. Extracted Load-Settlement Curves: I-225 and County Line Shafts.

5. TRINIDAD LOAD TESTS: EXAMPLE OF APPLICATION OF THE PROPOSED GUIDELINES FOR CONDUCTING NEW AXIAL LOAD TESTS

The comprehensive guidelines suggested in Chapter 4 for conducting new load tests were applied in the Trinidad project. This chapter provides specific details of all the steps employed for the planning, design, construction, and analysis of the Trinidad two load tests. CDOT engineers should benefit from this example for conducting future load tests.

5.1 Overview

As a part of the I-25 Trinidad project located in Trinidad, Colorado, two O-Cell load tests were performed on 48” diameter test shafts. Test # 1 is referred to as the “South Load Test” and Test # 2 is referred to as the “North Load Test.” Comprehensive subsurface exploration and laboratory testing was performed around the two test shafts by Ground Engineering (2003). Results of this investigation are presented in Appendix E (Part 1). LOADTEST Inc. (2003) performed the load tests and provided the testing reports (Part 2 of Appendix E presents some information reported in the test reports). Summary information on the layout, materials, and construction of the test shafts and the results from the load tests and the subsurface geotechnical investigation program are presented in Table A.5. Dr. Naser Abu-Hejleh from CDOT Research Office designed these load tests and later analyzed them following the procedure described in the previous chapter. The purpose of these two load tests was to check the recommended geotechnical design parameters for the drilled shafts in the construction project and to provide research data that would improve the accuracy of CDOT future design methodology for drilled shafts.

5.2 Subsurface Conditions and Strength Characteristics of the Bedrock

Four test holes were advanced around the proposed load test locations (Boring # 1A and # 1B around the north load test location, and # 2A and #2B around the south load test location). See Part 1 of Appendix E for locations of these two test holes and all obtained testing results. Location of test holes were selected following the criteria listed in Chapter 4. In the first test hole, SPT was conducted every 5 feet. In the 2nd test hole, 5 ft rock core runs were collected and the recovery ratio and the RQD were determined in the field. The subsurface conditions

encountered in the test holes consisted of man-made fill and natural sand and gravel underlain by very competent Pierre shale bedrock at depth approximately 28.5 to 20 ft below the existing grade (see Boring Logs in pages E-5 to E-10). The encountered Pierre shale was very well cemented, typical RQD from 85 to 100, fine grained, low to medium plastic, very hard, slightly moist, and gray to dark gray in color. Free groundwater was encountered at depth around 14 feet during drilling rising to a depth of 11 feet 6 days after drilling. The driving resistance was very high with typical SPT-N values of 100 blows for penetration of 2". Unconfined compressive tests were conducted on selected and intact rock core samples to determine the unconfined compressive strength, q_u , of the underlying bedrock (see Table 1 on page E-11). The unconfined compressive strength of the bedrock around the test shafts ranges from 330 ksf to 518 ksf. Classification tests on the bedrock samples were not possible because the bedrock was well cemented so it is called here in as Pierre Shale (not claystone or sandstone as at other locations).

5.3 Recommendations for Design and Construction of the O-Cell Load Tests

The two Trinidad O-Cell load tests were designed (as in Section 4.3.6) based on the results of subsurface geotechnical investigation presented before and the reported project information,

For typical pier shafts, the structure engineer provided in the construction plans two options for the diameters of the production shafts: two 4 ft diameter shafts each shaft supporting factored axial load (P_u) of 2600 kips, or one 7 ft shaft diameter that supports P_u of 5200 kips. For the main span shafts, two options were available in the plans: three 4 ft diameter shafts, each supporting P_u of 4863 kips, or two 7 ft shafts, each supporting P_u of 5280 kips. Therefore, two extreme scenarios need investigation: the 4 ft diameter shaft with P_u 4863 kips (main span) and the 7 ft diameter shaft with P_u of 5280 kips (typical shafts). *The design practice is to require the length of the rock socket (L) to be at least 3 times the shaft diameter ($L=3D$).* The geotechnical engineer for the construction project recommended a very conservative ultimate side resistance of 8 ksf with a resistance factor of 0.55 and ultimate base resistance of 300-ksf with a resistance factor of 0.5. These are very conservative design values as will be discussed later.

5.3.1 Selecting L and D for the Test Shafts

The layout of the test shafts for O-Cell load test should be similar to those planned in the production shafts. The production shafts should be designed based on the results of the load test. Therefore, the layout of the test shaft for the O-Cell load tests should be designed based on the expected minimum resistance values obtained from the load tests.

Based on the measured unconfined compressive strength, q_u , and the results of load tests at Broadway site, it was estimated that the ultimate unit side resistance, f_{\max} would fall between 20 and 40 ksf (20-40 ksf) and the ultimate unit base resistance, q_{\max} would be larger than 350 ksf. A maximum q_{\max} value of 400 ksf was set for design of the O-Cell load test because it felt it would be harder to use higher values in the design of the production shafts, even if justified by the load test. These resistance values are valid only if 4' diameter shafts are employed. For the 7' shafts, the minimum f_{\max} can be assumed 10 ksf and the minimum q_{\max} can be assumed 200 ksf (see scaling relations provided by Abu-Hejleh et. al., 2003).

Selecting D: The design calculations for the 7' ft diameter shafts, using f_{\max} and q_{\max} expected from the load tests, suggested that $L=3D$ will govern the design. Even if the conservative design values recommended by the geotechnical consultant are employed, $L=3D$ will continue to govern the design. Therefore, there were no benefits to the construction project from the O-Cell load test on the 7' shafts. Also, the costs of performing an O-Cell Load test on 7' shaft will be very large (higher capacity of loading system is needed). *Therefore, the diameter of the two test shafts is recommended to be 4'.*

Selecting L: For the 4' ft diameter shafts, L was calculated twice: first based on f_{\max} and q_{\max} recommended by the geotechnical engineer for the construction project (before the O-Cell load test is performed), and second based on the lower limits of side and base unit resistance values expected from the load test results. Resistance factors of 0.8 are employed because two load tests are performed in the projects. The design calculations suggested that the load test had the

potential to reduce L from 53 ft to 12 ft for the main span piers. This provided a rough estimate of the savings that should be expected from the load tests. The savings and design of the production shafts were finalized after the load tests had been performed.

L for the production shafts could extend from 12' (for the 4' shaft) to 21' (for the 7 ft shaft). It is important to get side resistance values of all rock layers expected in the production shafts. Therefore, L was selected as 12 ft for the 1st test shaft and 21 ft the 2nd test shaft.

Schematics drawings of the test shafts with diameters (D) of 4 ft are shown in Appendix E. For the 1st load test at the southern location, the socket length (L) is 11.25 ft. For the 2nd load test at the northern location, the socket length is 20 ft. The contribution of overburden to side resistance was eliminated by installing a temporary casing to top of rock and keeping it there until the test is complete. The concrete would then be placed to 1 ft below top of competent rock. The contractor was asked to ensure that the casing and concrete are not mechanically connected.

5.3.2 Selecting the Capacity and Location of the O-Cell

It is assumed in the calculations that bottom of the O-Cell will be placed around 1 ft above tip of the test shafts ($L_2 - L_1 = 1$ ft). This is very close to the placed location of the O-Cell relative to tip of the test shaft (see Part 2 of Appendix E).

1st Load Test (Southern Location, target L= 12 ft): The goal of the 1st load test is to obtain the side resistance up to 40 ksf with a secondary objective of confirming the base resistance up to 350 ksf that will be obtained from the 2nd load test (see previous chapter). Another goal is to investigate if the side resistance might be lessened past the peak resistance (referred to as brittle behavior).

It is important to obtain accurate information for the side resistance, more important than for the base resistance, because: (1) most of the resistance for working loads is provided by mean of side resistance, and (2) the true ultimate unit base resistance of hard shale bedrocks is very large

(Abu-Hejleh et. al., 2003). The results of the 1st load test should be obtained before the test shaft for the 2nd load test is constructed. This is to correct for any problem encountered in the 1st load test and to make any last minute modification to the location of the O-Cell in the 2nd load test (see previous chapter).

In the southern location, clay joints will be present along the sides of the socket (see Appendix E, Part 1) and this why this location was selected for the 1st load test to represent the worst field conditions for side resistance.

Note that perimeter and base area of shaft with $D= 4\text{ft}$ are, respectively, 12.57 ft and 12.57 ft^2 . By applying Eq. 4.1, length of the shaft above O-Cell for the 1st test shaft, L_2 , can be obtained as $L_2 < 12.57*350/ 12.57 *40 + L_1 = 8.75+1= 9.75\text{ ft}$, so L_2 of 10 ft was recommended (close to the placed length of $11.25-0.9= 10.35\text{ ft}$, see Appendix E). Now applying Eq. 4.2, Capacity of O-Cell $\geq 10*12.57* 40= 5028\text{ kips}$. Thus, $34''$ O-Cell is recommended with a capacity of 6000 kips in each direction (12000 kips of maximum total test capacity) is recommended for the 1st load test.

2nd Load Test (Northern Location): The goal of the 2nd load test (unless modified based on the results of the 1st load test) is to obtain the base resistance up to 400 ksf with a secondary objective of confirming the results of side resistance up to 20 ksf that will be obtained from the 1st load test.

By applying Eq. 4.3, $L_2 > (400*12.57)/(12.57*20) +L_1=20+1 =21\text{ ft}$. This is a little greater than the placed L_2 in Test 2 (19.2 ft , see Appendix E, Part 2). Capacity of O-Cell $\geq 400*12.57+ 12.57*1*40= 5531\text{ kips}$. Thus, $34''$ O-Cell is recommended with a capacity of 6000 kips in each direction (12000 kips of maximum total test capacity) is recommended for the 2nd load test.

5.3.3 Recommendations for the Construction and Instrumentation of the O-Cell load Tests

For each load test, it was recommended to use three (3) Geokon Model #4911 strain gages placed at one level shown in the drawing to assist in the determination of the side shear load transfer curves. Use three telltales to get compression data of the test shaft and measure any titling.

The construction recommendations described in the previous chapter for new load tests were employed in the Trinidad project. This included the specification of a contingent grouting procedure that would be employed if the test shafts turned to be wet during construction. A wet shaft is defined as a shaft filled with water. Also, the contribution of overburden to side resistance was removed by installation of a temporary casing to top of rock and keep this casing there until the test is complete. The concrete would then be placed to 1 ft below bottom of casing (see Part 2 of Appendix E). The contractor was asked to ensure that the casing and concrete are not mechanically connected.

It was requested that the contractor provides the project engineer with the following construction information of the test shafts.

- Times when excavation/drilling started and completed, and times when concreting started and completed.
- Methods/procedure for: excavation/drilling (e.g., auger), cleaning the base and sides of the borehole, and placement of the concrete.
- Were the hole sides and base cleaned before placement of the concrete? Was any water tipped into the borehole to aid in removal of cuttings? Were the sides of the rock socket wet or dry during drilling? 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). How much water was pumped out before placement of concrete?
- The slump of the fresh concrete, the unconfined compressive strength and stiffness of the concrete at time of load test.

- Information on the smoothness of the sides of the borehole in the bedrock rock including any estimates (even if rough) of the depth, width, and spacing of grooves. If possible, caliper the test pier borehole and obtain its roughness profile using laser devices or mechanical devices. At minimum, the depth of the deepest grooves should be obtained as this has a major influence on the side resistance of the shaft.

5.4. Construction of Test Shafts

Southern Test Shaft (for side resistance): drilling started on October 13, 2003 through the overburden but stopped shortly after that due to some construction problems. Drilling resumed on October 15 and the hole was stabilized with natural slurry. When the rock was encountered, the overburden was stabilized with a casing and the slurry was removed and the shaft in the rock was constructed using an auger under dry conditions thereafter. The shaft bottom was cleaned with 42-inch clean-out bucket. Then, the reinforcing cage and O-Cell assembly were inserted, concrete was placed with a tremie pipe, and then the casing was removed.

Northern Test Shaft (for end-bearing): Construction on October 9, 2003 proceeded as described above but the casing failed to seal the excavation from intrusion of water. Before the cage was placed, the water level in the shaft excavation was approximately 30 ft down from the ground surface. Note that the coring log for the North Load Test (see Appendix E, Part 1) suggests that the upper zone of the rock has a very low recovery ratio and RQD, indicating that the quality of the rock there is very poor (fractured). This fractured zone could provide an access to the overburden free water to seep through this poor rock into the test shaft hole. This fractured zone with low RQD was not noticed in the test hole near to the southern shaft and there was no problem with water seeping into that shaft holes. After base cleaning and placement of the reinforcing cage, grout was delivered into the base of the shaft through preinstalled PVC pipes. The wet grout extended past the O-Cell to about 1 ft above the cell, when concreting was begun. Several problems occurred during the concreting (tremie pipe was not available, slow and interrupted process of concrete placement) that may have led to low quality concrete placed in some parts of the Northern Test Shaft.

5.5 Load Testing Results and Analysis

Testing results from the two O-Cell load test results as reported by LOADTEST, Inc. are presented in Part 2 of Appendix E and briefly summarized in Table 5.1.

The measured unconfined compression test results (see Table 1 on page E-11) on rock core samples collected around and below the test shafts were carefully analyzed. An appropriate (on the high side) unconfined compressive, q_u , for the rock around the two test shafts was selected as 400 ksf and for the rock beneath the two test shafts was selected as 480 ksf.

5.5.1 Side Resistance:

Measured side resistance values at the end of the two load tests as reported by LOADTEST, Inc. (2003) are given in Table 5.1. As discussed in chapter 4, the average measured side resistance in the entire bedrock socket is more reliable than the measured side resistance values from strain gages. The average unit side resistance vs. the average side movement in the entire bedrock socket is shown in Figure 5.1. It is clear from Figure 5.1 that the ultimate true and peak unit side resistance, f_{max} , was reached in the South Load Test (26.4 ksf) at side movement of 0.75". There seems some limited brittle behavior passed the peak side resistance because of loss of pressure at that resistance that lessened the peak resistance to 23.5 ksf (movement of 1.5"). There was a good match between the results for side resistance from the two load tests, although there were many problems with the North Load Test (low quality concrete, wet hole). This suggests that the presence of water has minimal effect on the side resistance, as expected for very hard and competent bedrock shales.

Carter and Kulhawy recommended $f_{max} = 0.92 q_u^{0.51}$ for shafts with smooth bedrock socket, and $f_{max} = 2.05 q_u^{0.51}$ for shafts with intermediate roughness level. The results for the Franklin and Broadway test shafts suggested an intermediate roughness level for their bedrock sockets that was generated under normal drilling (Abu-Hejleh et. al. , 2003). Using these two equations with q_u of 400 ksf, f_{max} is predicted as 19.5 ksf for smooth socket and 43.5 ksf for intermediate roughened bedrock sockets. The measured f_{max} (26.4 ksf for peak and 23.5 ksf for residual) was

well below the expected value for intermediate roughened socket and a little bit larger than the resistance value for the smooth socket.

When side resistance vs. side movement for the Franklin, Broadway, SH 82 (Test # 2) and Trinidad load tests were placed in the same curve, there was a good match between them although the strength values of the rock significantly varies among these sites (74 ksf at Franklin, 145 ksf at Broadway, and 400 ksf for the Trinidad site). This observation could suggest that the side resistance in the three test shafts is generated because of the grooves created alongside the shaft holes during normal drilling. It is possible that these grooves were crushed during the shearing process in the Trinidad load test because the side movement developed in the Trinidad load test (1.5”) was much larger than those developed at other sites (< 0.5”, Franklin, Broadway, and SH 82- Test No. 2). This crushing may have caused the dropping of the side resistance to the residual side resistance of the bedrock socket, which is solely related to the strength of the bedrock and independent of the roughness conditions.

Table 5.1. Results of the Trinidad Load Tests as Reported by Loadtest, Inc. (2003)

Side Resistance Results (ksf)		Base Resistance Results (ksf)
Test # 1 (South Load Test)		
Top of Shaft to Strain Gage Level 1	28.08	256
Strain Gage Level 1 to O-Cell	24.89	
Entire bedrock Socket	26.4	
Test # 2 (North Load Test)		
Top of shaft to O-Cell	19.5	356
Top of Shaft to Strain Gage Level 1	15.5	
O-Cell to Strain Gage Level 1	25.4	
Entire bedrock Socket	20	

5.5.2 Base Resistance

Results of unit base resistance vs. settlement for the two test shafts are presented in Figure 5.2. These results suggest a linear relation between settlement and unit base resistance, suggesting that the measured response is far from the ultimate true unit base resistance. This behavior is similar to the response measured in the Broadway and SH 82 (Test #2) tests. The unit base resistance measured at the end of the 1st load test was 256 ksf and was 356 ksf for the 2nd load test.

It is not clear why we have the significant difference between results of the two load tests and the stiffer response of the 1st test (south shaft). It could be due to the variation in the strength of the bedrock beneath the test shafts or due to the presence of a fracture zone below the base of the Northern load test shaft. Note the presence of this fracture zone in the coring log at a depth around 50 feet and that the base of the northern shaft was placed at a depth of 48 feet. The variation in strength is not clear in the results of unconfined compressive strength performed on intact rock samples, but was clear in the results of the in-situ SPT, where N-value of 100/1” was measured under the South load test shaft and N-value of 100/3” was measured under the North load test shaft. Other possible reason for difference responses is the differences in the length of the bedrock socket length (12 ft for the 1st test and 20 ft in the 2nd test). Note that even much stiffer response was noticed in the SH 82 Test (Test # 2). The response from the 2nd test is more conservative, closer to the response of the Broadway test shaft, and extends over a wider range. Therefore, results from the Northern Load Test are recommended for design and research purposes.

Abu-Hejleh et. al. (2003) and FHWA defined the ultimate base resistance for very hard shales to correspond to a settlement of 5% the shaft diameter or 2.4 inches in this case. However, the load test was terminated at settlement of 1.2". The ultimate base resistance that corresponds to a settlement of 2.4" could be obtained from extrapolation (assuming linear response) as 700 ksf. But there are no data to support use of this very large value. Also, such large value will not be used in the design, even if measured. The Zhang and Einstein method, recommended by Abu-Hejleh et. al. (2003), predicts a $q_{max} = 21.4 (480)^{0.51} = 500$ ksf for bedrock with unconfined compressive strength of 480 ksf. For research purposes, it is reasonable to report the load test data and then suggest an ultimate unit base resistance of 500 ksf.

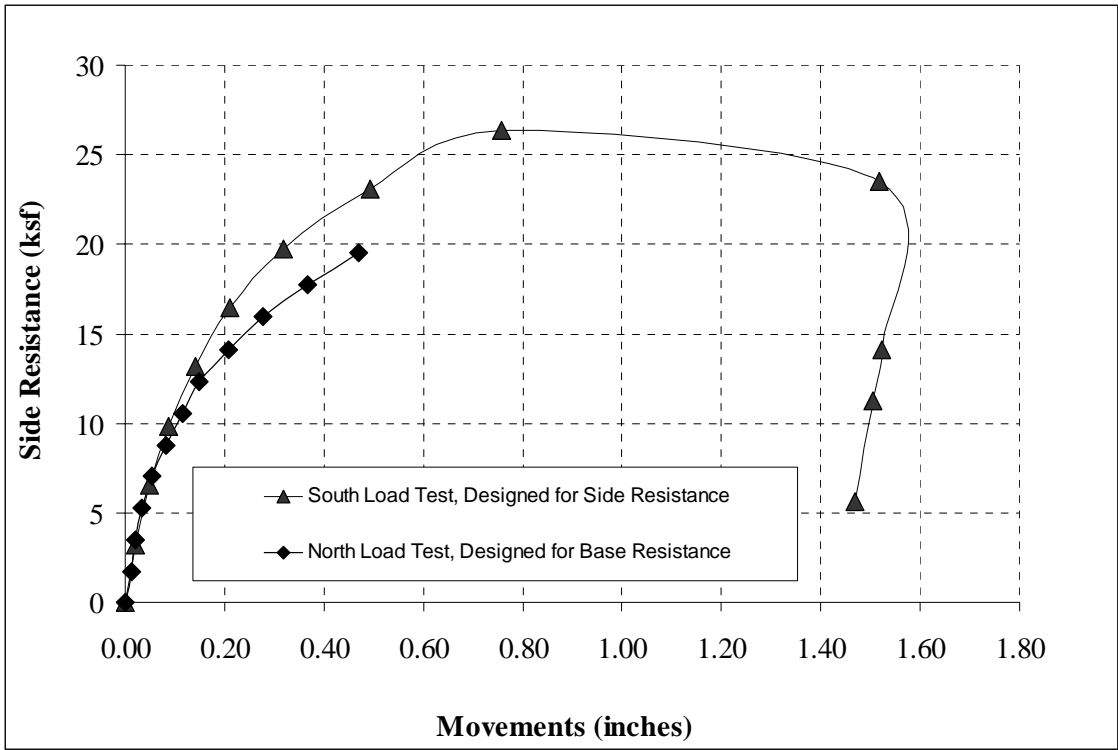


Figure 5.1. Unit Side Resistance vs. Upward Movement in the Bedrock Socket of the Trinidad Test Shafts.

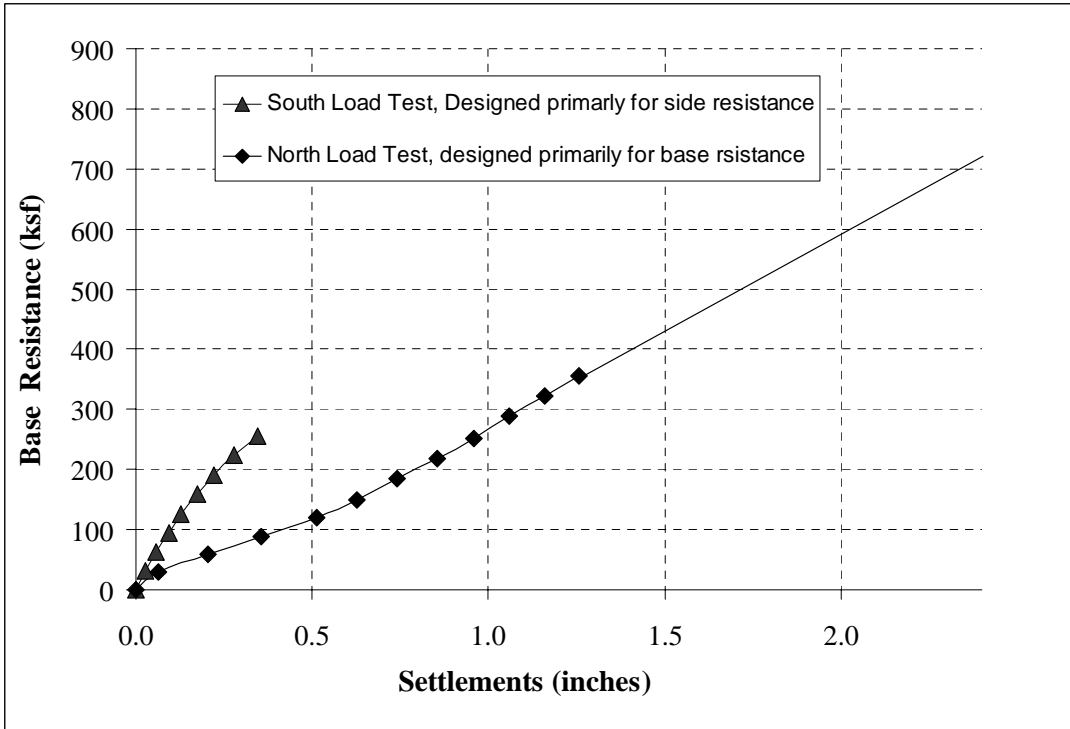


Figure 5.2. Unit Base Resistance vs. Settlement for the Trinidad Test Shafts.

5.6 Design Changes and Benefits Based on Load Test Results

Geotechnical design parameters before and after the load tests are performed are summarized in Table 5.2 for the geotechnical design of 4' shafts.

It was recommended in the design to use an ultimate side resistance of 20 ksf with no reduction due to wet hole conditions or lack of artificial roughening conditions. It is also suggested to use resistance factor of 0.8 in the LRFD method or FS of 2 in the ASD method. Justification is presented in the following:

1. There is a good match between results of f-w curves from the two load tests.
2. The 20 ksf value is a conservative value measured under wet and low quality concrete conditions in the 2nd load test. It is below the ultimate side resistance of almost 26 ksf measured in the 1st test.

For base resistance, it was recommended to use a very conservative value of 350 ksf (measured at the end of load test 2) for the ultimate base resistance and a resistance factor of 0.8 in the LRFD method. No reduction should be made even if the larger diameter shafts of 7 ft are employed.

Table 5.2. Geotechnical Design Parameters for the Trinidad's 4 feet Diameter Drilled Shafts before and after the O-Cell Load

		Before Load Tests (Based on Russ's Memo dated 3/7/03)	After Load Tests (Based on Naser's e-mail send 12/5/03)
Side Resistance	Ultimate Resistance (ksf)	4 to 8	20 ksf
	Resistance Factor	0.55	0.8
Base Resistance	Ultimate Resistance (ksf)	300	350
	Resistance Factor	0.5	0.8
Settlements of the Production Shafts		No information	0.27" Estimated @ working loads. And a top load- settlement curve for different shaft configurations.

A few number of the production shafts were 4 feet in diameter. The design calculations provided before conducting the O-Cell load indicated that the socket length for 4' shafts in the main span (with factored load of 4863 kips) is 59 ft. Using the geotechnical design parameters recommended based on results of load test, the required socket length for these shafts was calculated as 8 ft. Hence, the benefits of the load test could be dramatic if 4 ft diameter shafts were employed, reducing the length of the rock socket from 59 ft to 8 ft.

The designer selected 7-ft diameter for the design of most of the production shafts. The calculations showed that the base resistance load in this case will be huge that no side resistance will be needed from penetration of the shafts in the bedrock ($L=0$). As per the current practice of Colorado's structural engineers, minimum $L=3D$ (21 ft) was planned before conducting the load tests. The discussion on this issue in accordance with the recommendation of CDOT Research Report 2003-6 led to change in the current practice from using $L=3D$ (21 ft) to a value as required to just support the lateral load (L close to 14 ft). This saved the project around 7 feet of penetration on each shaft drilled in the Trinidad project.

According to Mr. Joe Garcia, the project engineer for the Trinidad Project, the savings to the Phase I of the project from conducting the load tests are \$113,400. More savings are expected in the future phases of the project.

6. SUMMARY AND CONCLUSIONS

6.1 Overview

Drilled shaft foundations embedded in weak rock formations (e.g., Denver blue claystone and sandstone) support a significant portion of bridges in Colorado. The type and general locations of Colorado's bridges are discussed in Section 2.2 of this report. Drilled shafts derive support by embedment in these weak rocks, typically found at relatively shallow depth in Colorado. The contribution of overburden to the drilled shaft axial capacity is often ignored. Thorough geotechnical design of a drilled shaft requires determination of a top load-settlement curve, q_{\max} (ultimate unit base resistance) of the rock layer beneath the shaft, f_{\max} (ultimate unit side resistance) of the rock layers around the shaft, the load factor and resistance factor (ϕ) in the LRFD (Load and Resistance Factor Design) method, and the factor of safety (FS) in the allowable stress design (ASD) method. The most accurate design method to estimate q_{\max} , f_{\max} , and settlements of drilled shafts is to conduct load tests on test shafts constructed as planned in the construction project. The load tests are expensive and therefore only considered in large projects. However, the very accurate design information obtained from the load tests could be used: 1) to design production shafts with more confidence (lowest FS or highest ϕ) and accuracy (leading to less conservative estimates of q_{\max} , f_{\max} , and settlements in most cases), resulting in significant savings to the project, and 2) as research data to improve the accuracy of simpler analytical design methods for drilled shafts that use data of simpler geotechnical tests like SPT, and/or UCT, and/or PMT).

Since January 1, 2000, it has been the policy of Colorado Department of Transportation (CDOT) to incorporate the new and more rational AASHTO LRFD method for the design of its structures, including drilled shafts. 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 Standard Penetration Test (SPT) and deviate from the AASHTO LRFD Design Methods. The margin of safety (or ϕ) and expected shaft settlement are unknown in these methods, both needed to implement the AASHTO LRFD design methods. To address all these needs and

shortcomings, the CDOT's strategic objectives for Colorado's drilled shafts socketed in weak rocks (Abu-Hejleh et. al., 2003) are to:

- Identify the most appropriate and accurate geotechnical design methods to predict the ultimate axial resistance and settlements of Colorado's drilled shafts socketed in weak rocks that are based on simple and routine geotechnical tests (SPT, UCT, and PMT).
- Identify the most appropriate resistance factors (ϕ) needed per the LRFD for the design methods identified in the 1st objective.

To fulfill these objectives, the measured resistance and settlement results of an adequate number of load tests on drilled shafts socketed in Colorado's shale bedrocks should be obtained and compared with predictions from design methods that use test data of simpler and more common geotechnical tests (SPT, UCT, and PMT) on the same shale bedrocks. CDOT Research Report 2003-6 (Abu-Hejleh et. al., 2003), titled "Improvement of the Geotechnical Axial Design Methodology for Colorado's Drilled Shafts Socketed in Weak Rocks," thoroughly documented and analyzed the results of four O-Cell load tests performed in 2002 as part of the T-REX and Broadway Viaduct projects. 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 subsurface geotechnical investigation was performed on the weak rock at the load test sites. This included 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 common geotechnical tests (SPT, UCT, and PMT), 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 common geotechnical tests, and 3) assessment of the CDOT and AASHTO design methods.

CDOT Research Report 2003-6 also outlined a long-term plan with six tasks to fulfill the strategic objectives listed above. This study was initiated to execute the following tasks in this plan:

- ❑ Compile and evaluate all available Colorado's past and reliable axial load test information.
- ❑ Determine CDOT future needs for performing new axial load tests on drilled shafts in CDOT future construction projects.

Several tasks were conducted in this study to address these objectives. A summary of the findings of these tasks are initially presented in this Chapter. Then, lessons learned from the study findings for future planning of load tests are furnished. Finally, CDOT's future needs for axial load tests on drilled shafts are listed in this Chapter.

6.2 Summary and Evaluation of Colorado's Records of Axial Load Tests on Drilled Shafts

There are relatively few historical load tests in Colorado that are of value to this study. All the acquired Colorado load test information is presented, discussed, and evaluated in Chapters 3, 5, and Appendices A, D, and E. The following information (if available) is presented for the load tests: construction, materials, and layout of the test shafts; geological and geotechnical description of the foundation bedrock around and below the test shafts including the results of SPT, UCT, and PMT; and results of the load tests. The compiled load tests (Table 3.1) are named after their location as: Fort Carson, 23rd Street Viaduct in Denver, I-270/I-76, SH82 Shale Bluffs in Pitkin County, T-REX along I-25 in Denver (I-225, County Line, and Franklin), Broadway Viaduct along I-25 in Denver, and Trinidad. Some reported information (e.g., results of load tests and of the geotechnical investigation) in the Testing Reports are furnished in Appendix D for the 23rd Street, I-270/I-76 and SH82 Shale load tests, and in Appendix E for the Trinidad load test. Load tests that have most of the information needed for analysis and evaluation are summarized in seven tables in Appendix A.

6.2.1 Types of Load Tests

With one exception (Fort Carson), the load tests were performed on CDOT projects. Historically, a challenge of load tests on drilled shafts has been that it has been very difficult and expensive to

obtain sufficient reaction to test full-scale drilled shafts with common production dimensions. As a result, most of the earlier drilled shaft load tests (all conventional load tests with load applied from top) were not full-scale (see Table 3.1 for diameter and length of the test shafts). Rather, the pier diameters were minimized and/or the length of the rock sockets shortened. Typically, those performing the tests attempted to run separate tests for end-bearing and side shear. As a result, most of the credible, earlier load tests were of limited capacity. For these and other reasons, it is difficult to extrapolate the measured test results to full-scale drilled shafts of the dimensions that typically support highway structures. Results from these tests should be considered with caution.

In recent years, a new technology has become available for performing load tests that overcomes some of the previous limitations. This is the Osterberg Cell (O-Cell). The most useful load tests have been the most recent. This is because the O-Cell test method was used, and because detailed subsurface information was obtained at each load test location (except the SH 82 O-Cell load tests.) The principle advantage of the O-Cell is that external reaction frames are not needed. Therefore, tests on full-scale production piers can be carried to very large capacities. The tests on SH-82 at Snowmass Canyon (Shale Bluffs), T-REX and Trinidad were performed using the O-Cell method. Most of these tests were carried to total loads well in excess of what could have been obtained using earlier reaction frame methods. According to the FHWA design manual (1999), the cost of an 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. An alternate load test method which may have some promise is the Statnamic test. However, the O-Cell results are likely to be more representative of the load/deflection relationships of production shafts. More specific details for conducting new load tests are presented in Section 4.3.4.

6.2.2 Discussion of Colorado's Past Load Tests

Trinidad, T-REX and Broadway O-Cell load tests. These are the most recent tests performed from 2002 to 2004. They have been planned well, documented, and analyzed in detail by the

CDOT Research Branch (Abu-Hejleh et. al., 2003 and Chapter 5). These tests should be considered for evaluation in the future with a rank of “*Very Good.*”

The 1970 Fort Carson tests. These are of little value to CDOT because they were not intended to replicate large, heavily loaded bridge foundation drilled shafts. The test objective stated that the tests were for small diameter, low capacity piers. This means that significant upward scaling is needed to make use of these data. Not only were the shafts small, they only extended a few feet below grade. Analysis of these data also requires information on SPT-N values and the unconfined compressive strength, both not provided. It is not believed that any useful improvement could be made to the test data by obtaining new geotechnical data at the load test site. *This test should not be among the selected load tests for evaluation in the future.*

The 23rd Street Viaduct and the I-270/I-76 load tests. These were performed for CDOT projects. Both sites have reasonable quality geotechnical data, including PMT for the I-270/I-76 load test sites. Both load test programs were performed using the same reaction beam and jacks. The reaction beam had a large, 1,000 ton capacity. Nevertheless, this capacity was not sufficient to test full-size production shafts to failure. Therefore, the shaft dimensions and embedment were limited. This factor should be considered for the analysis of the data from these load tests. With the use of shear rings for artificial roughening of the shaft holes in the bedrock, these tests suggested significant improvements to the side resistance. *The base and side resistance results from the I-270/I-76 load tests and just the side resistance (not base resistance) results from the 23rd Street load tests should be considered for evaluation in the future with a rank of “Good.”* These test results were rated “Good” not “Very Good” because dimensions and embedment of the test shafts are not as large as those of typical production shafts employed by CDOT.

The analysis of the 23rd Street load tests suggests that the strength reported in the test report for the Denver Blue shale bedrock below the test shafts may be low. Therefore, a better understanding of the strength of this shale should be obtained by new geotechnical data at the load test site. The test location is currently in a Coors Field parking lot. The Denver Metropolitan Major League Baseball Stadium District is the facility owner. The CDOT Research Branch made all the arrangements needed to perform subsurface geotechnical investigation at the load test

locations. However, the study panel recommended not conducting this investigation because bedrock conditions at that site have changed with time since the load test was performed and any investigation may add uncertainty to the results of the load test sites. In addition, costs for this geotechnical investigation work were significant: \$5000. *Based on this, the base resistance results from the 23rd Street load tests should be considered in the future with a rank of “fair”* due to: 1) uncertainty of the results from the geotechnical tests, and 2) dimension and embedment of the test shafts are not as large as those of typical production shafts employed by CDOT.

The SH-82 Shale Bluffs two load tests. There is very little useful geotechnical data in these two tests. The 1st load test is of little value for research purposes and additional geotechnical investigation at this test shaft location is not suggested for several reasons discussed in detail in Section 3.5.1. *This test should not be among the selected load tests for evaluation in the future.*

It is unfortunate that the 2nd load test (D= 3ft and L =29.7 ft) did not have the capacity to fail the shaft. It appears both the end-bearing and side shear components of the resistance were still in the linear range when the test was concluded at an upward movement of 0.07” and a downward movement of 0.09”. Two observations were noticed from the load test results:

- The trend of the unit side resistance vs. side movement data over 0.07” of movement measured in this test seems close to similar results reported for the Franklin, Broadway, and Trinidad load tests. The measured maximum unit side resistances at the end of the test were 4 ksf in the upper 13.4 ft of the bedrock socket, and 14.8 ksf in the lower 15 feet of bedrock socket, with an average value of 9.2 ksf in the entire bedrock socket.
- The maximum measured unit base resistance of 325 ksf at a settlement of 0.09” is a very high value when compared to all other load tests presented in this study, suggesting the rock strength at this site is much stronger/stiffer than the very massive bedrock shales encountered in the Broadway (q_u as high as 200 ksf) and Trinidad (q_u as high as 500 ksf) sites. It is possible that the rock strength at this site is close to or even exceeds the strength of the concrete.

Even though the test shaft did not fail, it is believed that much could be learned if material strength were available. It would be good to supplement the rock core recovery and RQD with strength data. For example, it could be determined if the excellent pier capacity would have been predicted based on the strength of the rock. Mr. Joseph Elsen from *Region 3* indicated that it is extremely difficult to access the load test site. Mr. Mark Vessely from CDOT Geotechnical Office has gone through the SH 82 records and has not located any relevant rock strength test results. It is also possible that the conditions of the rocks at that site may have changed with time since the load test was performed and any investigation may add uncertainty to the results of the load test sites. For all these reasons, it was decided not to conduct a subsurface geotechnical investigation at the load test site. *Only the two observations previously listed should be considered for evaluation in the future. We should be careful with accepting the measured base resistance value in this test.*

6.3 Lessons Learned for Planning Future Load Tests

This section summarizes the lessons learned for future planning of Colorado's axial load tests shale socketed in weak rocks from:

- Colorado's past load tests.
- Investigation on the construction methods and observations for Colorado's load test shafts.
- Investigation on the geology of Colorado's bedrock formations.

6.3.1 Lesson Learned from Colorado's Past Load tests

Type of Load test. The O-Cell load test should be considered in Colorado's future drilled shaft load tests until more cost-effective and innovative load test methods become available. It is also recommended to consider conventional load tests for low-capacity production shafts with ultimate capacity up to 1000 tons. The O-cell load test is more cost-effective than the conventional static load test for testing similar large-diameter shafts socketed in rock. Most of Colorado's good unit and side resistance data for Colorado's bedrock are generated from the O-Cell load tests. This test has been used in Colorado extensively over the last 7 years. Therefore, it

will be hard in the future to calibrate resistance data obtained from an innovative load test different than O-Cell. The O-Cell load test can mobilize the ultimate unit base resistance and side resistance in soft shale bedrocks (settlements larger than 2"), unit base resistance in harder shale bedrock over a settlement up to 3" (5% of the shaft diameter), and with a careful design the ultimate unit side resistance in the harder shale bedrock (drop of side resistance after reaching the peak was observed in the Trinidad load test).

Types of shale bedrock for future cost-effective load tests. Based on the results of the T-REX and Broadway load tests and the subsurface test data, CDOT Research Report 2003-6 evaluated the Colorado SPT-based (CSB) design method and the AASHTO/FHWA design methods. For conditions like those encountered in these load tests, the report proposed new design methods to predict the unit base and side resistances and settlements of Colorado's shafts embedded in soft to very hard to even much harder and massive shale bedrock based on results of routine geotechnical tests (SPT, UCT, and PMT). For the soft claystone ($20 \text{ bpf} < \text{SPT-N} < 100$), the proposed design method is based on the results of the SPT and is called the updated Colorado SPT-based (UCSB) design method. A factor of safety of 2 is recommended for the UCSB method.

In soft claystone like those encountered in the I-225 and County Line sites ($20 < \text{SPT-N} < 100$), the CSB and UCSB methods made reasonable predictions of end-bearing if a factor of safety of 2 is assumed. However, measured side resistance data were less than values predicted from the CSB method (FS ranged from 1.3 to 1.8) and agreed well with values predicted from the UCSB method. Analysis results of the Fort Carson, 23rd Street and the I-270/I-76 load tests, when no shear rings were employed for roughening the shaft holes (smooth shafts), also support the proposed UCSB method. No future load tests are recommended for the soft claystone with smooth shaft holes because the proposed UCSB method is: 1) the most accurate and economical since it employs the lowest possible factor of safety of 2; and 2) more conservative than the CSB method employed in Colorado for decades with overwhelming success using a factor of safety less than 2.

But with the use of artificial roughening (shear rings) in softer shale bedrock formations, the 23rd Street Viaduct and the I-270/I-76 load tests suggest that shear rings produce a measurable improvement in side shear capacity compared to drilled shafts of similar dimensions but drilled with no or minimal roughening. These load tests showed the benefits of artificial roughening to drilled shafts embedded in soft claystone shale bedrocks having SPT- N values larger than 50. This observation is very important, because many structure locations are in this kind of claystone bedrock. If shear rings were routinely used in these very weak formations, it may be possible to use greater side shear allowable design parameters. This would reduce drilled shaft dimensions with accompanying cost savings. Hence, new design methods (more load tests) for these types of weak rocks are needed when shear rings are employed.

In the harder bedrock shale formations like those encountered in the Broadway, Trinidad, SH-82 (Test No. 2), and the Franklin sites, the results of the O-Cell tests indicated that both end-bearing and side shear capacities were well above those predicted from the Colorado SPT-based (CSB) design methods (FS: 3.4 to 7). Hence, new and improved design methods for drilled shafts embedded in these bedrock shales will generate significant savings to CDOT. The side resistance vs. side movement was measured over a large movement (1.5”) only for the Trinidad load tests, where some limited brittle behavior was observed due to lessening of the peak resistance (occurred at movement of 0.75”). *This issue is worth investigating in future load tests.*

Based on the above, future drilled shaft load tests can be of much economical value in:

- Reducing drilled shaft embedment lengths in the harder bedrock shale formations because of higher allowable end-bearing and side resistance.
- Reducing drilled shaft embedment in softer bedrock shale formations because of the greater side resistance that may be available if shear rings are used.
- Reducing drilled shaft dimensions because of higher resistance factors with improved and more accurate design methods.

Different levels of roughness and dry and wet shaft hole conditions were encountered in the load tests. Learned lessons on these issues are covered in the next section.

6.3.2 Lessons Learned from the Investigation on Construction of Drilled Shafts

Scope of work: Colorado construction specifications and standards for drilled shafts are documented, construction information and observations of drilled shafts at the load test sites are presented and evaluated, and then recommendations for construction of production and test shafts are furnished.

Construction specifications and standards. Section 2.5.1 presents an overview of CDOT construction specifications and Colorado's methods for construction of drilled shafts. 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.

Construction information and observations at the load test sites. The construction methods for the test shafts employed in the T-REX and Broadway projects are described in detail in Section 2.5.2. These construction methods are representative of the typical construction procedure for production shafts employed in Colorado. The documented construction methods for the drilled shafts employed in the other load tests are also presented in Chapters 3 and 5, and Appendix A. Dry and wet shaft hole conditions are encountered in the load tests (discussed later). These are the roughness conditions experienced in the test shaft holes:

- Medium to rough holes were generated in the harder claystone and sandstone shale bedrock at the Franklin, Broadway, and Trinidad load test sites under normal drilling procedures using the auger and cutting teeth. It is expected that there will be no need for artificial roughening in these hard shale formations.
- In the soft claystone shale bedrocks, three levels of roughness were reported at the load test sites.
 - i. Smooth sides, no use of artificial roughening. Lowest values of unit side resistance for a given strength are reported in this case (the 23rd Street load tests and the I-270/I76 load tests).

- ii. Minimal roughening of the of the lower 10 feet of the bedrock socket by replacing the outer cutting teeth with a “roughening” tooth that extends about 1.7” from the sides of the auger. This simple and quick roughening procedure was performed in the I-225 and County Line shafts to remove any smear zone from the side of the shaft holes in order to allow the natural roughness to be effective. Values of unit side resistance for a given bedrock strength seem to be higher than those reported for the first case.
- iii. Artificial roughening with the use of side shear rings. One of the benefits of the two load tests performed in the early 1990s (23rd Street and I-270/I76 load tests) is the result that shear rings generate a measurable improvement in side shear capacity compared to drilled shafts of similar dimensions completely drilled without or with some roughening.

Recommendation to improve CDOT’s current practice for construction of production and test drilled shafts. Section 2.5.3 presents some additional recommendations to improve CDOT’s construction practice for drilled shafts (shaft cleaning, drilling and concrete placement, use of water, slurry, and casing, wet holes, and shaft roughness). For deep shafts cleaned following the standard procedure, but where a clean bottom cannot be verified, several alternatives are suggested. In order to prevent the bedrock socket of the shafts from being smooth or degrading with time, the following requirements (some in CDOT construction requirements) should be adhered to:

- Rapid and continuous drilling of the shaft hole with minimal use of water during drilling, followed by rapid and continuous placement of the fresh concrete.
- No use of drilling slurry or casing in the rock socket.
- Use of casing in the overburden when perched water is expected, and
- Quick removal of any water encountered in the rock socket.

Minimal artificial roughening (not rigorous with shear rings) as described before for all CDOT drilled shafts socketed in weak rocks is recommended if roughening under normal drilling is not observed. This roughening is simple, quick (in matter of few minutes), and easy to do, and it is recommended to consider if in doubt of roughening under normal drilling. The proposed roughening method shall be approved by the Project Engineer. The roughening requirement may be waived at the Engineer’s discretion and after consultation with the Geotechnical Engineer.

A wet shaft is defined as a shaft filled with water which occurs *when* water infiltrates into the shaft excavation at a rate higher than it can be pumped out. This condition was observed in rocks of high RQD (due to presence of fractures and joints in blocky rock) and when GWT (groundwater table) is located above the top level of the rock. Suggestions for placement of concrete in wet shaft holes are presented in Section 2.5.3. Exposure of certain bedrock shales to water for long periods, especially the softer claystone shale bedrock, could weaken its side resistance, so rapid drilling and placement of concrete are important factors in this case as discussed in Section 2.5.3. It should be remembered that the presence of natural water in the rocks (for example due to high ground water table) is accounted for in testing results of the lab strength tests, the in situ test (SPT and PMT), and the load tests. So no reduction should be made to any estimate of the side and base unit resistances or to measured strength of the rock due to the presence of natural water. The water problem may develop when the shale bedrock is subjected to excess water, for example, from perched water present in the overburden soil above fractured or weathered rocks that could seep in the shaft hole and influence the concrete to bedrock bond. This happened during the Trinidad load test (Section 5.4.4) and the 23rd Street load test with smooth shaft holes.

In the very hard shale bedrocks like those encountered in the Broadway and Trinidad sites, the load test results suggest that any influence of the water on load tests is minimal. Therefore, water is not expected to degrade what is called in this study hard shale bedrock. In this study, hard shale bedrock should be classified as rock-like (not soil-like) material (durable, sound, not sensitive to water, and has very small potential for creep) per Colorado Testing Procedure 26-90. The observations and findings of this study for hard shale bedrock are that large water infiltration in these types of rocks: will not cause caving, is due to presence of fractures and joints in blocky rocks, and will not degrade the rock so that deepening the hole may not be needed.

The depth of extra drilling due to wet hole conditions should be determined in consultation with the Geotechnical Engineer based on the estimated degradation of the sidewall materials on the exposure to the water. Load tests should be performed on wet and dry shaft hole conditions to investigate and quantify the influence of water for different types of shale bedrocks.

Additional recommendations for construction of test shafts: Most of the following recommendations are presented in Chapter 4 and were applied in the Trinidad load tests discussed in Chapter 5. Any developed design methods for drilled shafts based on load test results are applicable to other production shafts only if the construction methods employed in the production shafts are similar or better than those applied in the load test shafts. Therefore, future load test shafts in Colorado will be constructed with layouts, materials, and construction procedures like (or worse than) those employed for the production projects. Construction, materials, and layout information for the test shafts should be documented. Other recommendations are:

1. Method of drilling can influence the generated side resistance. Future load tests in Colorado *should be* drilled with augers having cutting teeth. This is the appropriate drilling method for the weak sedimentary rocks recommended (next section) in future Colorado load tests.
2. The load testing procedure for the test shafts should be described through revisions to Section 503 of CDOT construction specifications. Examples of these revisions when O-Cell load tests are employed are presented in Appendices B and C.
3. 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 (see Chapter 5 and Appendix E).
4. For the O-Cell load test, a contingent grouting procedure should be specified that would be employed if the test shafts are wet during construction. This procedure is described in Section 4.3.8. If the shaft is dry then no grouting is required.
5. Any available roughness conditions of the bedrock socket and the use of any artificial roughening tools should be documented. With minimal roughening, no inspection of the dimensions of grooves is needed herein (no measurements of depth and width of grooves), just witnessing that the contractor performs the work described herein. For inspection of roughening generated under normal drilling or under artificial roughening using shear rings, 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 mirrors and determine the depth of the deepest grooves.

6.3.3 Lessons Learned from the Investigation on Geology of Colorado's Bedrocks

Scope of Work: To list and describe Colorado's bedrock geological formations that are typically encountered during construction of drilled shafts, and for which the load test data are collected. Then, based on the results of this investigation, to: 1) select the type of geological formations that should be considered in future planning of load tests, and 2) determine if geology should be a factor in selecting the exact locations of future load tests.

Study findings: The geology of Colorado's highways is presented in Section 2.3 and the impact of geology on highway structure foundations is presented in Section 2.4. The greatest applicability of load test data is to major bridge structures. These are most commonly associated with rural interstate highways and urban controlled-access highway improvements. Much of the mileage of these types of highways is along the existing interstate highway grid, and in the Front Range Metropolitan areas. Tables 2.1 and 2.2 summarize the geological formations along Interstates I-25, I-70, and I-76, and along State Highway 50. SH-50 is listed because it is representative of many Colorado highways as it crosses the center of the State from Utah to Kansas, including valleys in the west, the Rocky Mountains, and the Eastern Plains. The feasibility of using drilled shafts in these formations is also presented in these tables. Table 3.1 lists the compiled Colorado axial load sites on drilled shafts and the names of their geological bedrock formations.

Tables 2.1 and 2.2 suggest that many of Colorado's highways alignments are in the Sedimentary Cretaceous and Tertiary Formations such as the Upper Dawson Arkose, Castle Rock Conglomerate, Denver, Arapahoe, & Lower Dawson Formations and the Laramie Formation, Fox Hills Sandstone, Pierre Shale, and Mancos Formations. Large parts of Colorado are underlain by the Pierre and Mancos Formations, the Pierre in the east half of the State, and the Mancos on the west part of the State (Tables 2.1 and 2.2). The most common formations are the Pierre, Mancos, Denver, and Dawson Formations and these cover the bedrock formations on which the load tests collected in this study were performed (see Table 3.1). These sedimentary 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. The sedimentary rocks encountered in the load test sites are highly variable with unconfined compressive strength ranging from 6 ksf (very stiff clay soils) to more than 500 ksf (Trinidad site). The variability of sedimentary rock requires that site-specific investigations be performed to assess local conditions and appropriate foundation types. The sedimentary rocks in Colorado can be called “weak” making them feasible for the use of drilled shaft foundation systems (see Tables 2.1 and 2.2).

Based on the above, sedimentary rock formations will most likely be encountered in drilled foundations supporting bridge structures in Colorado. In these formations, drilled shafts are capable of supporting large loads. The formations indicated as unlikely candidates for drilled shafts in Tables 2.1 and 2.2 are typically the very hard igneous and metamorphic rocks. Drilled shafts can be installed in these hard rocks but with high costs, making other foundation alternatives, such as shallow foundations or end-bearing H-piles, cheaper.

Rock type and strength are probably better delineators of engineering behavior than geologic formations, unless the depositional process or age of one formation is very different from another for the same rock type. It is believed that for Colorado’s weak sedimentary rocks, strength and consistency of the rock, not type of the geological formation, should be considered to estimate the capacity of the rock to support foundation loads, because:

- Similar geological depositional environments and age for many of Colorado’s sedimentary geological formations are likely be encountered in drilled foundations. Some of the most common geological formations in Colorado (Mancos, and Pierre) were deposited in the Cretaceous Sea (see Tables 2.1 and 2.2), so they are very similar in age, composition and character. The Denver and the Dawson have many similar characteristics. Thus, drilled shaft design parameters are often very similar in these formations.
- All the design methods recommended in AASHTO, FHWA, and the Canadian Foundation Design Manual are not in any way a function of the geological formation but are a function of the mass strength and type of the rock. Geotechnical engineers in Colorado, for routine projects, seek for easily (inexpensively) obtained parameters that are either direct or indirect measures of supporting material strength. At the simplest level, the Denver Method uses SPT N-Values as a rough indication of rock strength. Thus rock strength, usually

indirectly measured, is used to determine drilled shaft capacities, generally without regard to the geologic formations.

- Many of Colorado's load tests were performed on drilled shafts embedded in soft to very hard Pierre, Denver, Dawson, and Mancos bedrock formations (Table 3.1). The test results seem to indicate that strength and consistency, not geology, are the main parameters that control the capacity of the rock to resist external loads.

Based on the above, the study recommends not to limit Colorado's future load tests to any particular sedimentary geological formations because it is believed that drilled shaft load test results in one sedimentary geological formation in Colorado can readily be extrapolated to another sedimentary geological formation of similar in situ strength and type (claystone or sandstone). This conclusion will greatly facilitate applying the load test information obtained over greater areas of the state. It is also important to identify the type of the geological formation encountered at the load test site. It is probable that as a load test database is assembled, any influence of the bedrock geology on the capacity of the drilled shafts will be identified.

6.4 CDOT's Future Needs for Axial Load Tests on Drilled Shafts

Based on the lessons learned in this study and the recommendations of CDOT Research Report 2003-6, CDOT's future needs for axial load tests on drilled shafts are established: *Where, When, and How to perform future axial load tests on drilled shafts.* The recommended axial load testing program would generate net savings to the construction project (higher resistance values and lower factor of safety) in addition to providing research data for improvement of the design methodology for drilled shafts. After Colorado's design methods are improved based on sufficient number of load tests, additional loads tests may be performed in the future for just pure economical reasons.

6.4.1. Type, Location, and Number of Future Load Tests

The O-Cell should be considered in Colorado's future drilled shaft load tests until more cost-effective and innovative load test methods become available. It is also recommended to consider

conventional load tests for low-capacity production shafts with ultimate capacity up to 1000 tons. Colorado's future load tests should be performed on shafts embedded in weak sedimentary rocks with unconfined compressive strength (q_u) up to 500 ksf. The load tests should not be limited to any particular sedimentary geological formations because it is believed that drilled shaft load test results in one sedimentary formation can readily be extrapolated to another sedimentary formation of similar in situ strength and type. Future load tests in Colorado *should be* drilled with augers having cutting teeth. This is the appropriate drilling method for the weak sedimentary rocks recommended in future Colorado's load tests.

No future load tests are recommended for the soil-like sandstone shale bedrocks ($50 < \text{SPT-N} < 100$) as it is unlikely savings will be generated from improved design methods in these materials based on load tests. This type of weak rocks exists in Colorado but is not a typically encountered material. Denver traditional design methods or those recommended by the FHWA (see Abu-Hejleh, et. al, 2003) can be used for the design of drilled shafts embedded in this type of weak rocks.

No future load tests are recommended for the typical soft claystone shale bedrocks ($20 < \text{SPT-N} < 100$) with smooth shaft holes as those encountered at the I-225 and County Line sites because the UCSB design method recommended by Abu-Hejleh et. al (2003) for these materials is: 1) the most accurate and economical since it employs the lowest possible factor of safety of 2; and 2) more conservative than the CSB design method employed in Colorado for decades without failure using a factor of safety less than 2. Hence, future load tests in these soft shale bedrocks will generate no savings to the construction project and most likely will not lead to significant improvement in the accuracy of the recommended UCSB method.

Future Colorado load tests should be considered in the following three categories of sedimentary weak rocks (see Table 6.1):

- The firmer claystone shale bedrocks ($50 < \text{SPT-N} < 100$) *when* shear rings are employed during construction for artificial roughening of the shaft hole sides. In this case, side shear resistance may improve significantly as discussed before. A minimum of 7 new load test sites (one test per site) are recommended (2 for wet and 5 for dry shaft holes). Since the objective is to develop only accurate *side* resistance design methods (not base) for drilled shafts (dry or

wet), the main purpose of the load tests should be to determine (mobilize) the ultimate unit side resistance of the rock. Unit base resistance in this type of shale bedrock can be estimated from the current Colorado SPT-based (CSB) design method with a factor of safety of 2 or the UCSB method recommended in CDOT Research Report 2003-6 (presented in Section 3.2.2).

- ❑ Very hard claystone shale bedrocks with SPT-N value >120 bpf (or $> 50/5''$) and $q_u < 100$ ksf, and classified as rock-like material per Colorado Testing Procedure 26-90. This is a typical shale bedrock in Colorado encountered in the Franklin load test. The Franklin bedrock is a very hard, mostly thinly bedded, bluish gray, and sandy claystone bedrock with q_u ranging from 40 ksf to 90 ksf (average of 65 ksf) around the bedrock socket and around 41 ksf beneath the socket. 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. A minimum of 7 load test *sites* are recommended (2 for wet and 5 for dry shaft hole conditions). *Two load tests per site* should be performed: one mainly to obtain base resistance data and one mainly to obtain side resistance data. The load tests for side resistance should be designed to fully mobilize the side resistance over a settlement exceeding 1". The purpose of the load tests is to develop more accurate base and side resistance design methods of the drilled shafts in these materials when the shaft holes are either wet or dry. Once the design methods are improved for these materials (in the long-term not in the short term), long-term savings should be expected. It is expected that normal drilling using the auger and cutting teeth in this material will generate medium to rough holes.
- ❑ Very hard and massive shale bedrock with q_u less than 500 ksf, and SPT-N values >100 for granular-based rock, and $q_u > 100$ ksf for clay-based rock, and classified as rock-like material per Colorado Testing Procedure 26-90. This rock was experienced in the Broadway and Trinidad sites (see Chapter 5 for description). The Broadway bedrock is very hard, well-cemented (massive), bluish gray and clayey sandstone with claystone interbeds and q_u ranging from 97 ksf to 293 ksf (average of 145 ksf) around the bedrock socket and around 219 ksf beneath the socket. 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 ($83/6''$). A minimum of 5 load test *sites* are recommended (1 for wet and 4 for dry shaft hole conditions). *Two load*

tests per site should be performed: one mainly to obtain base resistance data and one mainly to obtain side resistance data. The load tests for side resistance should be designed to fully mobilize the side resistance over a settlement exceeding 1". The load tests will improve the accuracy of the design methods (reduce FS), and will result in higher allowable unit base and side resistance values for wet and dry shaft holes. The use of load tests in this case is expected to generate significant savings immediately to the construction project and in the long-term when the design methods are improved. It is expected that normal drilling using the auger and cutting teeth in this material will generate medium to rough holes.

Available load tests information on these three categories of weak rocks are identified and ranked in Table 6.1.

A wet shaft is defined as a shaft filled with water at the time of concrete placement, which occurs *when* water is infiltrating into the bedrock hole at a rate higher than it can be pumped out. Shafts are considered dry if water is pumped out and water depth is kept low when concrete placement started. A wet shaft hole condition was only encountered in one of the two Trinidad load tests, and this explains the relatively smaller number of wet load tests (only 2) required in each type of shale bedrock. The best scenario is to have wet and dry load tests side by side in the same site (as in the Trinidad load tests) in order to quantify the influence of water on the measured unit base and side resistances of the test shafts.

For inspection of roughening of shaft holes generated under normal drilling (expected for Categories II and III of rocks) and under artificial roughening (for Category I), 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 mirrors and determine the depth of the deepest grooves.

6.4.2. Guidelines for Conducting and Analyzing Colorado's Future Axial Load Tests

New load test data for the categories of weak rocks identified in the previous section can be obtained from other states' load tests. Guidelines for collecting appropriate load test data from outside Colorado are presented in CDOT Research Report 2003-6.

Table 6.1. Available Load Test Information for the Types of Shale Bedrock Recommended in Future Load Testing.

Test Name	Type of Shale Bedrock. Rank, any information needed in the future evaluation, and sources for the test information
	Type I. Firm Claystone shale bedrocks ($50 < \text{SPT-N} < 100$) where shear rings are employed during construction for artificial roughening. <i>Side resistance data are only needed.</i>
The 23 rd Street Viaduct Load Test	Dry. Rated "Good" because dimensions and embedment of the test shafts are not as those in the typical production shafts employed by CDOT. See Table A.6 and Chapter 3.
The I-270/I-76 load tests	Consider dry. Rated "Good" because dimensions and embedment of the test shafts are not as those in the typical production shafts employed by CDOT. The shear rings were not applied in the entire claystone layer. See Table A.7 and Chapter 3.
<u>Type II. Very hard sandy claystone bedrock shale with SPT-N value > 120 bpf (or $> 50/5$") and unconfined compressive strength (q_u) is less than 100 ksf, and classified as rock-like material per Colorado Testing Procedure 26-90.</u>	
Franklin	Consider dry. Rated "Very Good. See Table A.3 and Abu-Hejleh et. al. (2003)
Type III. Very hard and massive shale bedrock with q_u less than 500 ksf, and SPT-N values > 100 for granular-based rock, and $q_u > 100$ ksf for clay-based rock, and classified as rock-like material per Colorado Testing Procedure 26-90.	
Broadway Viaduct	Very Good. Dry. Table A.4, Chapter 3, and Abu-Hejleh et. al. (2003).
Trinidad	Very Good. 2 load tests: one dry and one wet. Table A.5 and Chapter 5.
Shale Bluffs, Test No. 2.	This test is not recommended for future evaluation but two lessons learned from this test should be considered. See Chapters 3 and 6.

However, the emphasis should be on acquiring data from new load tests performed in Colorado's future construction projects. Chapter 4 presents the following specific details for planning and conducting new load tests:

- I. Step-by-step procedure on when it is cost-effective to consider load tests as part of the subsurface geotechnical investigation in CDOT's future bridge construction projects during

different stages of the design phase. Load tests on drilled shafts are cost-effective if ALL the following conditions are met:

- 1) Projects with a large number of drilled shafts required to support large bridges and with total construction costs for all phases of the project exceeding \$10,000,000. The greatest number of large bridges that will be supported by drilled shafts will be associated with limited-access highway corridor improvement projects.
- 2) Penetration depth of the drilled shafts is controlled by axial load, not lateral load.
- 3) Presence of
 - a. Very hard claystone and/or sandstone shale bedrock with SPT-N value larger than 50/6" and confirmed to be rock-like geomaterial per Colorado Testing Procedure 26-90.
 - b. Firm claystone shale bedrock with SPT-N value larger than 50 and when artificial shear rings will be employed for roughening the sides of the shaft holes.
- 4) Net savings are expected based on cost-benefit analysis (described in Chapter 4).

Load tests should be performed early in the project timeline so that the results can be incorporated by the design team into the construction bid package and all the benefits from the load test can be realized. This is most feasible when the project/corridor geology and general subsurface conditions are obtained in the Environmental Assessment/Preliminary Engineering Phase of the project, and the drilled shaft load tests are performed during the FIR project phase. See Chapter 4 for more details.

II. Guidelines for planning, design, and construction of new load tests on drilled test shafts. This includes: 1) purposes and promotion of new load tests; 2) location and number of the load tests; 3) type of test shafts (production or sacrificial); 4) types, features, and costs of load tests; 5) geotechnical investigation around the test shaft; 6) design of the O-Cell load test; 7) construction and instrumentation of the test shafts; and 8) data collection at the load test site. The construction recommendations for test shafts were summarized in Section 6.3.2. Sample Guide Specifications for Osterberg Cell Load Testing of Drilled Shafts are presented in Appendix B. Revision of Section 503 of CDOT Standard Specifications to incorporate the Osterberg Cell Load Test in the Broadway construction project is presented in Appendix C.

III. Analysis procedure of Osterberg Cell (O-Cell) load test results. This includes: 1) analysis of O-Cell load test results to extract the load transfer curves; 2) definitions of tolerable settlement, ultimate unit base and side resistances; 3) construction of the equivalent top load-settlement curve from the results of the O-Cell test; and 4) construction of a simple equivalent top load-settlement curve from the results of simple geotechnical tests. The measured and analyzed results from four O-Cell load tests are thoroughly discussed in CDOT Research Report 2003-6. The analysis employed in that report to identify the proper design methods of drilled shafts can be used as a reference in analyzing the larger set of load tests that will be developed in Colorado in the future.

The comprehensive guidelines suggested in Chapter 4 for conducting new load tests, summarized in the previous subsection, were applied in the Trinidad project. Chapter 5 provides details of all the steps employed for the planning, design, construction, and analysis of the two Trinidad load tests. CDOT engineers should benefit from this example for conducting future load tests.

Once the assembly of a database with adequate number of load test results and other information is completed, the analysis procedure for load test results and the descriptions of different categories of weak rocks presented in this report should be revisited and modified if necessary.

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APPENDIX A. SUMMARY INFORMATION OF COLORADO'S LOAD TESTS ON DRILLED SHAFTS

Table A.1. Construction, Materials, Layout, Description of the Foundation Bedrock, and Results of all Tests for the I-225 Load 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 the Test Shaft						
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 Shaft						
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 A.2. Construction, Materials, Layout, Description of the Foundation Bedrock, and Results of all Tests for the County Line Load Test Shaft

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.						
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: 3193 psi			E_c: 500000 ksf	
Top Elevation of Ground 5886 ft		Depth from Ground to Top of Shaft, GWL, Top of Competent Rock, and Base of Shaft: 6, not encountered, 8, and 22 ft.			D= 4 ft	L = 14 ft (extends 0.5 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 at the northern margin of the Dawson Formation.						
Test Results For the Weak Rock around the Test Shaft						
Depth: from to (ft)	f _{max} (ksf)	f _{all} (ksf)	f _d (ksf)	SPT-N (bpf)	q _{ui} (ksf)	E _m (ksf)
Socket: 8 to 21.5	3.4	1.7	3	38	10.4	1800
Test Results For the Weak Rock Beneath the Test Shaft						
Depth below (ft)	q _{max} (ksf)	q _{all} (ksf)	q _d (ksf)	SPT-N (bpf)	q _{ui} (ksf)	E _m (ksf)
22	53	27	22	61	16.8	3200
Test Results For the Test Shaft						
Q _{max} = 1340 kips, Q _d = 876 kips, Q _{all} = 670 kips (76% from side resistance), w _{all} = 0.25"						

Table A.3. Construction, Materials, Layout, Description of the Foundation Bedrock, and Results of all Tests for the Franklin Load Test Shaft

O-Cell Load Test Date and Location: 1/11/02; Denver, along 2 nd pier column from south abutment beneath the west column of the Franklin Bridge over I-25.						
Excavation Method and Time: Auger drilling with use of slurry and casing only in the overburden; ~ 5 hours						
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.						
Concrete Placement Method and Time: Slowly by tremie pipe; 3 hours						
Concrete Slump: 8"		f'_c: 3410 psi			E_c: 530000 ksf	
Top Elevation of Ground Surface 5296 ft	Depth from Ground to Top of Shaft, GWL, Top of Competent Rock, and Base of Shaft: 0, 4, 4.5, and 25.3 ft.			D= 3.5 ft	L = 20.8 ft (extends 1.8 ft beneath O-Cell)	
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.						
Test Results For the Weak Rock around the Test Shaft						
Depth: from to (ft)	f _{max} (ksf)	f _{all} (ksf)	f _d (ksf)	SPT-N (bpf)	q _{ui} (ksf)	E _m (ksf)
Socket: 4.5 to 23.7	19	8.5	19	50/4"	64	11050
Test Results For the Weak Rock Beneath the Test Shaft						
Depth below (ft)	q _{max} (ksf)	q _{all} (ksf)	q _d (ksf)	SPT-N (bpf)	q _{ui} (ksf)	E _m (ksf)
25.3	236	118	71	50/5"	41	4700*
Test Results For the Test Shaft						
Q _{max} = 6612 kips, Q _d = 5024 kips, Q _{all} = 3306 kips (90 % from side resistance), w _{all} = 0.2".						

Table A.4. Construction, Materials, Layout, Description of the Foundation Bedrock, and Results of all Tests for the Broadway Load Test Shaft

O-Cell Load Test Date and Location: 1/10/02; Denver, Broadway Viaduct over I-25, along bent 6 (6 th bent from west abutment) beneath the center column.						
Excavation Method and Time: Auger Drilling use of slurry and casing only in the overburden, ~ 7 hours						
Conditions of Shaft Wall Sides and Bottom: Roughened-sided socket is expected with normal drilling procedure; dry hole.						
Concrete Placement Method and Time: Slowly by tremie pipe; 4 hours						
Concrete Slump: 7.5"		f'_c: 3936 psi			E_c: 580000 ksf	
Top Elevation of Ground Surface 5255 ft	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.			D= 4.5 ft	L = 30.1ft (extends 6.3 ft beneath O-Cell)	
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.						
Test Results For the Weak Rock around the Test Shaft						
Depth: From to (ft)	f _{max} (ksf)	f _{all} (ksf)	f _d (ksf)	SPT-N (bpf)	q _{ui} (ksf)	E _m (ksf)
20.8-30.8	17	8.5	15.9	50/3"	97	8900
30.8-40.8	35.1	17.5	35.1	**50/2"	210	23448
Socket: 17 to 40.8	24	12	24	**50/2.5"	145	15025
Test Results For the Weak Rock Beneath the Test Shaft						
Depth below (ft)	q _{max} (ksf)	q _{all} (ksf)	q _d (ksf)	SPT-N (bpf)	q _{ui} (ksf)	E _m (ksf)
47.1 ft	318	159	71	*83/6"	219	21900***
Test Results For the Test Shaft						
Q _{max} = 15276 kips, Q _d = 11362 kips, Q _{all} = 7638 kips (95 % from side resistance), w _{all} = 0.5".						

*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.

Table A.5. Construction, Materials, Layout, Description of the Foundation Bedrock, and Results of all Tests for the Two Trinidad Load Test Shafts

Load Test type, number, date, and Location. Two O-Cell load tests, one to measure side resistance with some information on base resistance (Test #1, South Load Test, October 15, 2003), and the 2 nd to measure end-bearing resistance with some information on side resistance (Test # 2, North Load Test, October 9, 2003). The tests were performed as a part of the I-25 Reconstruction Project, Trinidad, Colorado. For more details, see Chapter 5 and Appendix E.				
Excavation Method and Time, Conditions of Shaft Wall sides and Bottom, and Concrete Placement Method and Time: <u>South Load Shaft.</u> After drilling through the overburden to the upper surface of the claystone using slurry mixed in the hole, a temporary casing was installed and sealed into the shale bedrock. After removal of the slurry, the shaft in the rock was constructed using an auger under dry conditions. The shafts bottom was cleaned with 42-inch clean-out bucket. Concrete was placed with a tremie pipe, most likely at a high rate. <u>Northern Test Shaft.</u> Proceeded as described above but the casing failed to seal the excavation from intrusion of water. Before the cage was placed, the shaft hole in the bedrock socket was filled with water. After base cleaning and installation of the cage and O-Cell assembly, grout was pumped into the base of the shaft through preinstalled PVC pipes that extended past the O-Cell. The wet grout extended to about 1 ft above the cell when concreting was begun. Several problems occurred during the concreting (tremie pipe was not available, slow and interrupted process of concrete placement).				
Concrete Slump:		f'_c: 3540 psi for the North Test Shaft, 4880 psi for the South Shaft	E_c:	
Top Elevation of Ground	Depth from Ground to Top of Shaft, GWL, Top of Competent Rock, and Base of Shaft (all in ft) Test # 1. (South): 29.75, 11, 29, 41 Test # 2. (North): 28, 11, 26.5, 48	D= 48"	L = 1 st Test (10.35 ft above O-Cell and 0.9 ft below O-Cell. 2 nd Test: 19.2 ft above O-Cell and 0.8 ft below O-Cell	
Geotechnical and Geological Description of Weak Rock: Pierre shale, very well cemented, typical RQD from 85 to 100, fine grained, low to medium plastic, very hard, slightly moist, gray to dark gray in color.				
Test Results For the Weak Rock around the Test Shafts				
Test #	f_{max} (ksf), @ 0.01D or 0.48"	True f_{max} (ksf)	SPT-N (bpf)	q_u (ksf)
Test # 1 (South Test)	23	26.4@ 0.76"*	100/2.5"	400
Test # 2 (North Test)	20**		100/2.5"	400
* Resistance lessened passed the peak resistance; ** low quality concrete				
Test Results For the Weak Rock Beneath the Test Shafts				
Test #	q_{max} (ksf) @ end of test	q_{max} (ksf), @ 0.05D or 2.4"	SPT-N (bpf)	q_u (ksf)
Test # 1 (South Test)	256 @ 0.35"		100/1"	480
Test # 2 (North Test)	356@1.25"	>500	100/3"	480

Table A.6. Construction, Materials, Layout, Description of the Foundation Bedrock, and Results of all Tests for the three 23rd Load Test Shafts

Load Test type, number, date and Location. Conventional load test, three: 1 st for end-bearing, 2 nd for side resistance, and 3 rd for side resistance with shear rings. Performed around April of 1992. Near the NW corner of Wazee and Park Avenue. See Appendix D and Chapter 3 for more details.						
Excavation Method and Time: No information is provided but suspected to be auger drilling with no use of water, slurry, or casing.						
Conditions of Shaft Wall Sides and Bottom: 1 st test shaft: data suggests the shaft bottom was not thoroughly cleaned. 2 nd test shaft: perched water was present in the overburden soils that may have deteriorated the concrete to bedrock bond. During drilling of the rock, seepage was noticed in the bedrock fractures. 3 rd test shaft: no information.						
Concrete Placement Method and Time: No information.						
Concrete Slump:		f'_c:		E_c:		
Top Elevation of Ground 5179.45 ft	Depth from Ground to Top of Shaft, GWL, Top of Competent Rock, and Base of Shaft: See Appendix D			D= 2.6' (31")	L = 3.6 for the 1 st load test, 9.4" for the 2 nd test, and 9.5' for the 3 rd load test.	
Geotechnical and Geological Description of Weak Rock: Denver Formation. Blue claystone with occasional interbeds of sandstone and siltstone. RQD: 93 to 95%. N-values obtained with California Sampler ranges from 76 to 120. For analysis, recommended N- value equal, around, or even greater than 120 bpf. q _u ranged from 5 ksf to 25.2 ksf.						
Test Results For the Weak Rock around the Test Shafts						
Test # and Type	f _{max} (ksf), @ 0.25"	f _{max} (ksf), @ 0.01D or 0.31"	True f _{max} (ksf)	SPT-N (bpf)	q _u (ksf)	E _m (ksf)
Test with smooth sides	2.73	3.3	5.9	120	25.2	
Test with rough sides generated with shear rings	3.6	18.3	21.6			
Test Results For the Weak Rock Beneath the Test Shaft						
Test # and Type	q _{max} (ksf) @ 1"	q _{max} (ksf), @ 0.05D or 1.55"	True q _{max} (ksf)	SPT-N (bpf)	q _u (ksf)	E _m (ksf)
Test # 1, end-bearing	117.6	145	198 (@3")	Larger than 120	Larger than 25.2	

Table A.7 Construction, Materials, Layout, Description of the Foundation Bedrock, and Results of all Tests for the Two 1-270/I-76 Load Test Shafts

<p>Load Test type, number, date and Location. Two conventional load tests, one to measure end-bearing (Test 1), the other was a combination of side shear and end-bearing (Test 2). Performed late of 1992. The test site is located between SH 224 (south side) and Clear Creek, just north of the then existing I-270/I-76 interchange. See Chapter 3 and Appendix D for more details.</p>				
<p>Excavation Method and Time: No information on timing. Excavation for both shafts was performed using 30" helical auger. After drilling through the overburden to the upper surface of the claystone using slurry mixed in the hole, a temporary casing was installed and sealed into the claystone. Then, the slurry was removed and the remaining penetration of the drilled shafts in the claystone was drilled essentially dry.</p> <p>The 1st test shaft was inserted one foot in the weathered claystone layer and the 2nd test shaft was socketed 9-feet into bedrock (5 ft in the weathered claystone and 4 ft in the unweathered claystone). One shear ring was installed near the upper surface of the unweathered claystone and the remaining shear rings were installed in the weathered claystone.</p>				
<p>Conditions of Shaft Wall Sides and Bottom: A trace of water entered most of the shafts during drilling, with a maximum rate of 2 to 5 gpm in some shafts. Water depth was 2" or less in all shafts before placement of concrete.</p>				
<p>Concrete Placement Method and Time: No information on timing. The reinforcing steel was inserted before concrete placement, except for those shafts with higher inflows of water. In that case, the reinforcing steel was inserted thought he fresh concrete. The concrete was placed to about 2 feet below grade by directing a freefall down the center of the shaft, without striking the steel.</p>				
<p>Top Elevation of Ground ~5120 ft</p>	<p>Depth from Ground to Top of Shaft, GWL, Top of Competent Rock, and Base of Shaft See Appendix D</p>	<p>D= (30")</p>	<p>L = 1 ft for the 1st load test, 9 ft for the 2nd test.</p>	
<p>Geotechnical and Geological Description of Weak Rock: Denver Formation. Claystone bedrock, hard to very hard sandy and fractured.</p> <p>In the weathered claystone layer (1st 5 ft in the bedrock), the measured SPT N-value was 50 bpf and q_u is 10 ksf. In the unweathered claystone layer (extends 4 ft below the weathered layer), the measured SPT-N value was 100 bpf and $q_u= 23$ ksf.</p>				
<p>Test Results For the Weak Rock around the Test Shaft</p>				
<p>Test # and Type</p>	<p>True f_{max} (ksf)</p>	<p>SPT-N (bpf)</p>	<p>q_u (ksf)</p>	<p>E_m (ksf)</p>
<p>Test # 2 with shear rings in the upper 6 ft of the socket</p>	<p>12.4</p>	<p>72</p>	<p>16</p>	
<p>Test Results For the Weak Rock Beneath the Test Shafts</p>				
<p>Test # and Type</p>	<p>True q_{max} (ksf)</p>	<p>SPT-N (bpf)</p>	<p>q_u (ksf)</p>	<p>E_m (ksf)</p>
<p>Test # 1</p>	<p>47</p>	<p>50</p>	<p>10</p>	
<p>Test # 2</p>	<p>105</p>	<p>100</p>	<p>23</p>	

APPENDIX B: SAMPLE GUIDE SPECIFICATIONS FOR OSTERBERG CELL LOAD TESTING OF DRILLED SHAFTS (Prepared by LOADTEST, Inc)

1.0 Description

This work shall consist of furnishing all materials and labor necessary for conducting an Osterberg Cell (O-cell) Load Test and reporting the results. The Contractor will be required to supply material and labor as hereinafter specified and including prior to, during and after the load test. The drilled shaft used for the load test program will be instrumented by LOADTEST, Inc. (the Osterberg Cell supplier) or others, as approved by the Engineer. The Osterberg cell load test will be conducted by LOADTEST, Inc. or others, as approved by the Engineer, with the Contractor providing auxiliary equipment and services as detailed herein. The O-cell load test is a non-destructive test and is suitable for both dedicated test shafts and production test shafts. If the test shaft is constructed at a production shaft location (intended to carry structural service loads) it shall be left in a condition suitable for use as a production shaft in the finished structure.

2.0 Materials

The Contractor shall supply all materials required to install the Osterberg cell, conduct the load test, and remove the load test apparatus as required.

Osterberg Cell - The Contractor shall furnish one (1) or more Osterberg Cells as required for each load test, to be supplied by:

LOADTEST International, Inc.

World Headquarters

2631- D NW 41st Street
Gainesville, FL 32606

Phone **(800) 368-1138** or
 (352) 378-3717

Fax (352) 378-3934

The Osterberg cell(s) to be provided shall have a capacity of at least ____ kips in each direction and shall be equipped with all necessary hydraulic lines, fittings, pressure source, pressure gage and telltale devices.

Required materials include, but are not limited to the following:

1. Fresh, clean water from an approved source to be mixed with a water-soluble oil provided by LOADTEST, Inc., to form the hydraulic fluid used to pressurize the Osterberg Cell.

2. Materials sufficient to construct a stable reference beam system for monitoring movements of the shaft during testing. The system shall be supported at a minimum distance of 3 shaft diameters from the center of the test shaft to minimize disturbance of the reference system. A good quality, self-leveling surveyors level shall be provided to monitor the reference system.
3. Materials sufficient to construct a protected work area (including provisions such as a tent or shed for protection from inclement weather for the load test equipment and personnel) of size and type required by the Engineer and LOADTEST, Inc. In the case of cold weather, the protected work area shall be maintained at a temperature above 40° F in order to insure proper operation of the load testing equipment.
4. Stable electric power source, as required for lights, welding, instruments, etc.
5. Materials for carrier frame, steel bearing plates and/or other devices needed to attach O-cell to rebar cage, as required.

Materials supplied, which do not become a part of the finished structure become the responsibility of the Contractor at the conclusion of the load test and shall be removed from the job site.

3.0 Equipment

The Contractor shall supply equipment required to install the Osterberg cell, conduct the load test, and remove the load test apparatus as required. Required equipment includes but is not limited to:

1. Welding equipment, certified welding personnel and labor, as required, to assemble the test equipment under the supervision of LOADTEST, Inc. personnel, attach instrumentation to the Osterberg cell(s), and prepare the work area.
2. Equipment and labor to construct the steel reinforcing cage and/or placement frame including any steel bearing plates required for the test shaft.
3. Equipment and operators for handling the Osterberg cell, instrumentation and placement frame or steel reinforcing cage during the installation of the Osterberg cell and during the conduct of the test, including but not limited to a crane or other lifting device, manual labor, and hand tools as required by LOADTEST, Inc. and the Engineer.
4. Equipment and labor sufficient to erect the protected work area and reference beam system, to be constructed to the requirements of the Engineer and LOADTEST, Inc.
5. Air compressor (minimum 185 cfm, 100 psi) for pump operation during the load test.

4.0 Procedure

For the drilled shaft(s) selected for testing by the Engineer, the Contractor shall construct the drilled shaft using the approved shaft installation techniques until the drilled shaft excavation has been completed. This includes both dry and wet (slurry) methods.

The Osterberg Cell, hydraulic supply lines and other instruments will be assembled and made ready for installation under the direction of LOADTEST, Inc. and the Engineer, in a suitable area, adjacent to the test shaft, to be provided by the Contractor. When a steel reinforcing cage is required for the shaft, the Osterberg Cell assembly shall be welded to the bottom of the cage in conjunction with the construction of the cage. The plane of the bottom plate(s) of the O-cell(s) shall be set at right angles to the long axis of the cage. The Contractor shall use the utmost care in handling the test assembly so as not to damage the instrumentation during installation. The contractor shall limit the deflection of the cage to two (2) feet between pick points while lifting the cage from the horizontal position to vertical. The maximum spacing between pick points shall be 25 feet. The contractor shall provide support bracing, strong backs, etc. to maintain the deflection within the specified tolerance. The O-cell assembly must remain perpendicular to the long axis of the reinforcing cage throughout the lifting and installation process.

When the test shaft excavation has been completed, inspected and accepted by the Engineer, the O-cell assembly and the reinforcing steel may be installed. A seating layer of concrete or grout shall be placed by an approved method, in the base of the shaft to provide a level base and reaction for the O-cell. The preferred method is to install the O-cell assembly and deliver the seating layer using a pump line or tremmie pipe extending through the O-cell assembly to the base of the shaft. Depending on the configuration of the test assembly, it may be necessary to deliver the seating layer of concrete prior to installing the O-cell. In this case, the O-cell assembly shall be installed while the concrete or grout at the base is still fluid, under the direction of LOADTEST, Inc. and the Engineer. The Osterberg Cell should end up at least partially submerged and firmly seated into the base grout or concrete

After seating the Osterberg cell, the remainder of the drilled shaft shall be concreted in a manner similar to that specified for production shafts. However, if approved by the Engineer, the Contractor may use high early strength cement (Type III) in the mix to reduce the time between concreting and testing. At least four (4) concrete test cylinders, in addition to those specified elsewhere, shall be made from the concrete used in the test shaft, to be tested at the direction of LOADTEST, Inc. At least one of these test cylinders shall be tested prior to the load test and at least two cylinders shall be tested on the day of the load test.

During the load test, no casings may be vibrated into place in the foundation area near the load test. Drilling may not continue within a 100-foot radius of the test shaft. If test apparatus shows any interference due to construction activities outside of this perimeter, such activities shall cease immediately.

After the completion of the load test, and at the direction of the Engineer, the Contractor shall

remove any equipment, material, waste, etc. which are not to be a part of the finished structure. If the load test shaft is constructed at a production location and intended to carry service loads, the Contractor shall grout the interior of the Osterberg cell and annular space around the outside of the Osterberg cell using grouting techniques approved by the Engineer and LOADTEST, Inc.

5.0 Testing and Reporting

The load testing shall be performed by a qualified geotechnical engineer approved in advance by the Engineer. The geotechnical engineer must have a demonstrated knowledge of load testing procedures, and have performed at least 10 Osterberg cell load tests within the past two years.

The load testing shall be performed in general compliance with ASTM D-1143 (Quick Test Method). Initially the loads shall be applied in increments equaling 5% of the anticipated ultimate capacity of the test shaft. The magnitude of the load increments may be increased or decreased depending on actual test shaft capacity.

Direct movement indicator measurements should be made of the following: downward shaft end-bearing movement (min. of 2 indicators required), upward top-of-shaft movement (min. of 2 indicators required), shaft compression (min. of 2 indicators required). Total expansion of the O-cell may be measured and used to determine downward end bearing shaft movement.

Loads shall be applied at the prescribed intervals until the ultimate capacity of the shaft is reached in either end bearing or side shear, or until the maximum capacity or maximum stroke of the O-cell is reached, unless otherwise directed by the Engineer.

At each load increment, or decrement, movement indicators shall be read at 1.0, 2.0, 4.0 and 8.0 minute intervals while the load is held constant.

During unloading cycles the load decrement shall be such that at least 4 data points are acquired for the load versus movement curve. Additional cycles of loading and unloading using similar procedures may be required by the Engineer following the completion of the initial test cycle.

Dial gages, digital gages, or LVWDT's used to measure end bearing and side shear movement should have a minimum travel of 4 inches and be capable of being read to the nearest 0.001 inch division. End bearing movement may be alternately monitored using LVWDT's capable of measuring the expansion of the Osterberg Cell (6 inches). Dial gages, digital gages or LVWDT's used to measure shaft compression should have a minimum travel of 1 inch and be capable of being read to the nearest 0.0001 inch division.

Unless otherwise specified by the Engineer, the Contractor will supply eight (8) copies of a report of each load test, as prepared by LOADTEST, Inc. or others approved by the Engineer. An initial data report containing the load-movement curves and test data will be provided to the Engineer within ____ working days (*minimum 4 working days*) of the completion of load testing,

to allow evaluation of the test results. A final report on the load testing shall be submitted to the Engineer within ____ working days (*minimum 7 working days*) after completion of the load testing.

6. Post-Test Grouting Procedure for Test Shafts.

During the O-cell test the shaft separates on a horizontal plane at the base of the cell, creating an annular space between the upper section above the O-cell (side-shear section) from the lower section below the O-cell (end bearing section). The size of the annular space (and the size of the void within the O-cell itself) depends on the amount of expansion of the O-cell.

Once a production shaft has been tested, the engineer may want to reconnect the upper and lower shaft sections in order to be able to transfer service loads below the O-cell and into the base of the shaft. To do this, the interior of the O-cell (occupied by hydraulic fluid) and the annular space needs to be grouted.

6.1 Post-Test Grouting of Osterberg Cells

- a) The grout shall consist of Portland cement and water only, **NO SAND**.
- b) The grout shall be fluid and pumpable. An initial mix consisting of 4 to 6 gallons of water per 95-lb bag of cement is recommended. Adjust water to obtain desired consistency.
- c) The mixing shall be thorough to ensure that there are no lumps of dry cement. Pass the grout through a window screen mesh before pumping.
- d) Connect the grout pump outlet to one hydraulic line of the O-cell. Open the other line to allow hydraulic fluid to bleed.
- e) Pump the grout through the O-cell hydraulic line while collecting the effluent from the bleed line. Monitor characteristics of effluent material and stop pumping when it becomes equivalent to the grout being pumped.
- f) Take three samples of the grout for compression testing @ 28 days, if required.

Recommended pre-mixed amount of grout for grouting the O-cell:				
O-cell Diameter (Inches)	13	21	26	34
Grout Volume (Cubic Feet)	4	7	9	13

6.2 Post-Test Grouting of Annular Space around Osterberg Cells

- a) Prepare a fluid grout mix consisting of Portland cement and water only, **NO SAND**. The mixing procedures should be as outlined for grouting the O-cells. The quantity of grout should be at least three (3) times the theoretical volume required to fill the annular space and grout pipes.
- b) Pump water to “blow out” the grout pipes (minimum two provided on each shaft).
- c) Pump the fluid grout through one of the grout pipes until the grout is observed flowing from the second grout pipe or until 1.5 times the theoretical volume has been pumped.
- d) If no return of grout is observed from the second grout pipe, transfer the pump to the second pipe and pump grout through it until 1.5 times the theoretical volume has been pumped.
- e) If higher strength grout is deemed to be necessary, immediately proceed with pumping the higher strength grout (which may be a sand mix). The pumping procedures for this grout will be the same as described above for the initial cement-water grout. **The entire grouting operation must be completed before the set time for the initial grout has elapsed.**
- f) Take three (3) samples of each type of grout for compression testing @ 28 days.

Recommended pre-mix amount of grout for grouting of annular space:							
Shaft Diameter (Feet)	2	3	4	5	6	7	8
Grout Volume (Cubic feet)	25	30	40	50	65	80	100

7.0 Payment

The drilled shaft Osterberg Cell load tests shall be considered as any material, labor, equipment, etc. required above the requirements of drilled shaft installation. This item should include everything necessary to assemble, install, conduct and remove the drilled shaft load test, under the direction of the Engineer and LOADTEST, Inc. representatives. All costs associated with the normal production of the drilled shafts are measured and paid for elsewhere in the contract documents.

8.0 Basis of Payment

The complete and accepted "Drilled Shaft Osterberg Cell Load Test" shall be paid for at the contract price bid for "Drilled Shaft Osterberg Cell Load Test", each. This shall constitute full compensation for all costs incurred during the procurement, installation, conducting of the test, and subsequent removal of test apparatus and appurtenances.

Payments shall be made under:

Pay item:

Drilled Shaft Osterberg Cell Load Test

Pay unit:

Each

Ref: Sample Guide Specifications
Revised 10/10/02

**APPENDIX C: REVISIONS OF SECTION 503, OSTERBERG CELL LOAD
TEST (from Broadway's Construction Plans and Specifications Project)**

REVISION OF SECTION 503
OSTERBERG CELL LOAD TEST

Section 503 of the Standard Specifications is hereby revised for this project as follows:

Replace paragraph 1 of Subsection 503.07 with the following:

For any portion of the shaft socketed in claystone or sandstone, if the concrete is not placed the same day the drilling of the socket occurs, the Contractor shall increase the socket 1/3 of the specified socket prior to placing the concrete. The reinforcing cage shall extend to the new tip elevation.

Add Subsection 503.10, OSTERBERG CELL LOAD TEST, as follows:

(a) DESCRIPTION

This work shall consist of furnishing all materials and labor necessary for conducting Osterberg Cell (O-cell) Load Tests and reporting the results. The Contractor will be required to supply material and labor as hereinafter specified and including prior to, during and after the load tests. The drilled shafts used for the load test program will be instrumented by LOADTEST, Inc. (the Osterberg Cell supplier) or others, as approved by the Engineer. The Osterberg cell load test will be conducted by LOADTEST, Inc. or others, as approved by the Engineer, with the Contractor providing auxiliary equipment and services as detailed herein. The load test is a non-destructive test and the test shafts shall be left in a condition suitable for use as a production shaft in the finished structure.

Two (2) drilled shafts shall be tested as a part of this contract. One drilled shaft shall be located at Phase 2, Pier 6 (either shaft) and one shaft shall be located at Phase 2, Pier 8 (either shaft). These shafts shall be installed with a 28' socket, which results in an estimated ultimate resistance of 6350 kips. Upon completion of the tests these shafts shall be incorporated into the permanent structure with a required axial load capacity of 3100 kips (Resistance factors (ϕ) x Ultimate side and base resistances), with resistance factors and ultimate base and tip resistances as shown in the plans or as determined from the load tests.

(b) MATERIALS

The Contractor shall supply all materials required to install the Osterberg cells, conduct the load tests, and remove the load test apparatus as required.

1. Osterberg Cell - The Contractor shall furnish two (2) Osterberg Cells (one for each load test) to be supplied by:

-2-

REVISION OF SECTION 503
OSTERBERG CELL LOAD TEST

LOADTEST, Inc.
2631- D NW 41st Street
Gainesville, FL 32606
Phone (800) 368-1138 or
(352) 378-3717
Fax (352) 378-3934

LOADTEST, Inc.
5420 S. Klee Mill Road, Suite 4
Sykesville, MD 21784
Phone (800) 436-2355 or
(410) 552-1979
Fax (410) 552-1843

The Osterberg cell(s) to be provided shall have a capacity of at least 2200 tons in each direction and shall be equipped with all necessary hydraulic lines, fittings, pressure source, pressure gage and telltale devices.

2. Materials required include, but are not limited to, the following:
 - A. Fresh clean water from an approved source to mix with a water-soluble oil provided by LOADTEST, Inc., to form the hydraulic fluid used to pressurize the Osterberg Cell.
 - B. Materials sufficient to construct a stable reference beam system for monitoring movements of the shaft during testing, supported at a minimum distance of 3 shaft diameters from the center of the test shaft to prevent disturbance of the reference system. A good quality self-leveling, surveyors level shall be provided to monitor the reference system.
 - C. Materials sufficient to construct a protected work area (including provisions such as a tent or shed for protection from inclement weather for the load test equipment and personnel) of size and type required by the Engineer and LOADTEST, Inc.
 - D. Electric power, as required for lights, welding, instruments, etc.
 - E. Materials for carrier frame, steel bearing plates and/or other devices needed to adapt O-cell to reinforcing cage, as required.
 - F. Approved pumpable grout consisting of Portland cement and water only, no sand. The 28 day compressive strength shall be a minimum of 3000 psi.

-3-

REVISION OF SECTION 503
OSTERBERG CELL LOAD TEST

3. Materials supplied which do not become a part of the finished structure become the responsibility of the Contractor at the conclusion of the load test and shall be removed from the job site.

(c) EQUIPMENT

The Contractor shall supply equipment required to install the Osterberg cell, conduct the load test, and remove the load test apparatus as required. Equipment required includes but is not limited to:

1. Welding equipment, certified welding personnel and labor, as required, to assemble the test equipment under the supervision of LOADTEST, Inc. personnel, attach hydraulic fittings and telltales to the Osterberg cell(s), and prepare the work area.
2. Equipment and labor to construct the reinforcing steel cage and/or placement frame including any steel plates required for the test shaft.
3. Equipment and operators for handling the Osterberg cell, instrumentation and placement frame or reinforcing steel cage during the installation of the Osterberg cell and during the conduct of the test, including but not limited to a crane or other lifting device, manual labor, and hand tools as required by LOADTEST, Inc. and the Engineer.
4. Equipment and labor sufficient to erect the protected work area and reference beam system, to be constructed to the requirements of the Engineer and LOADTEST, Inc.
5. Air compressor (minimum 100 cfm) for pump operation during load testing.
6. Approved grout pump and mixer with experienced operator for the placing of grout below the cell during installation and for grouting the cell, the annular space around the cell and hydraulic lines upon completion of the test.

-4-

REVISION OF SECTION 503
OSTERBERG CELL LOAD TEST

(d) PROCEDURE

1. For the drilled shaft(s) selected for testing, the Contractor shall construct the drilled shaft using the approved shaft installation techniques, as contained in the Standard Specifications, until the drilled shaft excavation has been completed.
2. The reinforcing cage, Osterberg Cell, hydraulic supply lines and other attachments will be assembled and made ready for installation under the direction of LOADTEST, Inc. and the Engineer, in a suitable area, adjacent to the test shaft, to be provided by the Contractor. The Osterberg Cell assembly shall be welded to the bottom of the cage in conjunction with the construction of the cage. The plane of the bottom plate(s) of the O-cell(s) shall be set at right angles to the long axis of the cage
3. When the test shaft excavation has been constructed, inspected and accepted by the Engineer, a seating layer of concrete or grout shall be placed, by an approved method as contained in the Standard Specifications, in the base of the shaft. The Contractor shall then install the Osterberg Cell, reinforcing steel cage assembly in the test shaft (while the concrete or grout is still fluid) under the direction of LOADTEST, Inc. and the Engineer so that the Osterberg Cell is resting firmly in the concrete. The Contractor shall use the utmost care in handling the placement/test equipment assembly so as not to damage the instrumentation during installation. The contractor shall limit the deflection of the cage to two (2) feet between pick points while lifting the cage from the horizontal position to vertical. The maximum spacing between pick points shall be 25 feet. The contractor shall provide support bracing, strong backs, etc. to maintain the deflection within the specified tolerance.
4. After seating the Osterberg cell, the drilled shaft shall be concreted by an approved method as contained in the Standard Specifications and similar to that utilized for typical production shafts. At least eight (8) concrete compression tests cylinders shall be made from the concrete used in the test shaft. At least one of these test cylinders shall be tested prior to the load test and at least two cylinders shall be tested on the day of the load test.

-5-

REVISION OF SECTION 503
OSTERBERG CELL LOAD TEST

5. During the period required to perform the load test, no casings may be vibrated into place in the foundation area near the load test. Drilling, may continue provided, however that it be on shafts approximately 50 feet clear from the work area. If test apparatus shows any signs of negative effects due to construction activities, such activities shall cease immediately.
 6. After the completion of the load test, and at the direction of the Engineer, the Contractor shall remove any equipment, material, waste, etc. which are not to be a part of the finished structure. The Contractor shall grout the interior of the Osterberg cell and annular space around the outside of the Osterberg cell using grouting techniques approved by the Engineer and LOADTEST, Inc. and prepare the shaft for incorporation into the permanent structure as directed by the Engineer. Steps to prepare the shaft for incorporation into the permanent structure include, but are not limited to:
 - A. Removal of hydraulic lines and other test apparatus flush with the top of shaft.
 - B. Removal of oil, debris and other deleterious materials from the top of the shaft.
- (e) POST-TEST GROUTING PROCEDURES FOR DRILLED SHAFTS TESTED WITH AN OSTERBERG CELL

During the O-cell test the shaft breaks, on a horizontal plane, separating the upper section above the O-cell (side-shear section) from the lower section below the O-cell (end bearing section.) This creates an annular space, the size of which depends on the amount of expansion of the O-cell.

The contractor will be required to grout the O-cell and the annular space around the O-cell in order to reconnect upper and lower shaft sections.

-6-

REVISION OF SECTION 503
OSTERBERG CELL LOAD TEST

1. POST-TEST GROUTING OF OSTERBERG CELLS (O-CELLS)

- A. The grout shall consist of Portland cement and water only, NO SAND.
- B. The grout shall be fluid and pumpable. An initial mix consisting of 4 to 6 gallons of water per 95-lb bag of cement is recommended. Adjust water to obtain desired consistency.
- C. The mixing shall be thorough to ensure that there are no lumps of dry cement. Pass the grout through a window screen mesh before pumping.
- D. Connect the grout pump outlet to one hydraulic line of the O-cell. Open the other line to allow hydraulic fluid to bleed.
- E. Pump the grout through the O-cell hydraulic line while collecting the effluent from the bleed line. Monitor characteristics of effluent material and when it becomes equivalent to the grout being pumped, stop pumping.
- F. Take three samples of the grout for compression testing @ 28 days.

The minimum amount of pre-mixed grout for grouting the O-Cell shall be as specified by LOADTEST, Inc.

2. POST-TEST GROUTING OF ANNULAR SPACE AROUND OSTERBERG CELLS (O-CELLS)

- A. Prepare a fluid grout mix consisting of Portland cement and water only, NO SAND. The mixing procedures should be as outlined for grouting the O-cells. The quantity of grout should be at least three (3) times the theoretical volume required to fill the annular space and grout pipes.
- B. Pump water to "blow out" the bottom caps of the provided plastic grout lines (two on each shaft).
- C. Pump the fluid grout through one of the PVC pipes until the grout is observed flowing from the second grout pipe or until 1.5 times the theoretical volume has been pumped.

-7-

REVISION OF SECTION 503
OSTERBERG CELL LOAD TEST

- D. If no return of grout is observed from the second grout pipe, transfer the pump to the second pipe and pump grout through it until 1.5 times the theoretical volume has been pumped.
- E. Take three (3) samples of each type of grout for compression testing @ 28 days.

The minimum pre-mix amount of grout for grouting the annular space shall be 50 cubic feet.

(f) TESTING AND REPORTING

The load testing shall be performed by a qualified geotechnical engineer approved in advance by the Engineer. The geotechnical engineer must have a demonstrated knowledge of load testing procedures, (and have performed an Osterberg cell load test within the past four years).

The load testing shall be performed in general compliance with ASTM D-1143 (Quick Test Method). Initially the loads shall be applied in increments equaling 5% of the anticipated ultimate capacity of the test shaft. The magnitude of the load increments may be increased or decreased depending on actual test shaft capacity.

Direct movement indicator measurements should be made of the following: downward shaft end-bearing movement (min. of 2 indicators required), upward top-of-shaft movement (min. of 2 indicators required), shaft compression (min. of 2 indicators required). Total expansion of the O-cell shall be measured to determine downward end bearing shaft movement.

Loads shall be applied at the prescribed intervals until the ultimate capacity of the shaft is reached in either end bearing or side shear, or until the maximum capacity or maximum stroke of the O-cell is reached, unless otherwise directed by the Engineer.

-8-

**REVISION OF SECTION 503
OSTERBERG CELL LOAD TEST**

At each load increment, or decrement, movement indicators shall be read at 1.0, 2.0 and 4.0 minute intervals while the load is held constant.

During unloading cycles the load decrement shall be such that at least 4 data points are acquired for the load versus movement curve. Additional cycles of loading and unloading using similar procedures may be required by the Engineer following the completion of the initial test cycle.

Dial gages, digital gages, or LVWDT's used to measure end bearing and side shear movement should have a minimum travel of 4 inches and be capable of being read to the nearest 0.001 inch division. End bearing movement may be alternately monitored using LVWDT's capable of measuring the expansion of the Osterberg Cell (6 inches). Dial gages, digital gages or LVWDT's used to measure shaft compression should have a minimum travel of 1 inch and be capable of being read to the nearest 0.0001 inch division.

Unless otherwise specified by the Engineer, the Contractor will supply eight (8) copies of a report of each load test, as prepared by LOADTEST, Inc. or others approved by the Engineer. An initial data report containing the load-movement curves and test data will be provided to the Engineer within 7 calendar days of the completion of load testing, to allow evaluation of the test results. A final report on the load testing shall be submitted to the Engineer within 31 weeks after completion of all load testing on site.

(g) PAYMENT FOR REVISED SHAFT LENGTHS

The Engineer, to reflect the results of the load tests, may revise the length of any drilled shafts not having been installed upon completion of the load tests and review of the initial report. Payment for any revision to the plan lengths shall be made as follows:

1. For any shaft whose length is increased from the length shown in the Plans, The revised length shall be paid for in full based upon the Contractors unit prices (LF) for Pay Item 503-00054, Drilled Caisson (54 inch) or for Pay Item 503-00030, Drilled Caisson (30 inch).
2. For any shaft whose length is decreased from the length shown in the Plans, payment will be made for the average of the Plan length and the revised length based upon the Contractors unit prices (LF) for Pay Item 503-00054, Drilled Caisson (54 inch) or for Pay Item 503-00030, Drilled Caisson (30 inch).

-9-

REVISION OF SECTION 503
OSTERBERG CELL LOAD TEST

(h) METHOD OF MEASUREMENT

The drilled shaft Osterberg Cell load tests shall be considered as all material, labor, equipment, etc. required above the requirements of drilled shaft installation. This item should include everything necessary to assemble, install, conduct, report the results and remove the drilled shaft load test, under the direction of the Engineer and LOADTEST, Inc. representatives. All costs associated with the normal production of the drilled shafts are measured and paid for elsewhere in the contract documents.

(i) BASIS OF PAYMENT

The complete and accepted "Drilled Shaft Osterberg Cell Load Test" shall be paid for at the contract price bid for "Drilled Shaft Osterberg Cell Load Test", each. This shall constitute full compensation for all costs incurred during the procurement, installation, conducting of the test, reporting the results, and subsequent removal of test apparatus and appurtenances.

Payments shall be made under:

<u>Pay item</u>	<u>Pay Unit</u>
Caisson Load Test	Each

APPENDIX D: INFORMATION FROM PAST COLORADO LOAD TESTS (from Load Test Reports listed in the References)

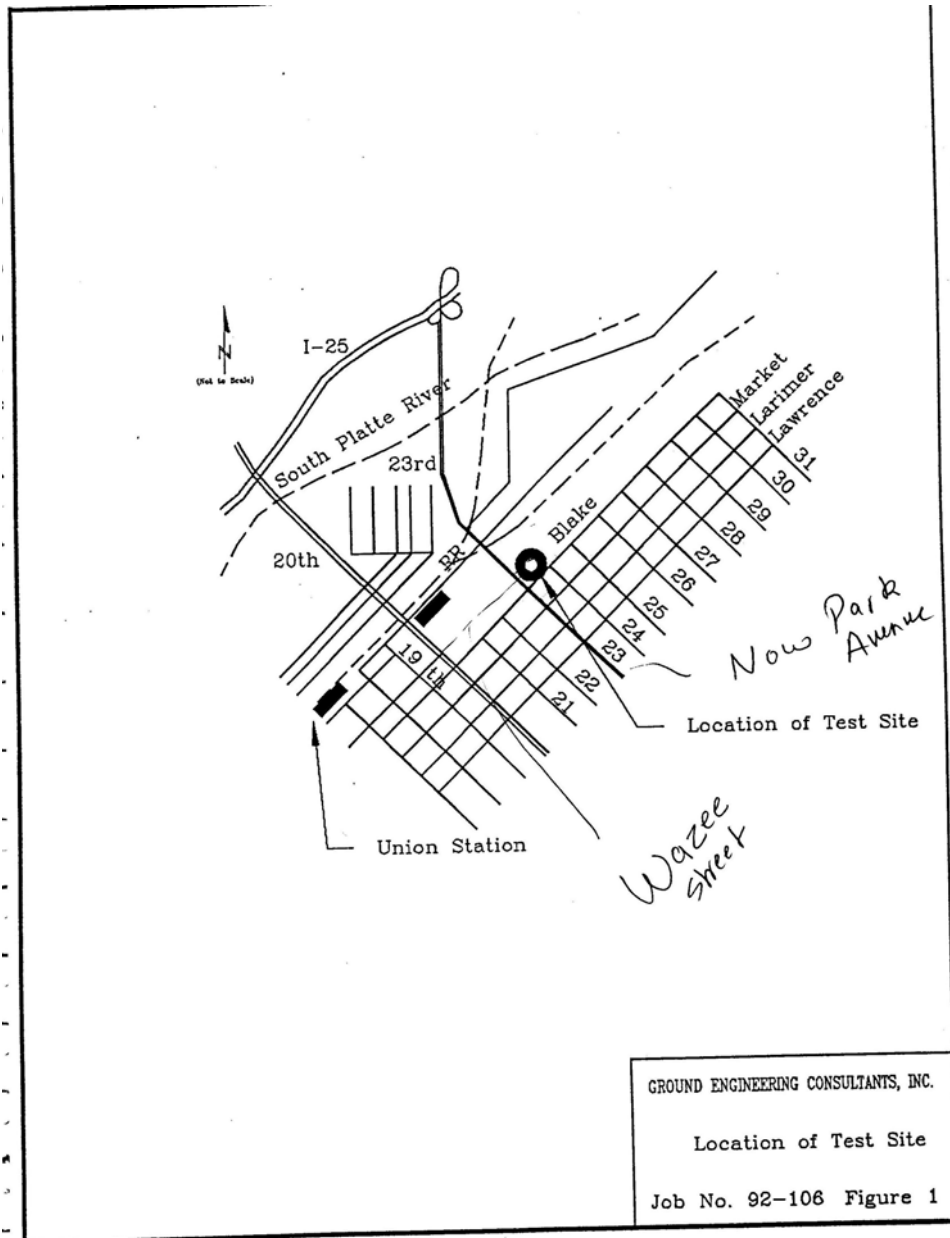
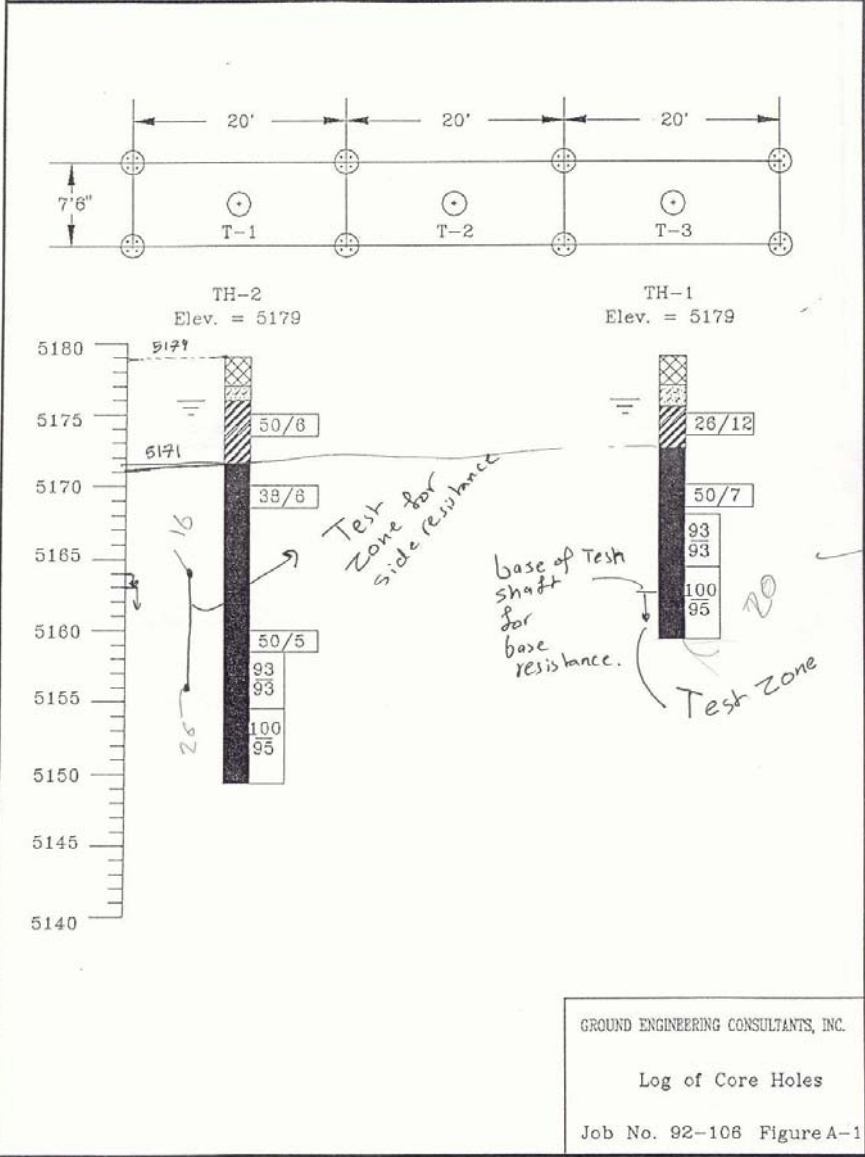


Figure D.1. The 23rd Street Viaduct Load Test: Location.



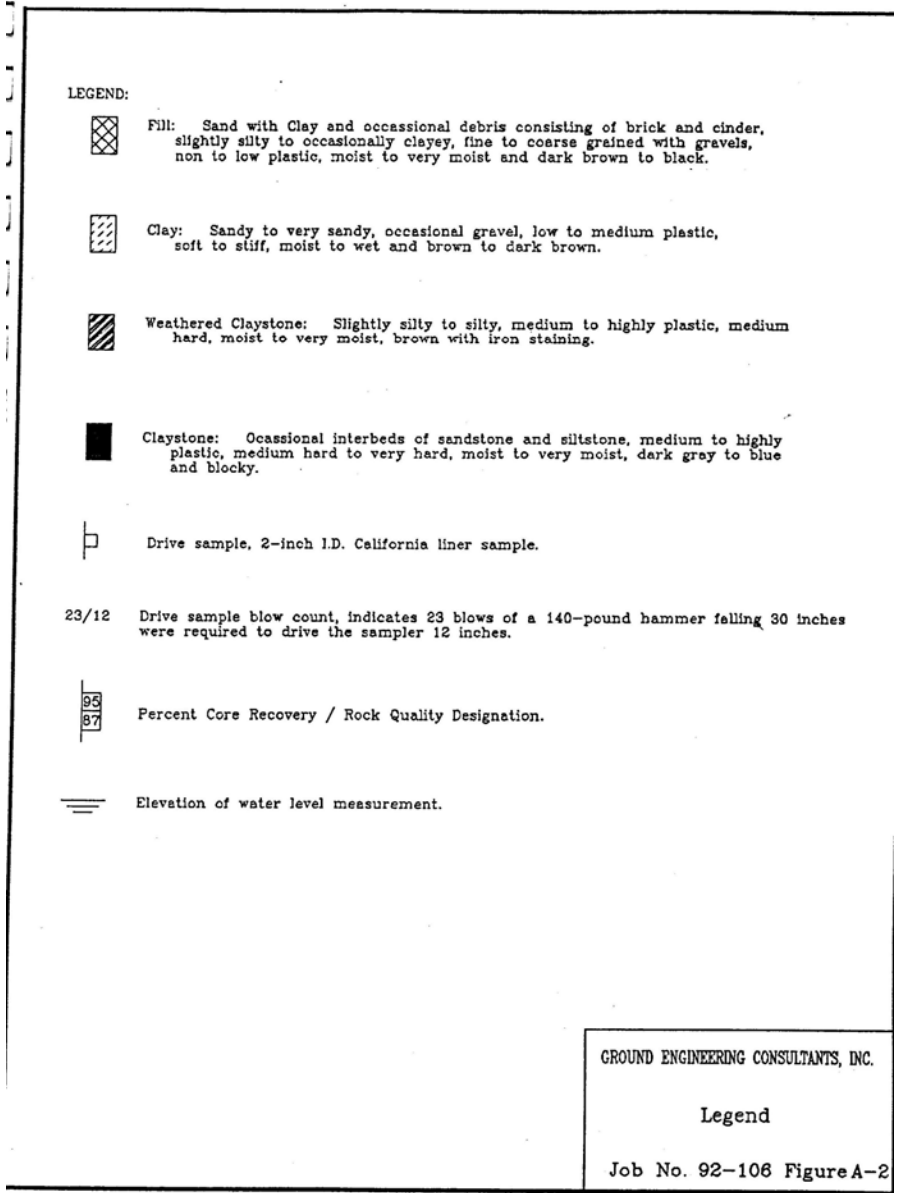


Figure D.3. The 23rd Street Viaduct Load Test: Log of Test Holes around the Test Shafts.

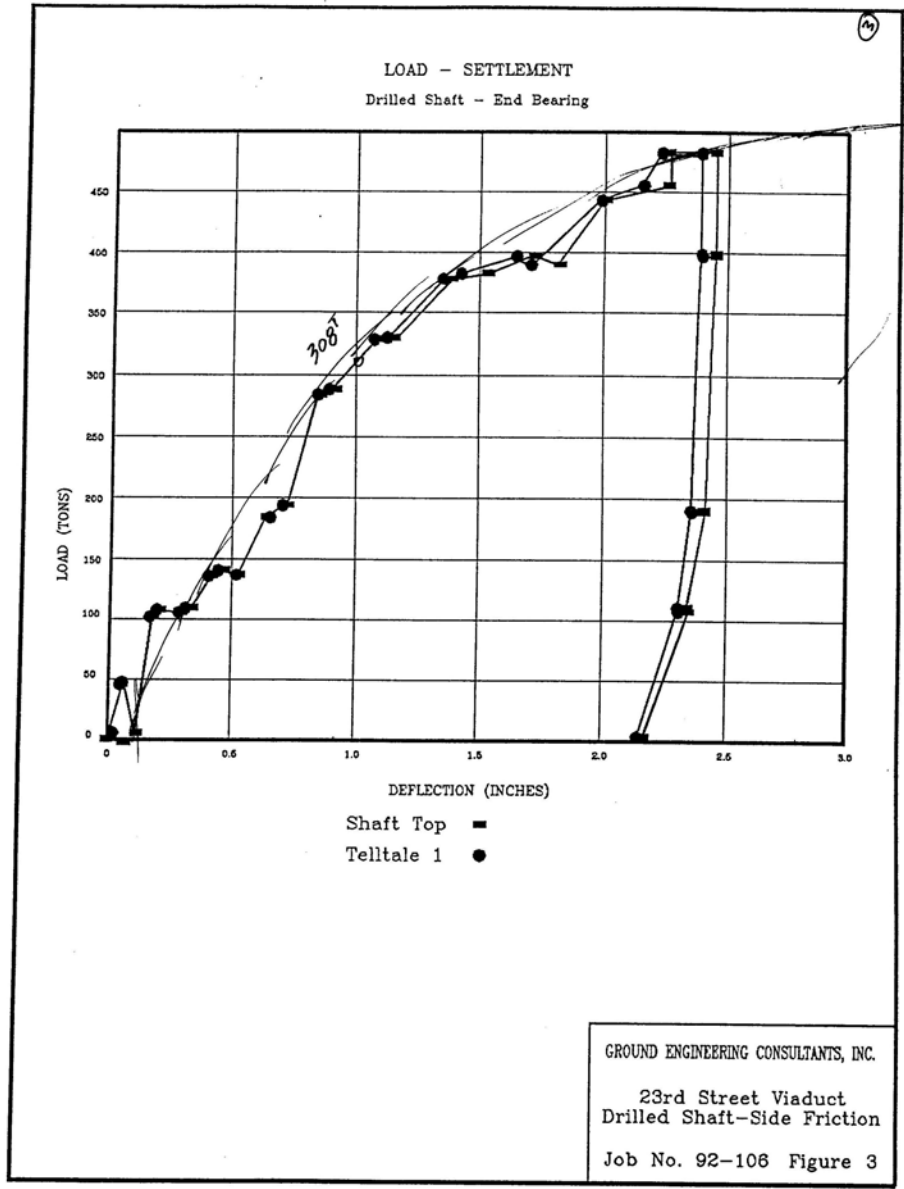


Figure D.5. The 23rd Street Viaduct Load Test: Load-Settlement Results for the End Bearing Test.

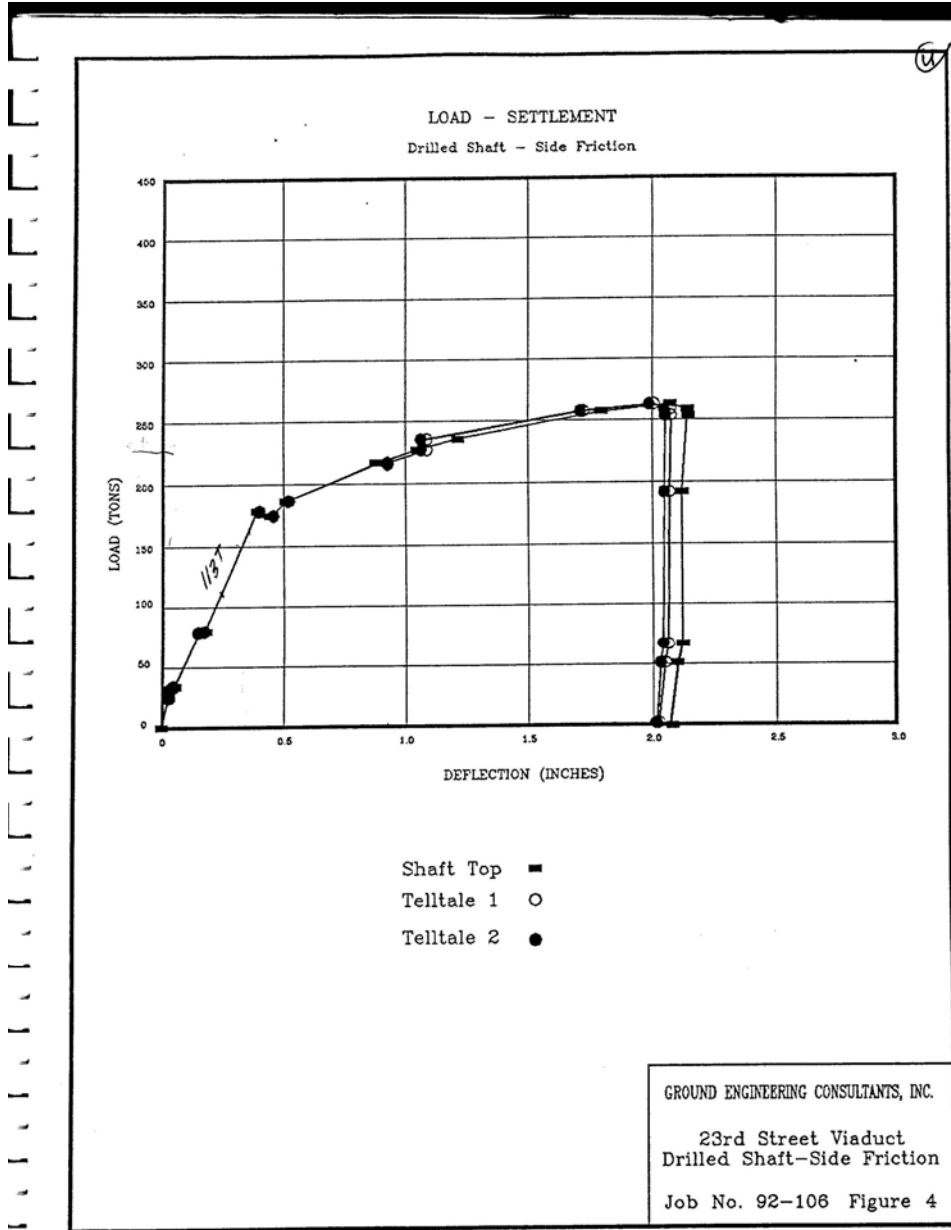


Figure D.6. The 23rd Street Viaduct Load Test: Load-Settlement Results for the Side Resistance Test without Shear Rings.

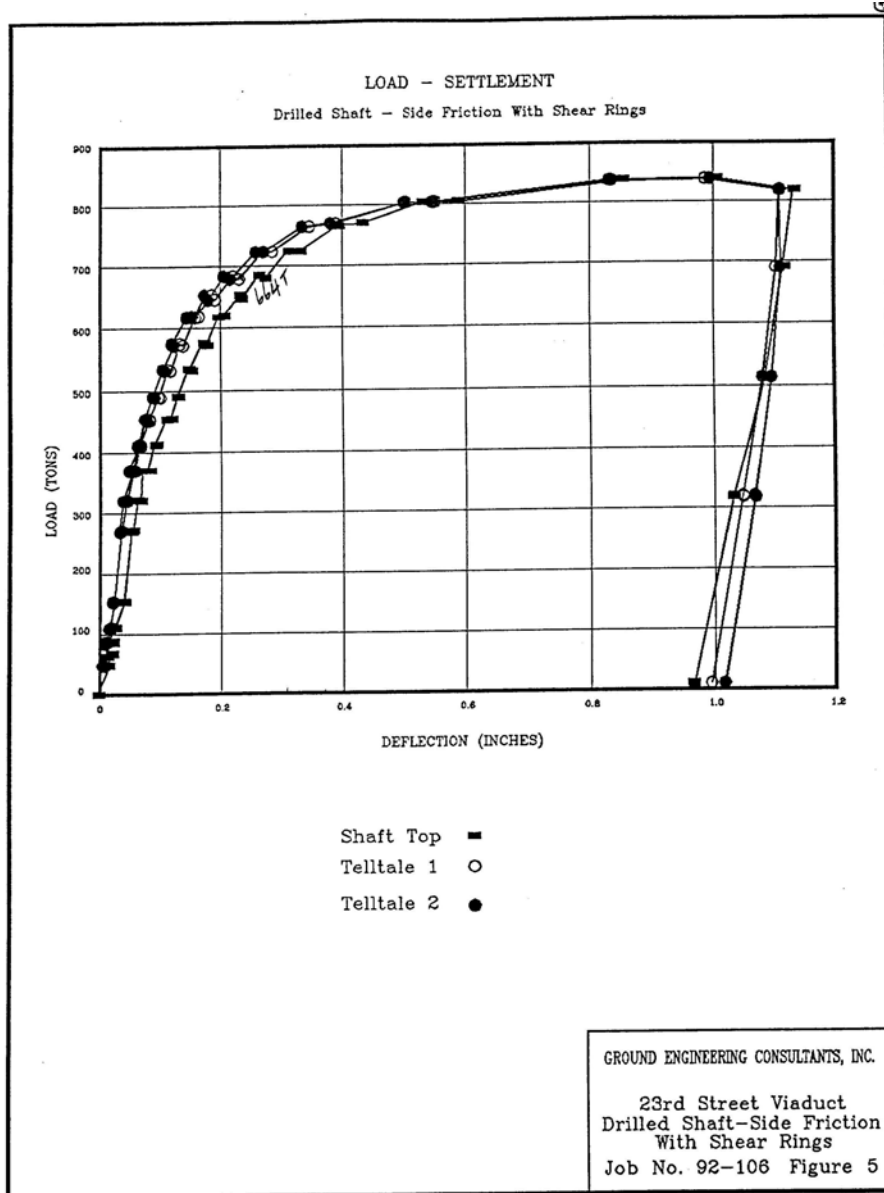
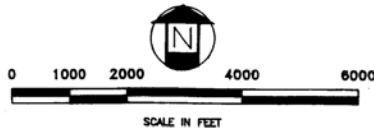
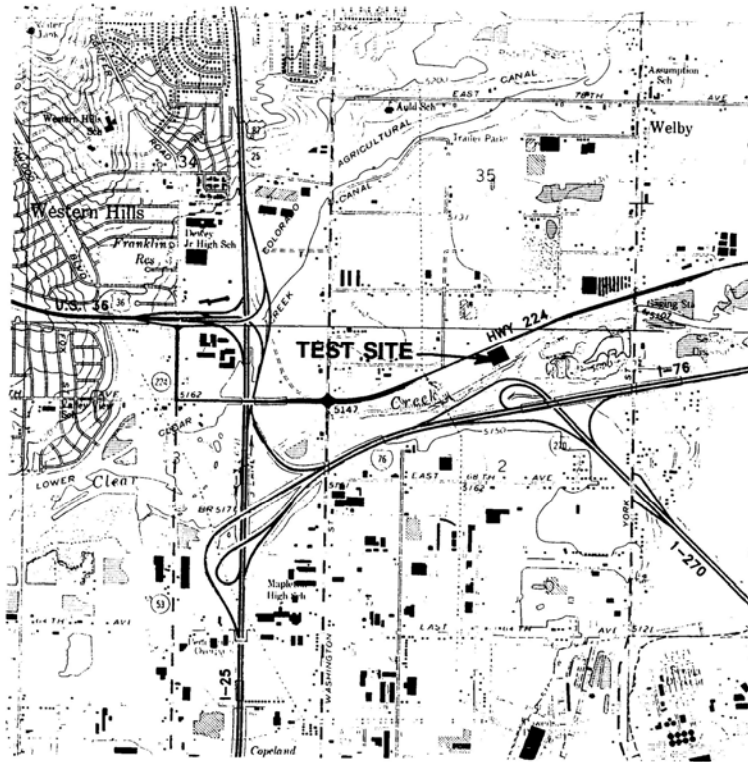


Figure D.7. The 23rd Street Viaduct Load Test: Load-Settlement Results for the Side Resistance Test with Shear Rings.



Job No. : 23040
Prepared by: GWD
Date: 9/9/92

LOCATION MAP
 FOUNDATION LOAD TEST
 CDOT PROJECT NO.
 IM-IR(CX) 25-3(107)



FIG. 1

Figure D.8. The I-270/I-76 load tests: Location.

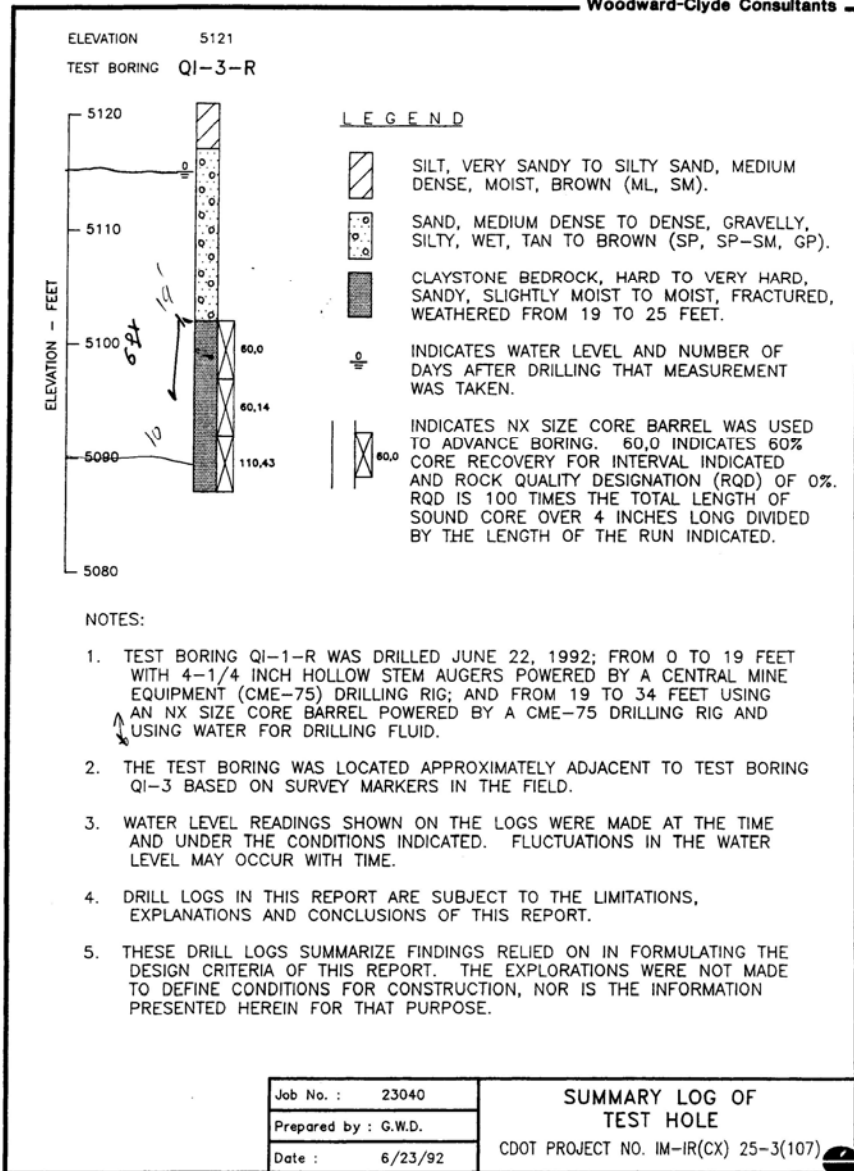


FIG. 2

Figure D.9. The I-270/I-76 Load Tests: Log of Test Hole around the Test Shafts.

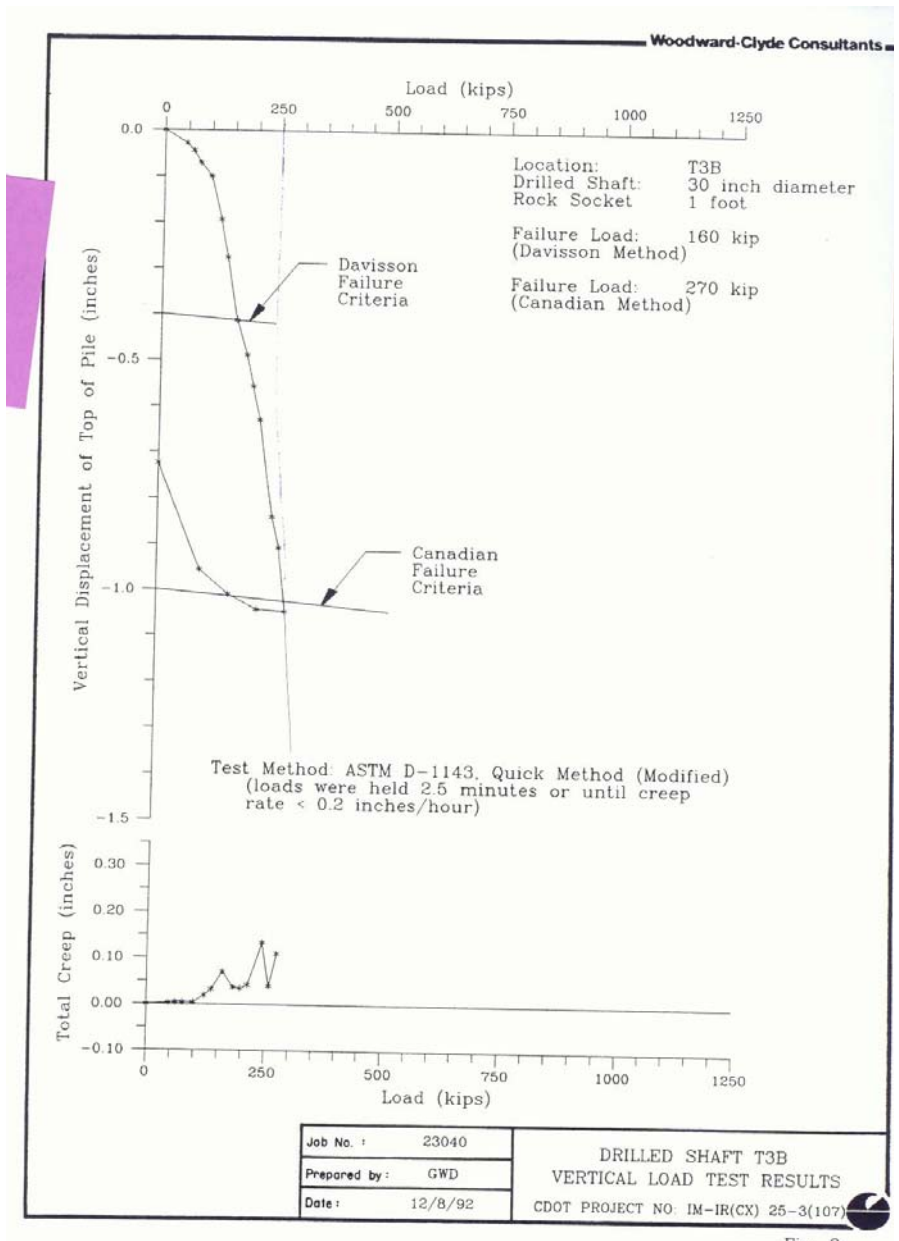


Figure D.10. The I-270/I-76 Load Tests: Results of Load Test 1.

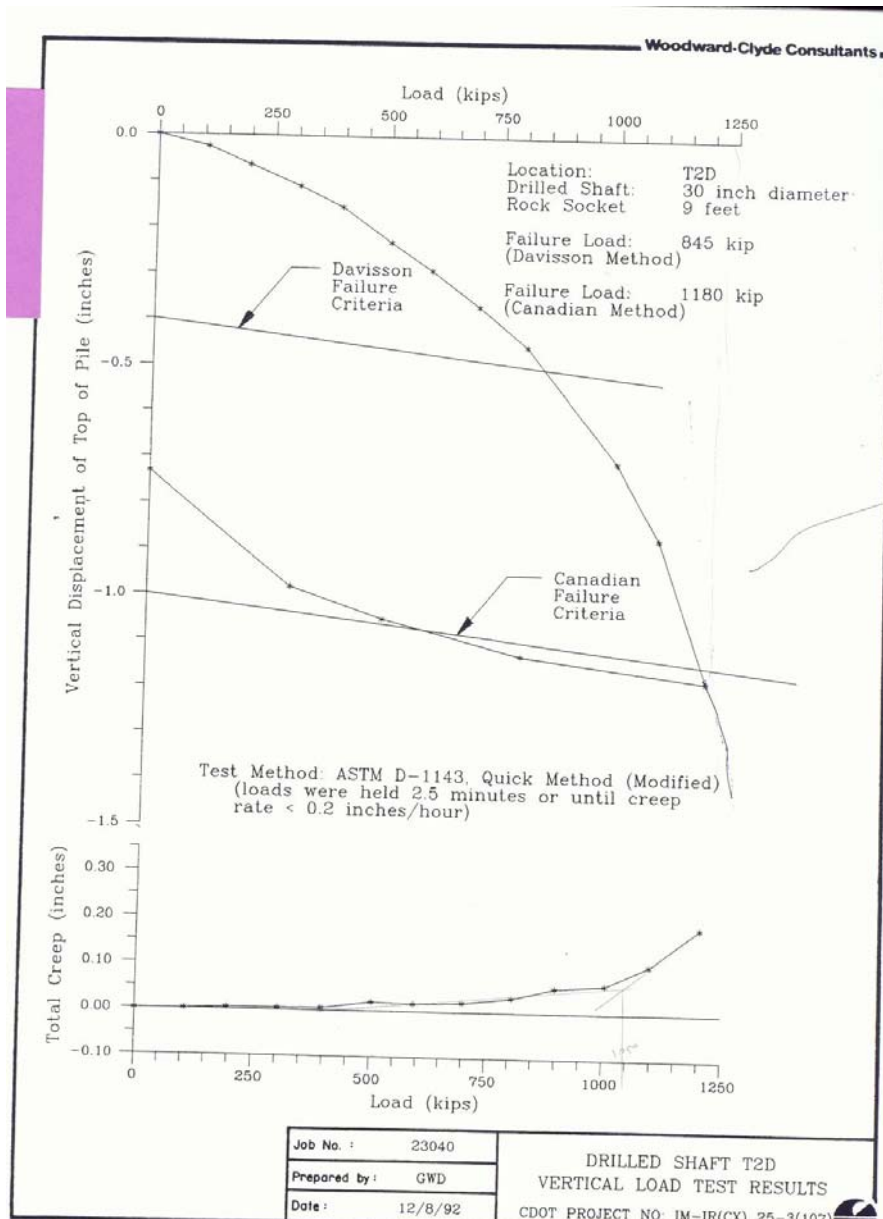


Fig. 12

Figure D.11. The I-270/I-76 Load Tests: Results of Load Test 2.

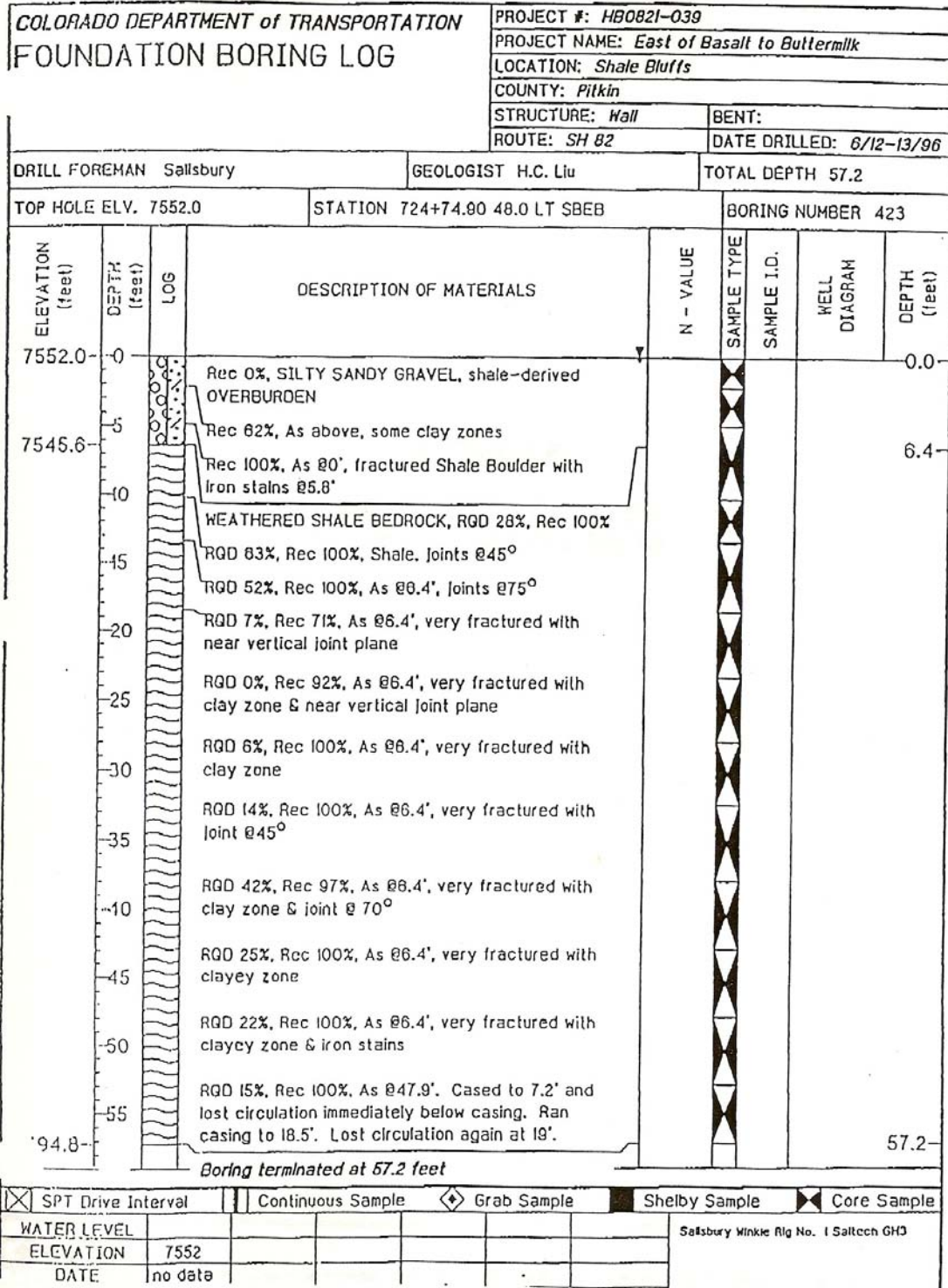


Figure D.12. The SH 82 Load Tests: Log of Test Hole for Load Test 1.

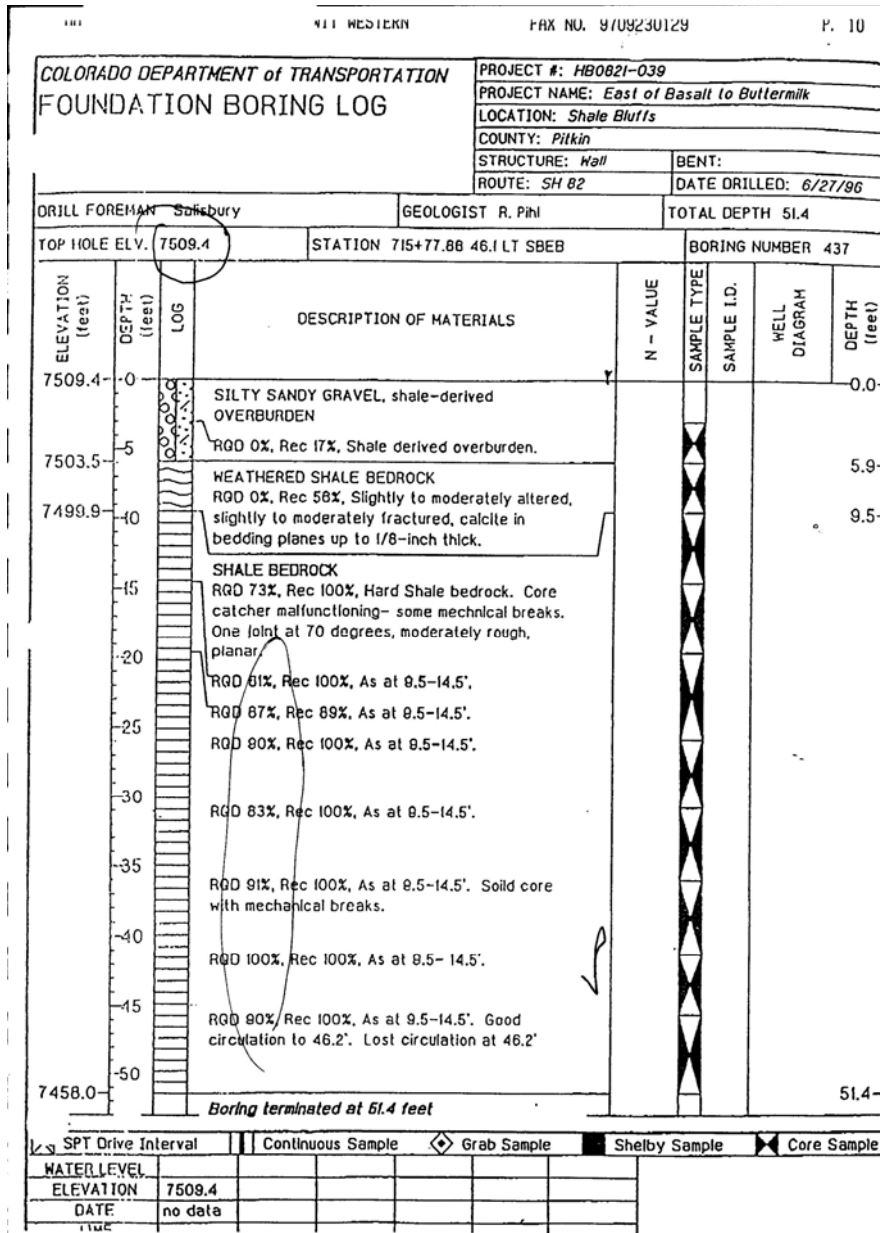


Figure D.13. The SH 82 Load Tests: Log of Test Hole for Load Test 2.

TABLE A: SUMMARY OF DIMENSIONS, ELEVATIONS, AREAS & PROPERTIES FOR ANALYSIS PURPOSES

Shaft:			
Nominal shaft diameter	=	0.9 meters	36 inches
O-cell size: Serial No. 8005-13	=	21 inches	21 inches
Length of concrete above break at base of O-cell	=	11.7 meters	38.4 feet
Length of side shear socket below break at base of O-cell	=	0.0 meters	1.3 feet
Shaft shear area above O-cell™ base	=	33.6 meters ²	361.9 feet ²
Shaft shear area below O-cell™ base	=	1.1 meters ²	12.3 feet ²
Shaft end area	=	0.7 meters ²	7.1 feet ²
Weight of concrete (above bottom of O-cell)	=	0.18 MN	20.0 tons
Estimated shaft modulus	=	31.7 GPa	4,595 ksi
Elevation of top of shaft concrete	=	+2292.3 meters	+7520.7 feet
Elevation of mud line	=	+2291.7 meters	+7518.7 feet
Elevation of bottom of 34" Osterberg Cell	=	+2280.6 meters	+7482.3 feet
<small>(The break between upward side shear movement and downward side shear and end bearing movement.)</small>			
Elevation of shaft tip	=	+2280.2 meters	+7481.0 feet
Compression Sections:			
Elevation of top of telltale used for shaft compression	=	+2292.6 meters	+7521.7 feet
Elevation of bottom of telltale used for shaft compression (on top of upper O-cell bearing plate)	=	+2281.0 meters	+7483.5 feet
Strain Gages:			
Elevation of strain gage Level 2	=	+2289.1 meters	+7510.2 feet
Elevation of strain gage Level 1	=	+2284.8 meters	+7496.2 feet
Miscellaneous:			
Top plate Diameter	=	NA mm	NA inches
Bottom plate Diameter	=	762 mm	30 inches
Water elevation*	=	+2281.1 meters	+7484.0 feet

Handwritten notes:
 A circle around +7510.2 feet with "14" written next to it.
 A line from +7496.2 feet to "16'" written next to it.
 The number "7482.3" is written below the other elevations.



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Figure D.14. SH 82 Load Test 2: Properties of the Test Shaft.

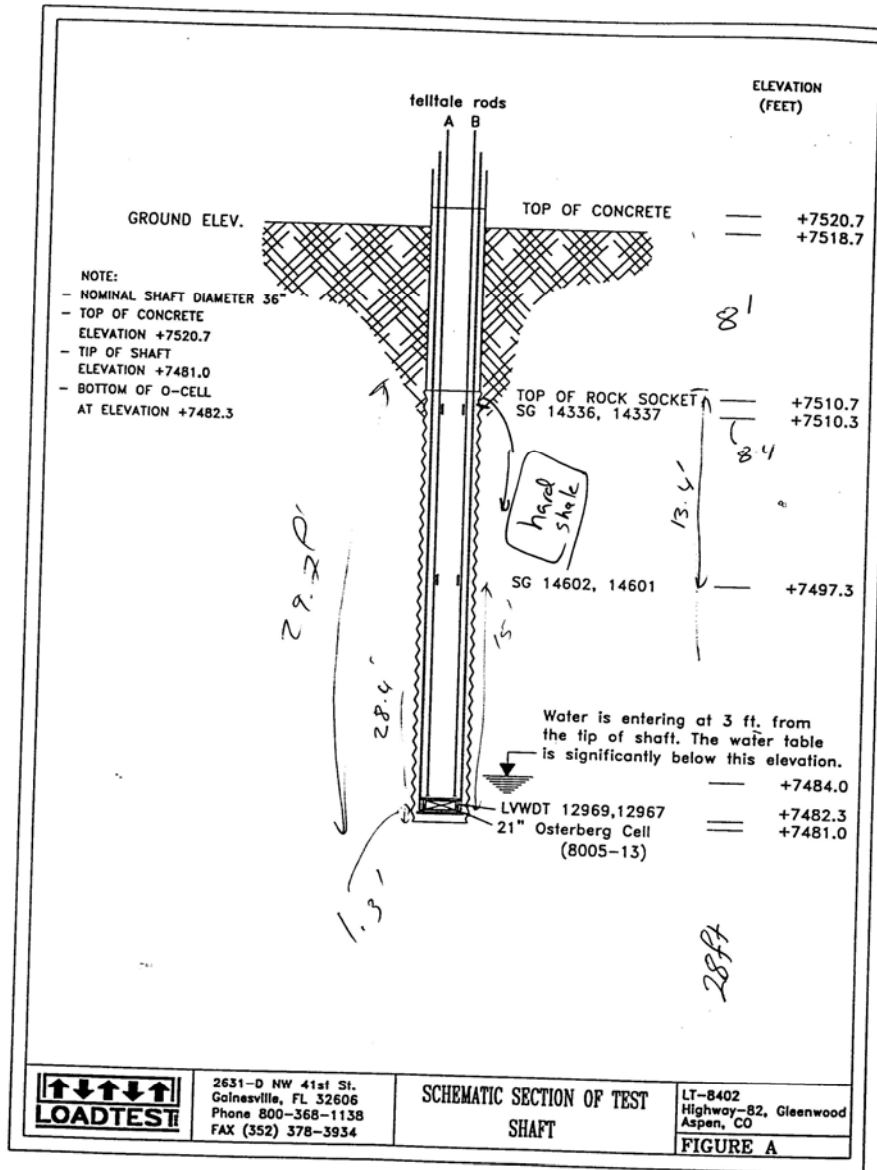


Figure D.15. SH 82 Load Test 2: Layout of the Test Shaft.

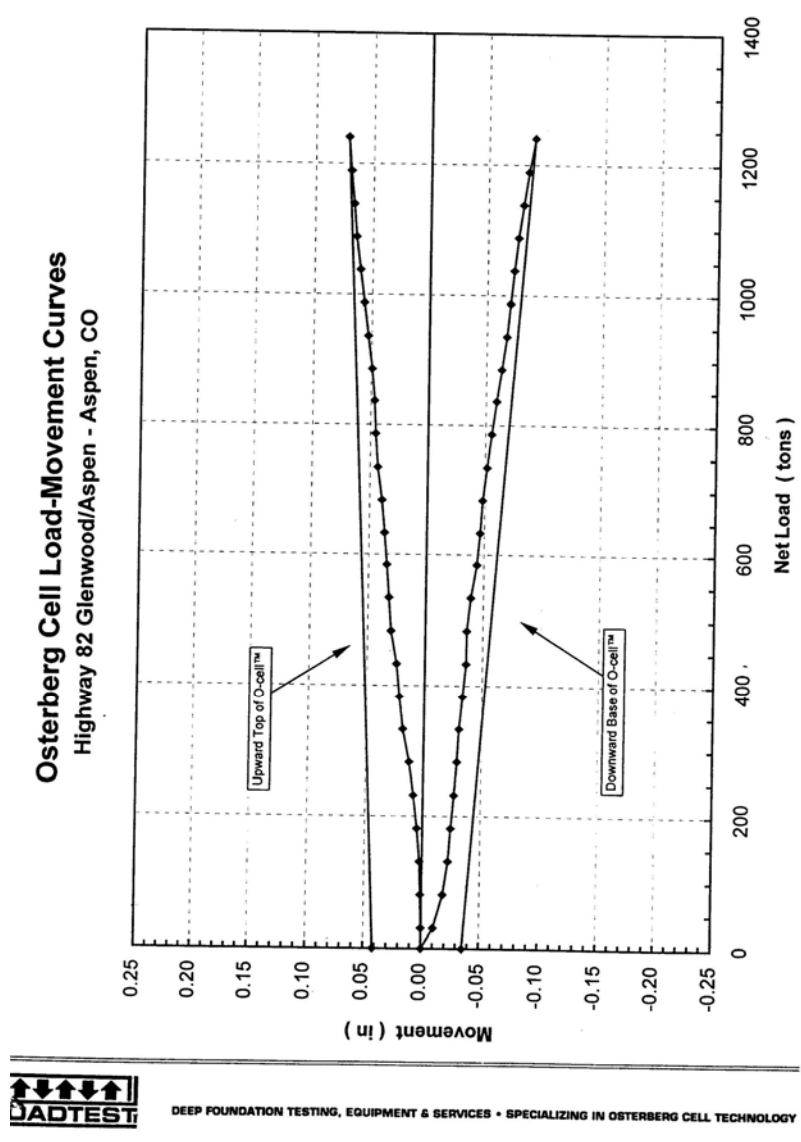


Figure 1 of 5

LOADTEST, Inc. Project No. LT-8402-2



Figure D.16. SH 82 Load Test 2: Testing Results.

APPENDIX E: RESULTS OF LOAD TEST INVESTIGATION IN THE TRINIDAD PROJECT

Part 1: Results of the Subsurface Geotechnical Investigation, pages E-2 to E-11.

Part 2: Results of the Two O-Cell Load Tests, Pages E-12 to E-15.

May 28, 2003

Subject: Subsurface Exploration and
Laboratory Testing, Caisson Load Test, I-25
Trinidad Project, Trinidad, Colorado

Task Order No. 6
Job Number 03-3099

Mr. Thomas Anzia
FELSBURG HOLT & ULLEVIG
7951 East Maplewood Avenue, Suite 200
Greenwood Village, Colorado 80111

Dear Mr. Anzia,

This letter presents the results of a subsurface exploration and laboratory testing program performed for the proposed caisson load test of the I-25 Trinidad Project located in Trinidad, Colorado. As requested by the CDOT Research Branch, a GROUND's engineer logged the test holes drilled at the project site and shipped the soil and core samples to GROUND's laboratory. A laboratory testing program was then performed on the soil and core samples.

Subsurface Exploration

The subsurface exploration was conducted on April 22, 23, 28, and 29, 2003. Locations of the proposed load tests were premarked by the representative of CDOT. A total of four (4) test holes were advanced in the proposed I-25 Viaduct areas, two (2) test holes near each of the proposed load test location. Near each proposed load test location, one test hole (Test Hole 1A or 2A) was advanced through the overburden soils into the underlying bedrock with 7-inch diameter continuous hollow stem augers, and the other test hole (Test Hole 1B or 2B) was advanced through overburden soils with 7-inch diameter hollow stem augers to the top of bedrock and advanced into the underlying bedrock with 2.5-inch I.D. coring. The approximate locations of the test holes are shown in Figure 1. The test holes were drilled and cored by the CDOT drilling crew, and were logged by an engineer of Ground Engineering Consultants, Inc.

Samples of the subsurface materials were taken with a 1-3/8 inch I.D. standard sampler and 2.5 inch I.D. coring. The sampler was driven into the various strata with blows from a 140-pound hammer falling 30 inches as described by ASTM Method D-1586. Penetration resistance values, when properly evaluated, indicate the relative density or consistency of the soils and bedrock. Depths at which the samples were taken and the penetration resistance values are shown on the drill logs of Test Holes 1A and 1B in Figure 2. Depths of the cores obtained and the core data are presented on the core logs of Test Holes 2A and 2B in Figures 3 and 4, respectively. The associated legend and notes are shown in Figure 5.

Measurements of the groundwater level were made in the test holes by lowering a weighted tape into the open holes shortly after completion of the drilling and 6 days after the drilling in Test Hole 1A. The location of the groundwater levels measured and the number of days subsequent to drilling are shown on the drill logs of test holes.

Laboratory Testing

Soil and core samples obtained from our exploratory holes were examined and classified in the laboratory by the project engineer. Laboratory testing included standard property tests such as natural moisture contents, dry unit weights, grain size analyses, and liquid and plastic limits. Unconfined compression test was conducted on selected core samples at approximately 5-foot intervals to determine the unconfined compressive strength of the underlying bedrock. Concentration of water soluble sulfates was determined on two selected samples of the overburden soils. Laboratory test results and AASHTO classifications are summarized on Table 1. The laboratory testing was conducted in general accordance with applicable ASTM and AASHTO specifications.

Subsurface Conditions

The subsurface conditions encountered in the test holes generally consisted of an approximate 8 to 10 foot layer of man-made fill overlying natural sand and silt and natural sand and gravel. These materials were underlain by Pierre shale bedrock at depths of approximately 28.5 to 29 feet below the existing grades. Detailed descriptions of each type of the materials encountered in the test holes are shown in Figure 5.

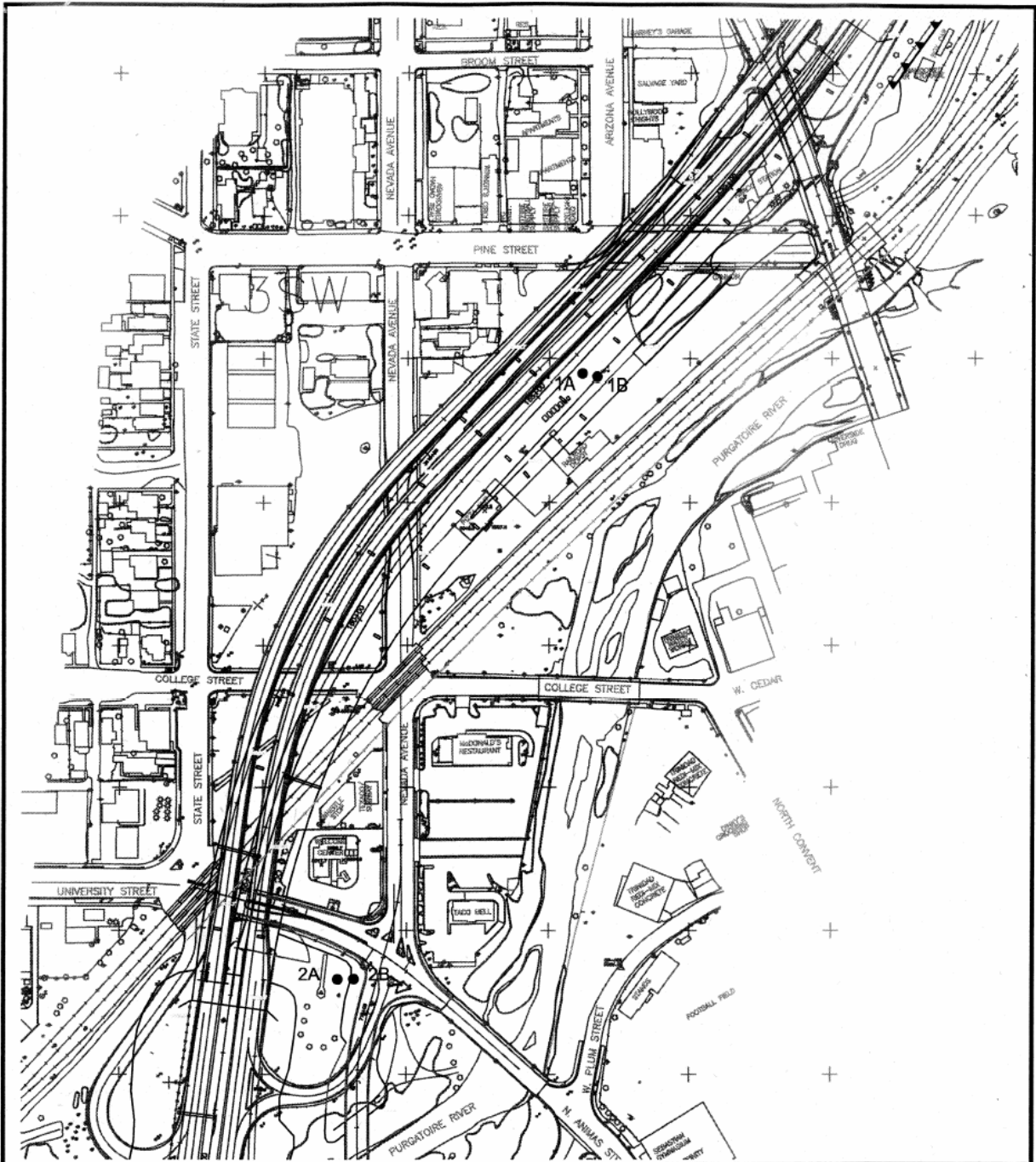
Free groundwater was encountered in Test Holes 1A and 1B at depths ranging from approximately 13 to 14.5 feet below the existing grades at the time of drilling. Groundwater was encountered in Test Hole 1A at a depth of approximately 11 feet below the existing grade 6 days after the drilling. Groundwater levels can be expected to fluctuate in response to seasonal variations in precipitation, periodic perched water conditions, and post-construction landscape irrigation.

Should you have any questions regarding the geotechnical data and laboratory test results presented herein, please contact our office.

Sincerely,
Ground Engineering Consultants, Inc.



Hsien-Hsiang (Sean) Chiang, Ph.D., P.E.

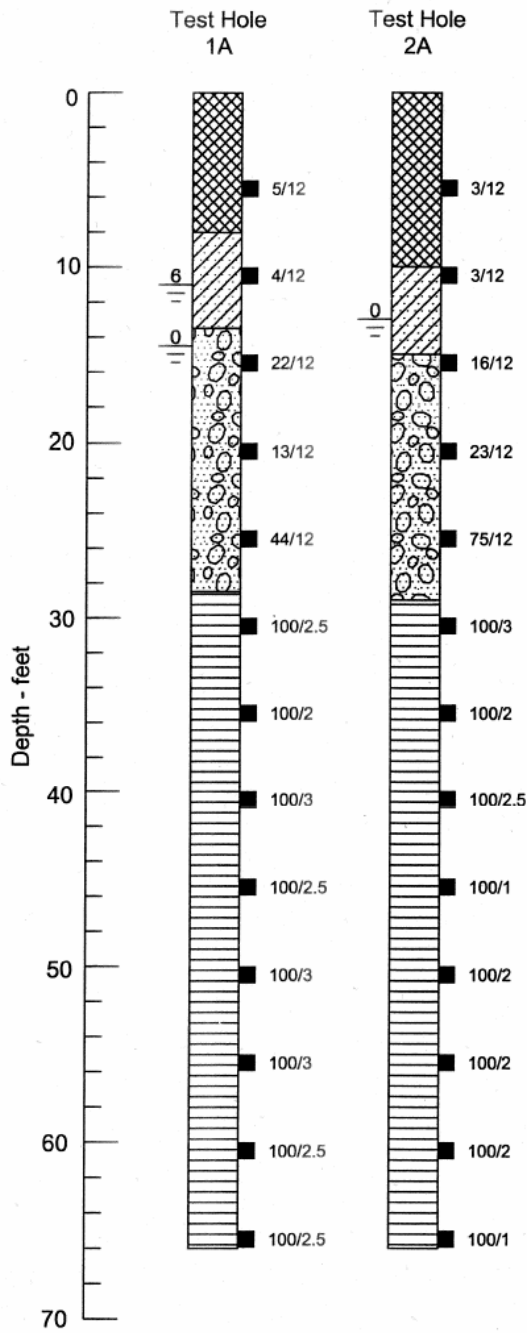


1
 ● Indicates test hole number and approximate location.



(Not to Scale)

GROUND ENGINEERING CONSULTANTS	
LOCATION OF TEST HOLES	
JOB NO. 03-3099	DRAWN BY: HS
FIGURE: 1	APPROVED BY: HHC
CADFILE NAME: 3099SITE.DWG	




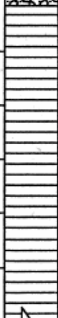


GROUND ENGINEERING CONSULTANTS	
LOGS OF TEST HOLES	
JOB NO. 03-3099	DRAWN BY: HS
FIGURE: 2	APPROVED BY: HHC
CADFILE NAME: 3099LOG.DWG	

GROUND ENGINEERING CONSULTANTS, INC.

CORING LOG

PROJECT NO. 03-3099	PROJECT I-25 AT TRINIDAD	SURFACE ELEV. -	BORING NO. TH-1B
LOGGED BY: D. EVIG	LOCATION NORTH LOAD TEST	DEPTH TO GROUNDWATER 14.5 feet	SHEET 1 OF 2
DATE 04/22/03	COORDINATES -	DEPTH TO BEDROCK 28.5 feet	HOLE DIAMETER 7" / 4"
DRILLER DAKOTA		TOTAL DEPTH 67 feet	TREND -
DRILL RIG 550	CAD FILE NAME 3099DRLOG01.DWG	FIGURE 3	PLUNGE -

STRATA ELEVATION	SAMPLE DATA			CORE DATA		FRACTURES	DEPTH (FEET)	GRAPHIC LOG	LITHOLOGIC DESCRIPTION	NOTES ON DRILLING CONDITIONS		
	SAMPLE TYPE	SAMPLE NO.	BLOW COUNT	PERCENT RECOVERY	RQD						DESCRIPTION	LOG
							0		Fill: Sand and Clay			
							10		Sand and Silt			
							20		Sand and Gravel			
	NX	R1		-	-	Only crumbled sample	30		Pierre Shale Bedrock			
	NX	R2		32	23							
	NX	R3		94	94							
							40					

GROUND ENGINEERING CONSULTANTS, INC.

CORING LOG

PROJECT NO. 03-3099	PROJECT I-25 AT TRINIDAD	SURFACE ELEV. -	BORING NO. TH-1B
LOGGED BY: D. EVIG	LOCATION NORTH LOAD TEST	DEPTH TO GROUNDWATER 14.5 feet	SHEET 2 OF 2
DATE 04/22/03	COORDINATES -	DEPTH TO BEDROCK 28.5 feet	HOLE DIAMETER 7" / 4"
DRILLER DAKOTA		TOTAL DEPTH 67 feet	TREND -
DRILL RIG 550	CAD FILE NAME 3099DRLOG02.DWG	FIGURE 3 (CONT.)	PLUNGE -

STRATA ELEVATION	SAMPLE DATA				CORE DATA		DEPTH (FEET)	GRAPHIC LOG	LITHOLOGIC DESCRIPTION	NOTES ON DRILLING CONDITIONS	
	SAMPLE TYPE	SAMPLE NO.	BLOW COUNT	PERCENT RECOVERY	ROD	FRACTURES					
						DESCRIPTION					LOG
							40		Pierre Shale Bedrock		
	NX	R4		99	97				Clay joint approx. 5.5" thick at 43 feet		
	NX	R5		100	88				Clay joint approx. 2" thick at 49 feet		
	NX	R6		96	100	4.3" Fracture Zone Diagonal Fracture Evidence of Snail Smell			Clay joint approx. 0.5" thick at 51 feet		
	NX	R7		98	94						
	NX	R8		100	100						
						EOC					
							70				
							80				

GROUND ENGINEERING CONSULTANTS, INC.

CORING LOG

PROJECT NO. 03-3099	PROJECT I-25 AT TRINIDAD	SURFACE ELEV. -	BORING NO. TH-2B
LOGGED BY: D. EVIG	LOCATION SOUTH LOAD TEST	DEPTH TO GROUNDWATER 13.0 feet	SHEET 1 OF 2
DATE 04/29/03	COORDINATES -	DEPTH TO BEDROCK 29 feet	HOLE DIAMETER 7" / 4"
DRILLER DAKOTA		TOTAL DEPTH 69.0 feet	TREND -
DRILL RIG 550	CAD FILE NAME 3099DRLOG03.DWG	FIGURE 4	PLUNGE -

STRATA ELEVATION	SAMPLE DATA			CORE DATA		DEPTH (FEET)	GRAPHIC LOG	LITHOLOGIC DESCRIPTION	NOTES ON DRILLING CONDITIONS		
	SAMPLE TYPE	SAMPLE NO.	BLOW COUNT	PERCENT RECOVERY	ROD					FRACTURES	
										DESCRIPTION	LOG
						0	Fill: Sand and Clay with Gravel				
						10	Sand and Silt				
						20	Sand and Gravel				
	NX	R1		100	74	Broken Pieces	Pierre Shale Bedrock				
	NX	R2		100	100		Clay joint approx. 1.5" thick at 32.5 feet				
						40					

GROUND ENGINEERING CONSULTANTS, INC.

CORING LOG

PROJECT NO. 03-3099	PROJECT I-25 AT TRINIDAD	SURFACE ELEV. -	BORING NO. TH-2B
LOGGED BY: D. EVIG	LOCATION SOUTH LOAD TEST	DEPTH TO GROUNDWATER 13.0 feet	SHEET 2 OF 2
DATE 04/29/03	COORDINATES -	DEPTH TO BEDROCK 29 feet	HOLE DIAMETER 7" / 4"
DRILLER DAKOTA		TOTAL DEPTH 69.0 feet	TREND -
DRILL RIG 550	CAD FILE NAME 3099DRLOG04.DWG	FIGURE 4 (CONT.)	PLUNGE -

STRATA ELEVATION	SAMPLE DATA			CORE DATA		DEPTH (FEET)	GRAPHIC LOG	LITHOLOGIC DESCRIPTION	NOTES ON DRILLING CONDITIONS		
	SAMPLE TYPE	SAMPLE NO.	BLOW COUNT	PERCENT RECOVERY	RCD					FRACTURES	
										DESCRIPTION	LOG
	NX	R4		100	94	40		Pierre Shale Bedrock			
	NX	R5		100	100			Vertical Crack			
	NX	R6		98	98	50		Small Horizontal Crack			
	NX	R7		100	100			Brown Spot			
	NX	R8		100	98	60					
	NX	R9		92	85			Clay joint approx. 1" thick at 61.5 feet			
								Clay joint approx. 2" thick at 68.5 feet			
						EOC					
						70					
						80					

LEGEND:



Fill: Clayey sand and gravel to sandy clay, occasionally silty, fine grained to gravel, non to medium plastic, compacted, moist, tan to brown in color.



Sand and Silt: Silty sand to sandy silt with occasional sandy clay layers, fine to medium grained with occasional gravel, non to medium plastic, stiff to very stiff or loose to medium dense, moist to wet, brown to dark brown to gray in color.



Sand and Gravel: Clayey to silty, fine grained to gravel with occasional cobbles, non to low plastic, medium dense to dense, moist to wet, tan to brown in color.



Pierre Shale: Very cemented, fine grained, low to medium plastic, very hard, slightly moist, gray to dark gray in color.



Drive sample, 1-3/8 inch I.D. standard sample

23/12 Drive sample blow count, indicates 23 blows of a 140-pound hammer falling 30 inches were required to drive the sampler 12 inches.

$\frac{0}{\text{---}}$ Depth to water level and number of days after drilling that measurement was taken.

NOTES:

- 1) Test holes were drilled on 04/22-29/03 with 7-inch diameter continuous hollow stem augers.
- 2) Locations of the test holes were premarked by the representative of CDOT.
- 3) Elevations of the test holes were not measured and the logs of the test holes are drawn to depth.
- 4) The test hole locations and elevations should be considered accurate only to the degree implied by the method used.
- 5) The lines between materials shown on the test hole logs represent the approximate boundaries between material types and the transitions may be gradual.
- 6) Groundwater level readings shown on the logs were made at the time and under the conditions indicated. Fluctuations in the water level may occur with time.

GROUND ENGINEERING CONSULTANTS	
LEGEND AND NOTES	
JOB NO. 03-3099	DRAWN BY: HS
FIGURE: 5	APPROVED BY: HHC
CADFILE NAME: 3099LEG.DWG	

**TABLE 1
SUMMARY OF LABORATORY TEST RESULTS**

Sample Location	Natural Moisture Content (%)	Natural Dry Density (pcf)	Gradation			Atterberg Limits		Unconfined Compressive Strength (psf)	Water Soluble Sulfates (%)	AASHTO Classification (GI)	Soil or Bedrock Type
			Gravel (%)	Sand (%)	Passing No. 200 Sieve	Liquid Limit (%)	Plasticity Index (%)				
1A 5	16.1	SD			67	33	16		0.08	A-6 (8)	Fill: Sandy Clay
1A 10	17.2	SD			57	25	6			A-4 (1)	Sand and Silt
1A 15	13.4	SD	54	39	7	23	9			A-2-4 (0)	Sandy Gravel
1A 20	20.5	SD	21	71	8	NV	NP			A-1-b (0)	Gravelly Sand
1A 25	11.2	SD	50	42	8	20	2			A-1-a (0)	Sand and Gravel
1B 35 - 35.5	3.2	157.5						470,380			Pierre Shale
1B 39.5 - 40	4.4	159.4				31	15	396,830		A-6 (14)	Pierre Shale
1B 45 - 45.5	4.3	156.8						352,170			Pierre Shale
1B 50 - 50.5	4.1	158.0				30	12	335,980		A-6 (11)	Pierre Shale
1B 55 - 55.5	3.8	155.6				28	12	438,270		A-6 (11)	Pierre Shale
1B 59.5 - 60	4.1	155.2						518,540			Pierre Shale
1B 65.3 - 65.8	4.1	154.9				32	16	477,110		A-6 (15)	Pierre Shale
2A 5	37.0	SD			45	40	2		< 0.01	A-4 (0)	Fill: Sand and Silt
2A 10	30.9	SD			69	31	13			A-6 (7)	Sandy Clay
2A 15	12.0	SD	72	25	3	24	8			A-2-4 (0)	Sandy Gravel
2A 20	7.8	SD	57	37	6	20	1			A-1-a (0)	Sandy Gravel
2B 34.5 - 35	4.3	151.5						349,580			Pierre Shale
2B 40.5 - 41	4.1	154.5				29	17	346,340		A-6 (15)	Pierre Shale
2B 45.5 - 46	3.5	156.9						517,890			Pierre Shale
2B 50 - 50.5	4.1					25	10	454,510		A-4 (8)	Pierre Shale
2B 55 - 55.5	4.1	156.1						426,610			Pierre Shale
2B 60 - 60.5	4.7	154.7				26	10	191,620		A-4 (8)	Pierre Shale
2B 64.5 - 65	4.8	157.9				34	16	324,980		A-6 (16)	Pierre Shale

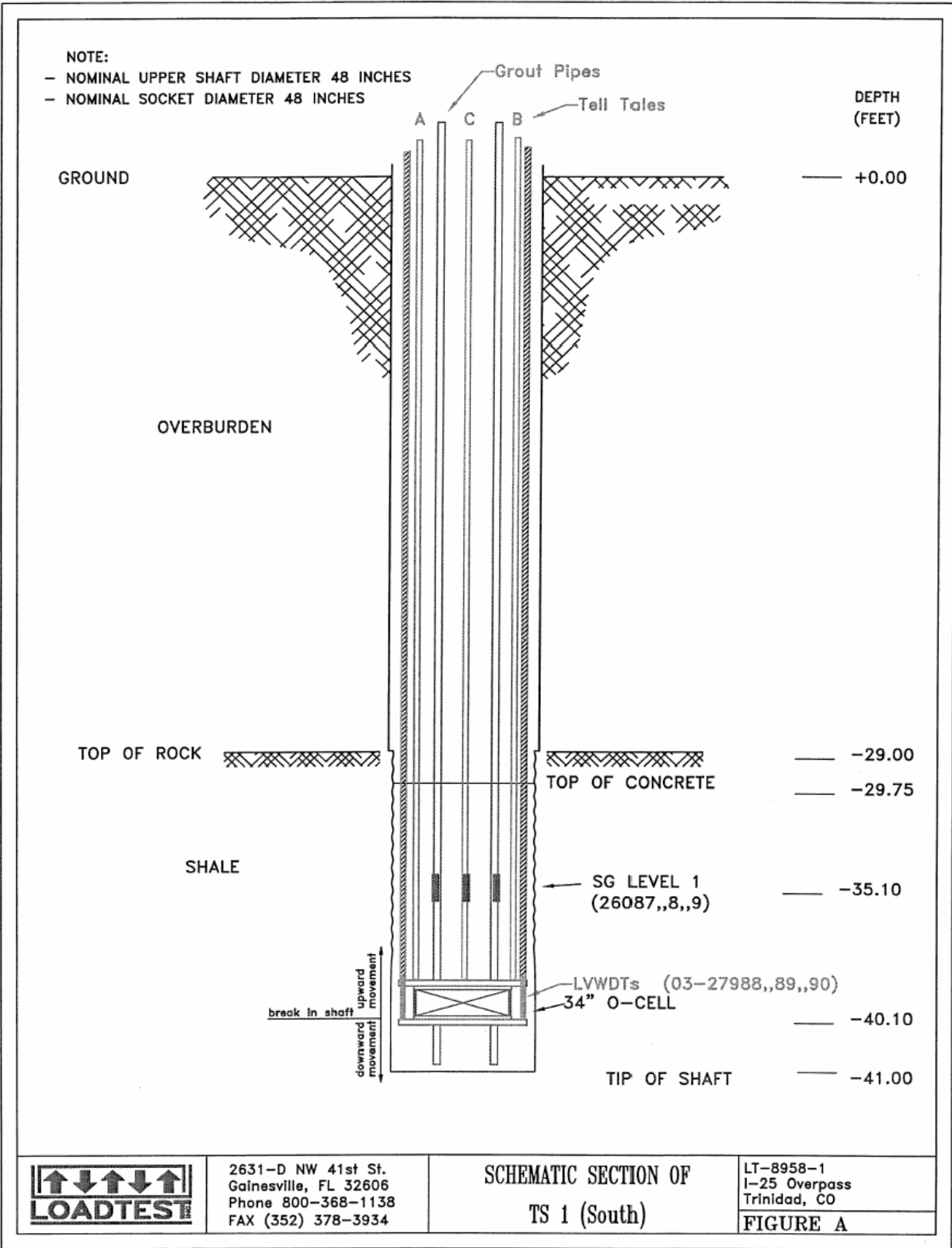


Figure D.1. Layout of the South Load Test Shaft (LOADTEST, Inc., 2003)

Osterberg Cell Load-Movement Curves Trinidad, CO - I-25 Overpass - Test Shaft 1 (South)

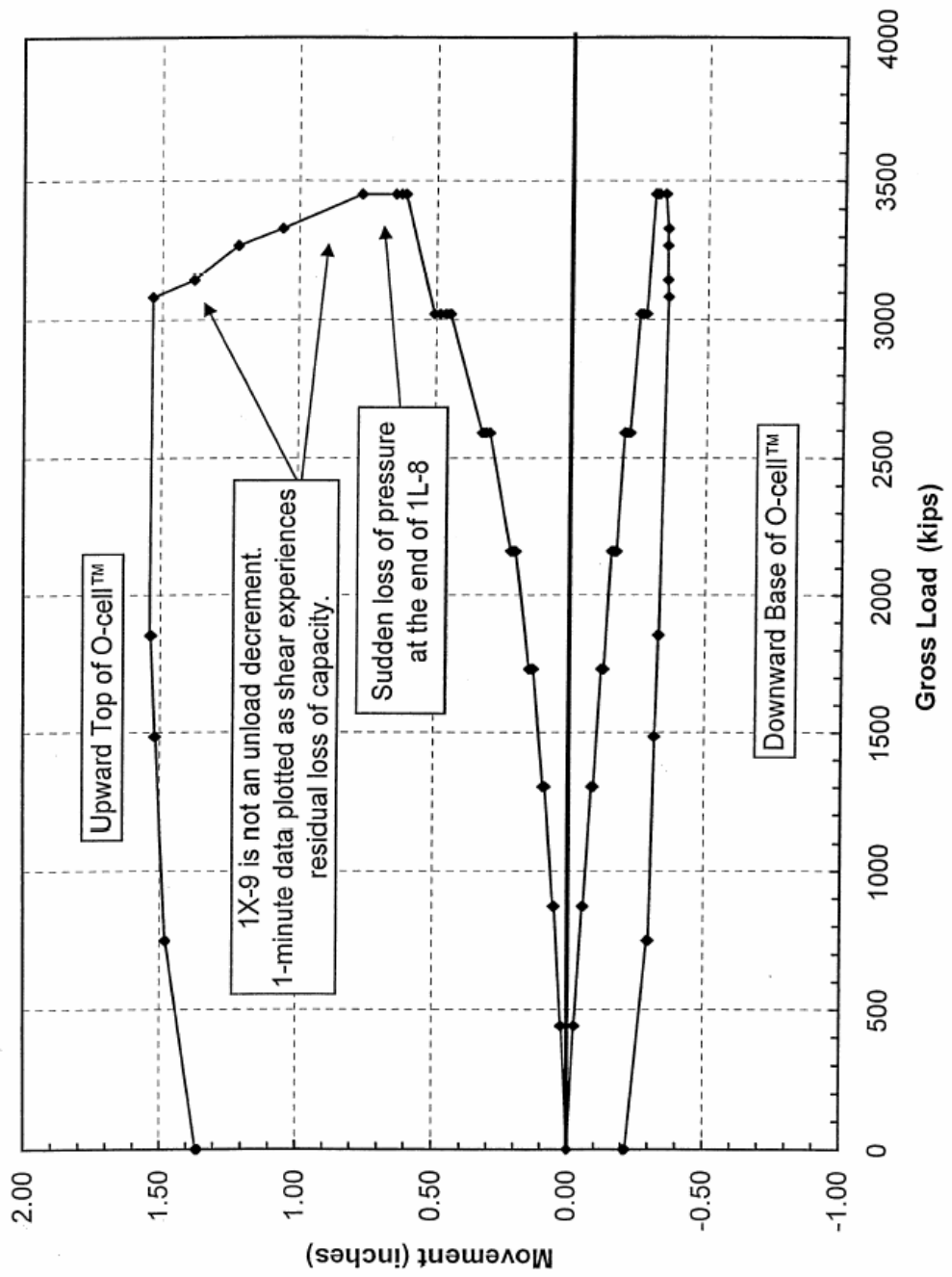


Figure D.2. Results of South Load Test (LOADTEST, Inc., 2003)

**TABLE B: SUMMARY OF DIMENSIONS, ELEVATIONS, AREAS & PROPERTIES
FOR ANALYSIS PURPOSES**

Shaft:			
Nominal shaft diameter: Depth -28.0 ft to -48.0 ft	=	48 inches	1219 mm
O-cell size: (Serial nos.: 2031-2)	=	34 inches	864 mm
Length of concrete from break at base of cell to tip	=	0.8 feet	0.2 meters
Shaft shear area from break at base of cell to tip	=	10.1 feet ²	0.93 meters
Shaft end area	=	12.6 feet ²	1.17 meters
Weight of shaft from break at base of cell to top of shaft	=	24.2 kips	0.11 MN
Estimated shaft unit stiffness: Depth -28.0 ft to -48.0 ft	=	6.38E+06 kips	28.4 GN
Depth of top of shaft concrete	=	-28.0 feet	-8.5 meters
Depth of ground surface (reference)	=	+0.0 feet	+0.0 meters
Depth of break at base of O-cell™	=	-47.2 feet	-14.4 meters
Depth of shaft tip	=	-48.0 feet	-14.6 meters
Casings:			
Depth of top of temporary casing: 48 inches O.D.	=	+1.0 feet	+0.3 meters
Depth of bottom of temporary casing: 48 inches O.D.	=	-30.0 feet	-9.1 meters
Compression Sections:			
Depth of top of telltale used for shaft compression	=	0.5 feet	0.2 meters
Depth of bottom of telltale used for shaft compression	=	-45.8 feet	-14.0 meters
Strain Gages:			
Depth of strain gage Level 1	=	-39.2 feet	-11.9 meters
Miscellaneous:			
Top plate diameter	=	39.0 inches	991 mm
Top plate thickness	=	2.0 inches	50.8 mm
Bottom plate diameter	=	42.0 inches	1067 mm
Bottom plate thickness	=	2.0 inches	50.8 mm
Water elevation	=	-31.0 feet	-9.4 meters
LVWDT radii - no: 03-27985	=	19.0 inches	483 mm
LVWDT orientation - no.: 03-27985	=	0 degrees	
LVWDT radii - no: 03-27986	=	19.0 inches	483 mm
LVWDT orientation - no.: 03-27986	=	180 degrees	
LVWDT radii - no: 03-27987	=	19.0 inches	483 mm
LVWDT orientation - no.: 03-27987	=	270 degrees	
Vertical re-bar size	=	# 8	
Hoop re-bar size	=	# 4	
Number of vertical bars	=	12	

Figure D.3. Layout Information for the North Load Test Shaft (LOADTEST, Inc., 2003)

Osterberg Cell Load-Movement Curves
 Trinidad, CO - I-25 Overpass - Test Shaft 2 (North)

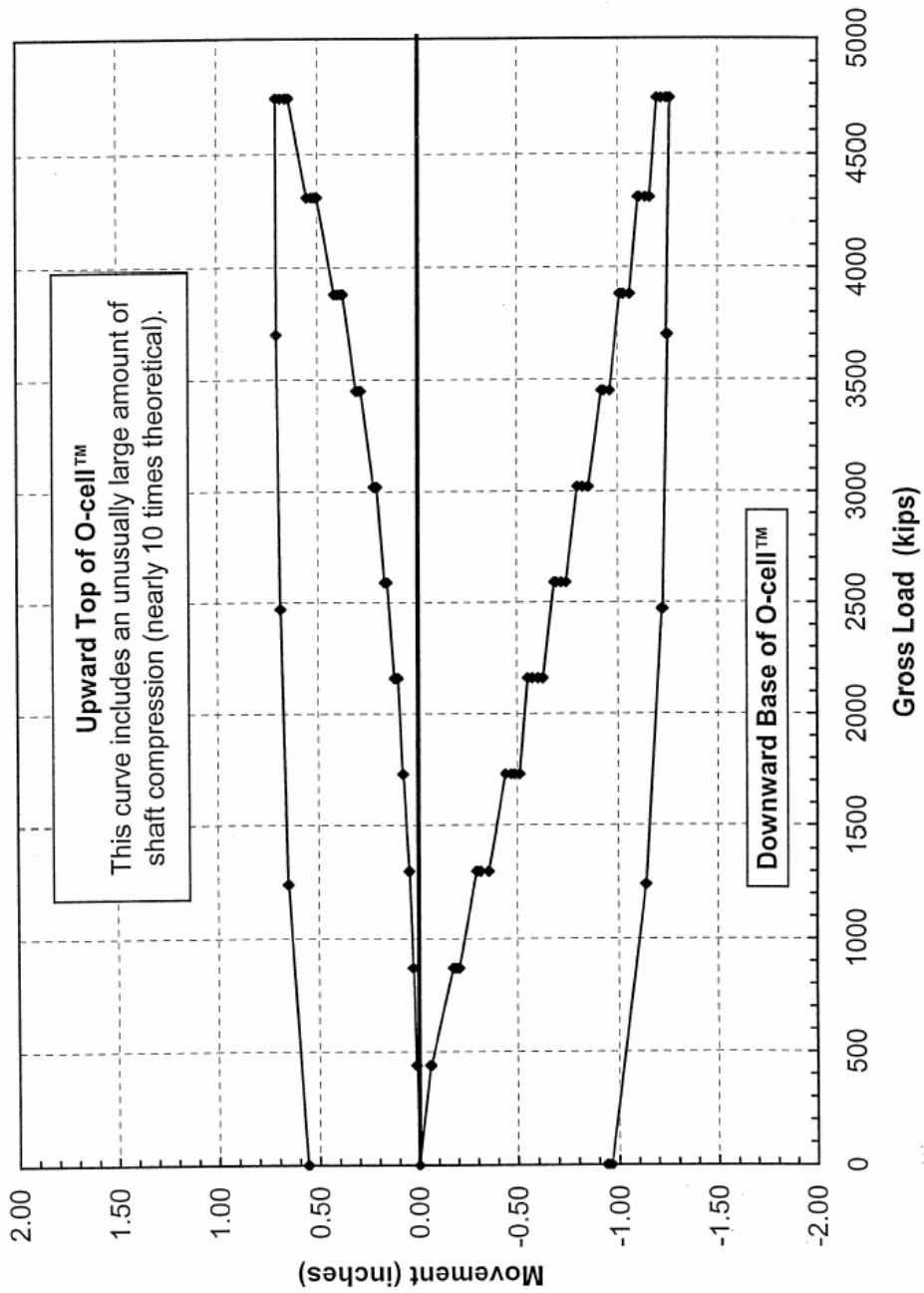


Figure 1 of 4



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LOADTEST, Inc. Project No. LT-8958-2

Figure D.4. Results of the North Load Test (LOADTEST, Inc., 2003)