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MoDOT

PARSONS
Design Directive/Revision

DESIGN DIRECTIVE NO: **B- 001** (Number Consecutively)

DESIGN DIRECTIVE LOG: DESIGN DIRECTIVE NOTEBOOK

DATE: 5/1/07

SUBJECT: **DRILLED SHAFT SKIN FRICTION IN SHALES**

DESCRIPTION:

Methodology for the design of drilled shafts in shale.

PURPOSE: Uniform geotechnical design approach.

SUPPORTING INFORMATION:

See attached.

APPROVAL:

SIGNED: ______________________
DESIGN MANAGER

ACKNOWLEDGEMENT:
Please sign and return this copy to the Deputy Design Manager. Retain one copy for your design directive file.

SIGNED: ______________________
DISCIPLINE DESIGN MANAGER

DATE: 5/1/07

MEMORANDUM

▶ **TO:** Wayne Duryee

▶ **FROM:** Tom Cooling

▶ **SUBJECT:** Design Skin Friction and End-bearing of Drilled Shafts in Shale, I-64 Project

▶ **DATE:** 4-25-07

This memo supersedes prior memos on this subject and includes comments that you have provided. To the best of my knowledge the proposed approach is acceptable to MoDOT based on our recent communication.

As we discussed recently, the FHWA methodology for design of drilled shafts in shale as given in the Drilled Shafts Design Manual (FHWA-IF-99-025) is very conservative for shales. Consequently, we propose to design based on recent work by the Colorado DOT in the Denver area shales as noted in the attached paper. We also reviewed the thesis by Alan Miller with MoDOT. Both the CDOT and MoDOT design methods are based on results of Osterberg Cell (O-Cell) tests in clay shales.

Unconfined Compressive Strength Based on N-Values

All methods to design shafts in shale are based on the unconfined compressive strength. As we have discussed, the shale is very difficult to core and obtain testable samples. As a result, our practice in St. Louis has been to base the design of drilled shafts in shale on N-values, which are easily obtained. We also refer to the water content of the samples as an indirect index of strength. Generally, water contents of about 10% or less indicate high quality shale.

To get an idea of the relationship between Qu and N, we ran Menard Pressuremeter tests for the recent Lambert Airport Expansion project. Results of the tests are plotted on the attached relationship of Qu to N by various sources (Figure 1). The airport results compared to other correlations are as follows.

Relationships Between N value and Qu (ksf)	
Method	**Relationship**
Lambert Airport based on PMT tests in shale	Qu= 0.19N
CDOT tests based on Qu tests of samples	Qu= 0.24 N (for N ~ 50 or less)
AASHTO	Qu = 0.27 N (for clay soil)
MoDOT Bridge Manual	Qu = 0.2N (for clay soil)

The airport data are more conservative than the other methods, but close. It is also very close to the MoDOT relationship for clay soil. We propose to use the Lambert relation.

URS

Shaft Design Methods

Results of several drilled shaft methods to calculate skin friction and end-bearing are summarized on the attached Table 1. These methods are:

1. CDOT method as discussed in the attached paper
2. "Denver Rules" – An empirical method used in the Denver Shales for many years as noted the CDOT paper based on N-values only. We have used this method locally with success.
3. FHWA method as noted in the drilled shafts manual. It produces the most conservative results.
4. Alan Miller's simplified method from his thesis
5. Modified Rowe and Armitage method referenced by Alan Miller's thesis.

Note that all CDOT and Modified Rowe and Armitage are fairly close as is the Simplified Miller method for N values more than about 100. The Denver rules are least conservative, while the FHWA method is about half of the others.

We propose to follow the CDOT method which gives values that are similar to empirical values we have used locally (5 ksf skin friction, 40 ksf end bearing for shales with N >100).

Additional clarifications that we propose for use of the CDOT design procedure are as follows.

- N values will generally be back-figured from the number of inches that the SPT sampler penetrates under 50 hammer blows. For example 50 blows for 4 inches of penetration is a calculated N-value of 150 blows/ft.
- The CDOT design procedure will also be used for sandstone and siltstone as well as shale unless those materials are competent enough to obtain high quality cores in which case the cores would be obtained and tested.
- The CDOT design procedure would be limited to shafts of 6 feet or less in diameter. Larger diameter shafts will be reviewed on a case-by-case basis and may require deviation from the CDOT method.
- If coal seams are found in test borings, shafts will generally be designed to bear 5 ft+/- below the coal seam. If it is impractical to drill below the coal seam, shafts may be stopped a minimum of 5 feet or one shaft diameter above the coal seam whichever is larger.
- Shaft design will include both end-bearing and skin friction, both based on the CDOT method.

- We intend that the geotechnical engineer determine the shaft design parameters and provide these to the bridge designer. For example the geotechnical engineer may recommend the following for a typical drilled shaft.

Elevation	Typical N-value	Allowable Skin friction (ksf)	Allowable End-Bearing (ksf)
480 to 470	100	3.6	30.5
470 and below	150	4.4	37.5

We do not intend to let the bridge designer select the N value and the design parameters directly.

Field Quality Control

The level of field quality control is key in selecting factors of safety for design as well as assurance that the shafts are built as intended. In selecting the factors of safety for design (generally 2.5 as noted in Table 1), we have assumed the following field quality control and construction related issues;

- Drilled shaft construction will generally follow MoDOT 701 specifications.
- A foundation inspection hole will be drilled at each shaft and SPT samples will be obtained at 5 ft intervals to a depth of 10 feet below the bearing elevation of the shaft. Foundation inspection holes will be drilled prior to drilled shaft construction. The rig obtaining the SPT measurements will have a calibrated hammer that delivers at least 60% of the theoretical SPT hammer energy.
- The geotechnical engineer or his representative (Terracon or URS for shafts designed by each, respectively) will log the foundation inspection holes. The geotechnical engineer (Terracon or URS) will determine the top of the rock socket for design purposes and the tip elevation based on the required rock socket length shown on the plans.
- Drilled shafts in shale, sandstone, or siltstone, will be concreted the same day they are opened, or no later than 12 hours after they are opened. Shafts remaining open longer than that will be reamed to 1 ft diameter larger and one foot deeper to remove softened shale along the walls of the socket and at the bottom of the shaft.
- If the socket appears to be degrading from seepage prior to 12 hours, the contractor will be instructed to roughen the walls of the shaft with the auger or cleaning bucket creating grooves about 3 inches deep, 3 inches tall at a pitch of about 2 feet.
- The walls and bottom of the shaft will be cleaned with drilling tools (typically earth auger or bucket) to the satisfaction of the geotechnical engineer (Terracon or URS). Inspection will be done from the ground surface using mirrors or high powered lights to view the bottom of the hole. Typically the engineer will look to see the grooves in the base of the shaft cut by the tools to verify that the bottom is adequately cleaned. Based on past

experience, the shafts should be able to be cut with earth augers unless limestone stringers or lenses are encountered in which case a rock auger will be needed.

- Concrete will have a minimum slump of 6 inches and will be placed via a hopper and "elephant trunk" or concrete pump.
- Any water seeping into the shaft will be pumped to within 3 inches of the bottom of the shaft prior to concreting. We expect that most shafts will have minimal seepage.
- All shafts will be tested by Crosshole Sonic Logging (CSL) per MoDOT 701. Shafts that show anomalies will be further investigated by other methods such as coring at the engineer's recommendation (Parsons, URS, or Terracon).

Attachments:
Figure 1 –Qu vs N relationship
Table 1 – Comparison of Skin Friction and End-Bearing, Various Methods
TRB Paper Explaining CDOT method

Cc; Dave Harwood, Terracon
Vincent Gastoni, Parsons
Tony Stirbys, Parsons
Doug Cauble, URS

Figure 1 - Unconfined Compressive Strength of Shale Based on N Values

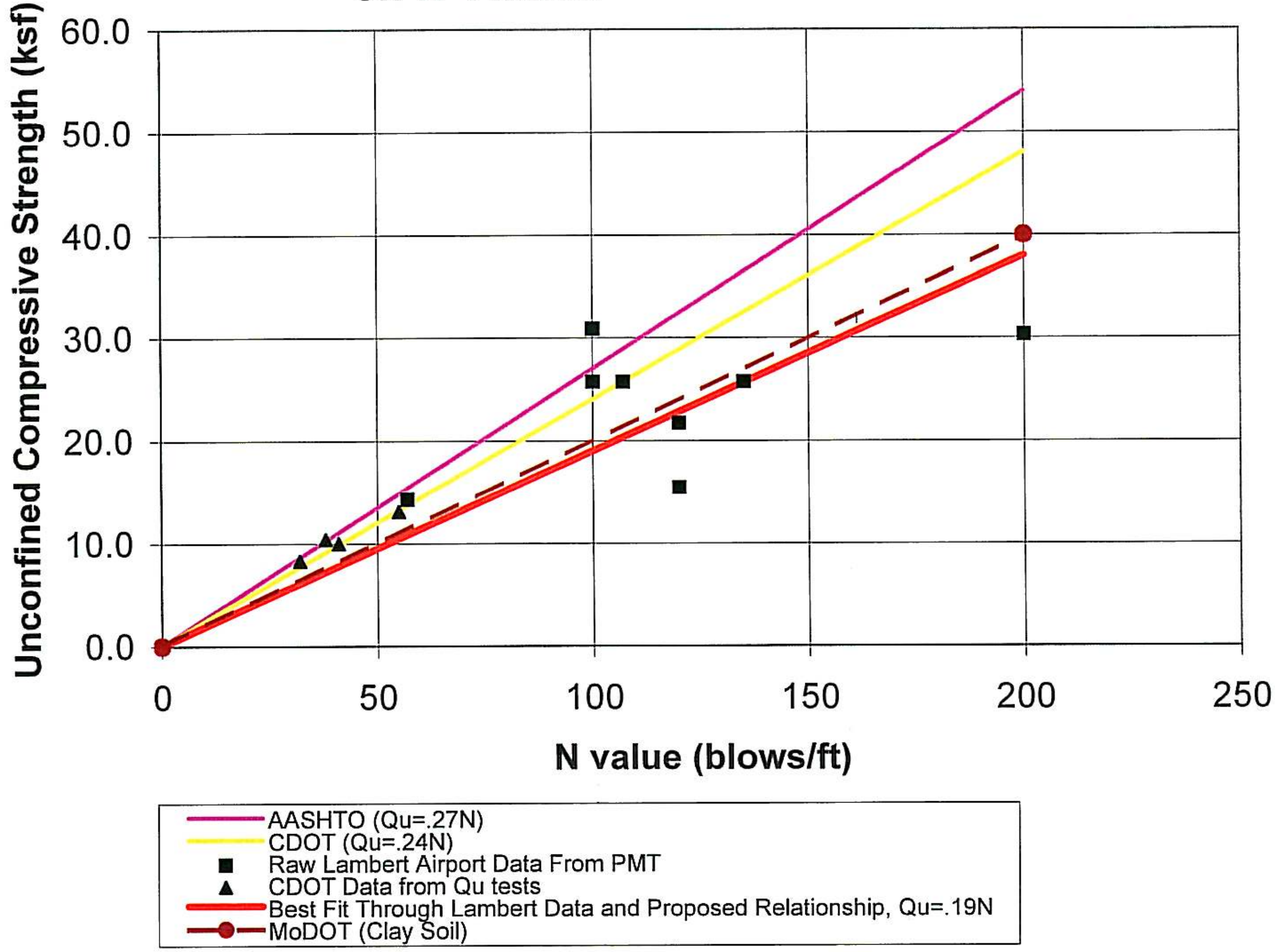

TABLE 1 - Comparison of Skin Friction and End Bearing Values, Various Methods and Proposed Relationship for Design of Drilled Shafts in Pennsylvanian Shale, I-64 Project, St. Louis, MO.

N value blows/ft	Qu (ksf) Qu=.19N	fall = Allowable Skin Friction (ksf) FHWA	"Denver Rules"	CDOT	Simplified Miller	Modified Rowe & Armitage by Miller	qall= Allowable End Bearing , ksf FHWA	"Denver Rules"	CDOT
50	9.5	0.6	2.5	**1.9**	1.1	2.3			
60	11.4	0.7	3.0	**2.2**	1.4	2.5			
70	13.3	0.8	3.5	**2.6**	1.6	2.7	(Neglect End-Bearing for N<100)		
80	15.2	1.0	4.0	**3.0**	1.8	2.9			
90	17.1	1.1	4.5	**3.4**	2.1	3.0			
100	19	1.2	5.0	**3.6**	2.3	3.2	27	50	**30.5**
110	20.9	1.3	5.0	**3.7**	2.5	3.3	29.7	50	**32.0**
120	22.8	1.4	5.0	**3.9**	2.7	3.5	32.4	50	**33.5**
130	24.7	1.5	5.0	**4.1**	3.0	3.6	35.1	50	**34.9**
140	26.6	1.7	5.0	**4.2**	3.2	3.8	37.8	50	**36.2**
150	28.5	1.8	5.0	**4.4**	3.4	3.9	40.5	50	**37.5**
160	30.4	1.9	5.0	**4.5**	3.6	4.0	43.2	50	**38.8**
170	32.3	2.0	5.0	**4.7**	3.9	4.2	45.9	50	**40.0**
180	34.2	2.1	5.0	**4.8**	4.1	4.3	48.6	50	**41.2**
190	36.1	2.3	5.0	**4.9**	4.3	4.4	51.3	50	**42.3**
200	38	2.4	5.0	**5.1**	4.6	4.5	54	50	**43.5**

Note: Factor of Safety (FS) =2 on CDOT skin friction for N values less than 90. For all other methods FS =2.5 except "Denver Rules"

FHWA skin friction assumes that the mid-depth of the shaft is at a depth of 20 feet.
FHWA values based on Qu=.27N
FHWA allowable skin friction = fall = $\alpha\phi$Qu/2.5 where α and ϕ are based on depth of shaft, slump, and RQD
FHWA allowable end bearing = Qu

Except for FHWA, Qu =.19N

For CDOT skin friction values, use fall=.037N for N<90, for N> 90 use fall=2.05(Qu)^0.5/2.5
CDOT End Bearing = Allowable End Bearing = 17 x (Qu)^0.51/2.5

Denver Rules - Allowable End Bearing =N/2 not to exceed 50 ksf. Allowable skin fricition, N/20 not to exceed 5 ksf.

Simplified Miller = Allowable skin friction = 0.3 x Qu/2.5

Modified Rowe and Armitage by Miller = Allowable skin friction = 0.4 x(Qu)^0.5/2.5 (units in Mpa)

Improvement of the Geotechnical Axial Design Methodology for Colorado's Drilled Shafts Socketed in Weak Rocks

Naser M. Abu-Hejleh, Michael W. O'Neill,
Dennis Hanneman, and William J. Attwooll

Drilled shaft foundations embedded in weak rock formations support a large percentage of bridges in Colorado. Since the 1960s, empirical methods that entirely deviate from the AASHTO design methods have been used for the axial geotechnical design of these shafts. The margin of safety and expected shaft settlement are unknown in these empirical methods. Load tests on drilled shafts provide the most accurate design and research data for improvement of the design methods. Four Osterberg axial load tests were performed in Denver on drilled shafts embedded in soil-like claystone, very hard sandy claystone, and extremely hard clayey sandstone. An extensive program of simple geotechnical tests was performed at the load test sites, including standard penetration tests (SPT), unconfined compressive strength tests (UCT), and pressuremeter tests (PMT). Information on the construction and materials of the test shafts was documented, followed by thorough analysis of all test results. Conservative equations were suggested to predict the unconfined compressive strength and mass stiffness of weak rocks from SPT and PMT data. Colorado Department of Transportation (CDOT) and AASHTO–FHWA design methods for drilled shafts were thoroughly assessed. 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 were developed. The qualifications and limitations for using these design methods are presented (e.g., construction procedure, field conditions). Finally, a detailed strategic plan to identify the most appropriate design methods per LRFD for Colorado's drilled shafts was developed.

Drilled shaft (pier or caisson) foundations support a significant percentage of bridges in Colorado. These shafts derive support by embedment in weak sedimentary rock formations (not the overburden usually neglected in the design). Drilled shafts in these weak rocks are attractive when compared with driven piles, because the boreholes in these materials are relatively stable. Further, weak rocks are generally not difficult to excavate and are typically found at relatively shallow depth. Two prevalent geologic formations for the weak rocks in Colorado are the Pierre and Denver Formations (*1*). These formations consist of weakly cemented claystone, siltstone, sandstone, and interbedded sandstone–claystone, with composition consisting of varying amounts of fine-grained to very coarse-grained sediments. According to Jubenville and Hepworth the unconfined compressive strength for the Denver Formation ranges from 6 kips per square foot (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 (2). Abu-Hejleh et al. reported unconfined compressive strengths greater than 300 ksf for these weak rocks and provided a geotechnical and geologic description of bedrock formations likely to be encountered in the Denver metropolitan area and other populated areas along the Front Range Urban Corridor in Colorado (*3*).

Thorough geotechnical design of these drilled shafts requires determination of a top load–settlement curve (q_{max}), the rock layer beneath the shaft (f_{max}), the rock layers around the shaft, the load factor and resistance factor (ϕ) in the load and resistance factor design (LRFD) method, and the factor of safety (FS) in the allowable stress design (ASD) method. Since January 1, 2000, it has been the policy of the Colorado Department of Transportation (CDOT) to incorporate the new and more accurate AASHTO LRFD method for the design of its highway structures (including drilled shafts). However, an understanding of Colorado's weak bedrock behavior and design practice has not been adequate to fully implement the new design method.

Since the 1960s, empirical methods and "rules of thumb" have been used to design drilled shafts in Colorado that are geared toward the ASD method using the results of the standard penetration test or SPT (N-value in terms of blow counts to drive 1 ft or blows per foot (bpf)). These methods entirely deviate from the AASHTO design methods that are based on the unconfined compressive strength q_u for clays and rocks (*4, 5*). However, the AASHTO design methods were developed for conditions different from those encountered in Colorado (e.g., not for the weak rocks often encountered in Colorado). O'Neill et al. (*6*) and FHWA (*7*) presented advanced design methods for a new category of geomaterials at the boundary between soil and rock called intermediate geomaterials (IGM). By definition, a clay-based IGM has 10 ksf $< q_u <$ 100 ksf, and a sand-based IGM has 50 $< N <$ 100 bpf. These design methods consider additional parameters, such as the elastic modulus of the rock mass (E_m), that can be determined from the pressuremeter test (PMT), the elastic modulus of the shaft (E_c), length (L) and diameter (D) of the rock socket, and construction information for the drilled shafts (e.g., cleanliness and roughness of the shaft, and time and methods of excavation and concrete placement). CDOT design methods and the FHWA–

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Transportation Research Record: Journal of the Transportation Research Board, No. 1936, Transportation Research Board of the National Academies, Washington, D.C., 2005, pp. 100–107.

AASHTO design methods are described later in this paper, and more details of these methods are provided by Abu-Hejleh et al. (*3*).

Load tests on drilled shafts provide the most accurate design information and research data for improvement of the design methods for drilled shafts. CDOT strategy is to correlate the measured results of a large number of load tests on drilled shafts embedded in typical weak rocks in Colorado with predictive design methods that use results of simple geotechnical tests to identify the most appropriate (*a*) geotechnical design methods to predict the ultimate axial resistance and settlement of Colorado's drilled shafts and (*b*) resistance factors needed in the LRFD method.

As part of the construction requirements for the T-REX and I-25–Broadway projects along I-25 in Denver, Colorado, four Osterberg (O-cell) load tests on drilled shafts were performed in 2002 at the I-225, County Line, Franklin, and Broadway sites. The bedrock at the load test sites represents the range of typical weak rock encountered in Denver: soil-like claystone (at I-225 and County Line) to very hard sandy claystone (Franklin) to even much harder clayey sandstone (Broadway). To maximize the benefits of this work, the O-cell load test results and information on the construction and materials of the test shafts were documented, and an extensive program of simple geotechnical tests (standard penetration tests (SPT), unconfined compressive strength tests (UCT), and pressuremeter tests (PMT)) was performed at the load test sites. Analysis of all test data, information, and experience gained in this study were used to provide (*a*) a best-fit equation to predict the unconfined strength of weak rocks from SPT and PMT data, (*b*) assessment of the CDOT and AASHTO–FHWA design methods; and (*c*) 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. A strategic long-term plan and other products to fulfill CDOT objectives previously mentioned were also developed. This work is briefly described in this paper. For more details, see Abu-Hejleh et al. (*3*).

GEOTECHNICAL INVESTIGATION AROUND THE TEST SHAFTS

Description

A CDOT drilling crew advanced at least three test holes within 10 ft of the edge of each test shaft. In the first test hole, SPT with an automatic hammer, in accordance with ASTM D1586, was performed at 5-ft vertical intervals to obtain the *N*-value (bpf).

In the second test hole, cores from the weak rock were collected using the triple- and double-wall core barrels with the wire line technique producing 5-ft core runs. The use of a continuous sampler, in which the sampler was advanced (pushed) with a hollow stem auger, in the soil-like claystone was not successful. The recovered samples were preserved and transported to the laboratory. Percentage recovery and rock quality designation (RQD) for each core run were then determined as soon as possible. Laboratory testing performed on the rock core specimens included natural moisture content and dry density, percentage passing the No. 200 sieve, and Atterberg Limits to classify the rock (e.g., clayey sandstone). On intact core specimens, UCT was performed to determine the unconfined compressive strength, q_u, and initial Young's modulus of the stress-strain curve (E_i) of intact rock.

The third test hole and the corresponding test pocket for the standard soil Menard PMT was drilled carefully. The PMT was used to measure directly the in situ (mass) elastic modulus of the rock (E_m). Several elastic deformation moduli can be calculated from linear portions of the PMT curves (*8*), including an initial modulus, *E*, a reload modulus, E_r, and an unload modulus. The PMT also was used to indirectly estimate the undrained shear strength and then the corresponding unconfined compressive strength q_u using two procedures [FHWA manual (*8*)] presented in Equations 1 and 2.

Testing Results and Discussion

For purposes of this study, clay-based geomaterials with SPT > 20 are defined as clay-based rocks, and sand-based geomaterials with SPT > 50 are defined as sand-based rocks.

The results of SPT and UCT on all weak rock layers encountered at the four load sites are summarized in Tables 1 to 3. CDOT geotechnical standards were used to describe the weak rock layers according to their standard penetration resistance as firm ($20 < N < 30$), medium hard ($30 < N < 50$), or hard ($50 < N < 80$). CDOT standards describe the bedrock for the Franklin and Broadway test shafts as very hard ($N > 80$). The measured unconfined compressive strength values ranged from 8 to 16 ksf for the claystone at the I-225 and County Line shafts, 40 to 80 ksf for the Franklin sandy claystone, and 85 to 300 ksf for the Broadway clayey sandstone. The County Line and I-225 bedrock can be classified as very stiff clays according to the AASHTO (*5*) and Canadian Geotechnical Society (*9*) manuals, and at the boundary between stiff clays and cohesive IGM according to the FHWA manual (*7*). Therefore, design methods for stiff clays are appropriate for these weathered rocks, requiring no RQD information. Indeed, no reliable cores for UCT could be obtained at the I-225 and County Line sites, indicating that any measured RQD values for these geomaterials are meaningless or questionable. In this study, the bedrock at the County Line and I-225 sites is referred to and was analyzed as "soil-like claystone," with no required information on RQD. The Franklin and Broadway bedrock is classified as rock according to AASHTO (*5*) and weak rock according to the *Canadian Foundation Engineering Manual* (*9*). The FHWA manual classifies the Franklin bedrock as cohesive IGM and the Broadway bedrock as rock. In this study, the bedrock at the Franklin site is described as "very hard sandy claystone bedrock," and the Broadway bedrock is described as "very hard clayey sandstone bedrock."

Analysis and assessment of the results of simple geotechnical tests revealed the following:

- Reliable cores were recovered from the very hard claystone and sandstone bedrock, which led to reliable unconfined compressive strength data for these rocks.
- It was very difficult to collect reliable core specimens for unconfined compressive strength tests in the soil-like claystone. The unconfined compressive strength data vary from 2.2 ksf to 10.4 ksf for the second rock layer encountered in the County Line site, and no reliable core specimens could be recovered from the first rock layer encountered in the I-225 site. Only one unconfined compressive strength test (13.1 ksf) was available for the second rock layer in the I-225 site, and it was close to the value estimated from one PM test (12 ksf). The larger strength data (13.1 ksf) were selected to be on the conservative side (i.e., the correlation factor between strength and resistance, like α, would be smaller with the larger strength data).
- The initial modulus measured from the PM test (*E*) was selected over the reload modulus (*Er*) to represent the mass Young's modulus of the rock mass (E_m) and to predict the response the shafts in the initial design range (service loads) where small strains and movements occur (*3*).

TABLE 1 Assessment of Colorado SPT-Based Design Method

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

- For indirect estimation of the unconfined compressive strength (ksf) of weak rocks from PMT results (units in ksf), the most appropriate equation for soil-like claystone is as follows:

$$q_u = 0.5(P_L - P_o)^{0.75} \tag{1}$$

and for the hard claystone and sandstone bedrock as in the Franklin and Broadway sites is

$$\frac{2(P_L - P_o)}{q_u} = 1 + \ln\frac{E_m}{1.33q_u} \tag{2}$$

where P_o is the at-rest earth pressure and P_L is the yield pressure, which corresponds to initiation of plastic deformation.

- The best conservative correlation equation between unconfined compressive strength of rock (q_u, ksf) and the SPT-N value (bpf) for soil-like claystone bedrock is

$$q_u(\text{ksf}) = 0.24N \tag{3}$$

The FHWA manual (7) recommends q_u (ksf) = 0.27 N.

- The SPT and PMT produced more consistent strength data for the soil-like claystone (and most likely in fractured weak rock) and could be made very reliable if the correlation equations suggested previously were refined and made more accurate. On the basis of these equations, the strength data for all the soil-like claystone layers encountered in the County Line and I-225 sites were either identified or finalized.
- A direct design equation relating q_{max} or f_{max} to the results of SPT and PMT should be developed for soil-like claystone rocks, but the unconfined compressive strength data provides a reliable source of strength data for design methods currently established for the very hard claystone and sandstone rocks.
- A conservative ratio between the mass stiffness of the rock and unconfined compressive strength of intact rock (E_m/q_u) equal to 150 for claystone bedrock and 100 for the hard sandstone bedrock should be used. For the soil-like claystone, the ratio [E_m (ksf)/N] can be taken as 40. For all materials, a best-fit expression can be taken as E_m (ksf) = $1{,}016q_u^{0.5}$ and for design purposes as E_m (ksf) = $600q_u^{0.5}$.

O-CELL LOAD TESTS

Description of Test Shafts

The construction of the test shafts was representative of the typical construction procedure for production shafts used in the Denver area and in Colorado in general. For each test shaft, the following information was obtained:

- Construction information (date, location, methods and timing of excavation and concrete placement, conditions of shaft wall sides and bottom),
- Materials information (concrete slump, f'_c for the concrete compressive strength, and E_c for the shaft composite Young's modulus), and
- Layout information (D is the diameter of the shaft, L is length of the bedrock socket, and depths to top of shaft, groundwater level, competent rock, and base of the shaft).

Shaft diameters ranged from 3.5 to 4.15 ft. The shafts were drilled with an auger flight at the tip of the drill rig Kelly bar. The drillers

TABLE 2 Measured Unit Ultimate Side Resistance Values and Predicted Values from FHWA and AASHTO Design Methods

	Soil-Like Claystone (I-25 at I-225 and County Line)				Very Hard Sandy Claystone (I-25 at Franklin)	Very Hard Clayey Sandstone (I-25 at Broadway)		
q_u	8.3	13.1	10	10.4	64	97	210	145
Measured f_{max}	2.6	3.6	3.1	3.4	19	17	35.1	24
Predicted from:								
AASHTO (*5*) for stiff cohesive soils, $f_{max} = \alpha q_u$, $\alpha = 0.28$ to 0.16 for soils with q_u from 8 to 36 ksf.	2.1	2.5	2.3	2.4				
Equation recommended by Turner et al. (*1*) for stiff cohesive soils, similar to AASHTO method but with $\alpha = 0.21 + 1.1/q_u$	2.8	3.8	3.2	3.3				
Horvath and Kenny method for smooth rock sockets as presented in FHWA (7) and recommended in AASHTO (*5*) for rocks with $q_u > 40$ ksf with resistance factor of 0.65, $f_{max} = 0.95\ q_u^{0.5}$	2.7-	3.4	3.0	3.1	7.6	9.4	13.8	11.4
FHWA (*7*) method based on the analysis performed by Carter and Kulhawy on load test data provided by Rowe and Armitage: $f_{max} = \mu\ q_u^{0.5}$								
1. For smooth socket, $\mu = 0.92$.					7.5	9.1	13.4	5.1
2. For regular clean rock sockets with grooves between 0.04 and 0.4 in. occurred under normal drilling, $\mu = 2.05$.					16.7	20.2	29.7	24.7
3. Rough socket, $\mu = 2.75$.					22.3	27.1	39.9	33.1
O'Neill et al. method (*6*) presented in FHWA (1999) for cohesive IGM								
1. Smooth.					5.9	7.8	16.8	11.6
2. Rough socket at displacements equal to 1% the shaft diameter.					12.8	14	32.8	21.8

NOTE: All units are in ksf.

did not add any water during drilling. When the shafts reached their intended depths at the I-225 and County Line sites (soft claystone), the sides and bottom of the hole were cleaned.

At the Franklin and Broadway sites, the slurry method was used in drilling through the overburden, followed by installing and screwing casing into the top 1 to 2 ft of the rock. After the slurry in the casing was removed, the rock socket was drilled as described earlier but with no roughening tooth for cleaning purposes. The drillers at these two sites believe that normal drilling in the very hard bedrock at the Franklin and Broadway sites creates not only clean but also naturally rough sockets, as reported in the literature (*7*).

Concrete was placed relatively slowly with a tremie pipe to keep the concrete under water and to avoid mixing the concrete with this water (e.g., 18 in. of water existed at the hole bottom in the Franklin shaft when placement of concrete started). A seating layer of plain concrete (called a "reaction socket") was pumped in the base of the shafts. Then, the reinforcing steel cage, instrumented with a diaphragm O-cell at the bottom of the cage and two sets of strain gages at two levels in the cage above the O-cell, was inserted in the wet concrete at the top of the reaction socket. The remainder of the concrete was slowly pumped by tremie pipe, and then the temporary casing was removed. The length of the reaction socket was large (6.3 ft) only for the Broadway shaft.

Load Tests

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 also were instrumented to record the upward deflection of the shaft head and upward and downward movement of the O-cell as the load was applied in increments. 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.

LOADTEST, Inc., (*10*) performed the O-cell test and provided the following test results: gross O-cell load versus upward and downward movement of the O-cell, and, from results of strain gages, unit side resistance for different zones across the test shafts versus the upward movement of the O-cell, and the equivalent top load versus settlement curve. It was assumed in the LOADTEST, Inc., analysis and in this study that the measured relations for side resistance versus

TABLE 3 Measured Unit Ultimate Base Resistance Values and Predicted Values from FHWA and AASHTO Design Methods

	Soil-Like Claystone (I-25 at I-225 and County Line)		Very Hard Sandy Claystone (I-25 at Franklin)	Very Hard Clayey Sandstone (I-25 at Broadway)
q_u	13.1	16.8	41	219
Measured q_{max}	55	53	236	318
q_{max} predicted from:				
A. Methods where q_{max} corresponds to or is very close to the full plastic mobilization of the resistance (theoretical ultimate resistance)				
1. $q_{max} = 4.5q_u$ (for very stiff clays) with resistance factor of 0.55 (*5*). AASHTO (*4*) recommends $4.3q_u$ for intact claystone and sandstone, and $5q_u$ for intact sandstone.	59	75.6	185	986
2. Canadian Foundation Manual for rocks and IGM in which layering in the geomaterial is horizontal. This method was presented in FHWA (7) and referenced by AASHTO (*5*) with resistance factor of 0.5. For intact rock (joints are closed), $q_{max} = 4.08q_u$ when $L/D > 6$.	45	48	167	893
B. Method for rocks where q_{max} correspond to the occurrence of fracturing in the rock				
$q_{max} = 2.5\ q_u$ (FHWA method for massive and cohesive intermediate geomaterials or rock, *7*)	33	42	103	547
C. Methods where q_{max} is defined by displacement criteria				
1. O'Neill et al. (*6*) method for cohesive intermediate geomaterials at displacements equal to 5% the shaft diameter.			145	271
2. Zhang and Einstein method presented in FHWA manual (*7*) $q_{max} = 21.4(q_u)^{0.51}$			142	334

NOTE: All units are in ksf.

upward movement (as in an O-cell test) in any zone were equivalent to side resistance versus downward movement in that zone. The Broadway site's production shafts were redesigned on the basis of the load test results. The ultimate side resistance was increased from as low as 4.8 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.

Analysis Methodology of O-Cell Load Test Data

For research 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) versus unit side resistance (f) until the maximum unit side resistance f_{max} is reached for all weak rock layers encountered across the test shaft. The compressibility of the high capacity Franklin and Broadway shafts and presence of a reaction socket under the O-cell were considered in extracting the f-w and q-w curves. There are many sources for errors in estimating the side resistance based on strain gages. Therefore, it was deemed more accurate to estimate the average shaft side resistance along the rock socket above the O-cell, as no data were from strain gages. This approach for estimating side resistance was recommended by the ongoing NCHRP Project 21-08.

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 (*3*). The adopted definitions of ultimate resistance in this study (which could be adjusted in the future when more data become available) are as follows:

- For the soil-like claystone (County Line and I-225 sites), f_{max} and q_{max} 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} corresponds to displacement of 5% of the shaft diameter, but not to exceed 3 in., and f_{max} corresponds to a displacement of 1% of the shaft diameter, but not to exceed 0.6 in. The 3- and 0.6-in. 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 (level out) until a movement of 5% of the shaft diameter occurred.

Estimated q_{max} and f_{max} were used 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 the base area and side areas of shaft in the rock socket, respectively. Assuming an FS of 2 for the load test results, the allowable design base and side unit resistance values and loads are determined, respectively, as $q_{all} = q_{max}/2$, $f_{all} = f_{max}/2$, and $Q_{all} = Q_{max}/2$. The procedure adopted in this study to construct the load–settlement curve for rigid and compressible shafts from f-w and q-w load transfer curves is similar to the procedure described in the LOADTEST, Inc., test reports and is based on the same assumptions (*10*). This curve is needed by the structural engineer to ensure that the settlements at service loads are smaller than the tolerable settlements, established as 0.65 in. for the T-REX and Broadway projects.

It would be of interest to get load transfer curves and an axial load–settlement curve for drilled shafts as a function of the results of simple geotechnical tests, as was done for laterally loaded shafts using p-y curves. 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), where Q_d is the resistance load of the shaft at a settlement of 0.01D. This paper defines q_d and f_d, needed to calculate $Q_d = A_b q_d + A_s f_d$, respectively, as the unit base resistance and unit side resistance that correspond to a settlement of 0.01D. At a settlement of 0.05D, most of the base and side resistances for soil-like claystone rocks, needed to compute Q_{max}, are mobilized. The design methods recommended in this paper present the best correlation equations between the measured q_{max}, f_{max}, q_d, and f_d values and the measured results from the simple geotechnical tests (N from SPT or q_u from strength tests). Then, the developed approximated load–settlement curves can be adjusted for elastic compression of the shafts.

Results of O-Cell Tests and Discussion

All the load test results are summarized in Tables 1 to 3. The full side and base plastic resistance were almost mobilized by the end of the load tests for both the County Line and I-225 shafts (with soil-like claystone). Therefore, and to be conservative, the measured side and base resistances at the end of the O-cell load tests for these shafts were called f_{max} and q_{max}, respectively.

For the Franklin and Broadway production shafts, the side shear resistance did not reach ultimate conditions at the end of the tests as defined before, but did start to show signs of being hyperbolic (under relatively small movements of 0.13 in. in the Franklin shaft to 0.33 in. in the Broadway shaft). The extrapolated side resistance at side movement that corresponds to 1% of the shaft diameter was called the ultimate side resistance, f_{max}. The measured unit base resistance values at settlement that correspond to 5% of the shaft diameter for the Broadway and Franklin shafts were called q_{max}. At the end of the O-cell load test on Broadway shaft, the base resistance continued to develop and increased almost linearly with settlement without any sign of yielding. Similar response was noticed in the corresponding PMT conducted in the very hard rock layer under the base of the Broadway shaft. These observations suggest that the base resistance load measured at the end of the O-cell load test on the Broadway shaft was far from the true ultimate base resistance load of the shaft. For the Franklin shaft, the base resistance almost leveled out after reaching the yield point and indicated that the base resistance load measured at the end of the O-cell test for the Franklin shaft was very close to the ultimate true base resistance load of the shaft.

The lessons learned from the results of load tests are as follows:

- Design equations to predict f_{max} and q_{max} are discussed in subsequent sections. The best-fit design equations to determine the additional design parameters required in construction of an approximated load–settlement curves are as follows:
  - f_d (ksf) = 0.06N and q_d = 0.42N for soil-like claystone,
  - f_d (ksf) = f_{max}, and q_d (ksf) = 1.7q_u (ksf) for the very hard sand claystone as in the Franklin site, and
  - f_d (ksf) = f_{max} and q_d (ksf) = 0.32q_u (ksf) for the very hard clayey and well-cemented sandstone as in the Broadway site.
- The settlements of all shafts that correspond to the design loads (w_{all}) ranged from 0.25 in. for the County Line and I-225 shafts to 0.5 in. for the Broadway shaft, smaller than the tolerable settlement of 0.65 in., suggesting that the design loads calculated on the basis of the strength limit control the design. For the Broadway shaft, this settlement was relatively large (0.5 in.). If the ultimate resistance for the very hard sandstone at the Broadway site was selected to correspond to higher displacements than those used in this study, the settlements that will be developed at the design loads could exceed 0.65 in. and the serviceability limit would control the design. This requires accurate information on the load–settlement curve, which is very difficult to obtain.
- In the soil-like claystone at County Line and I-225 sites, 70% of the resistance to working loads is provided by means of side resistance. In the very hard claystone and sandstones at, respectively, Franklin and Broadway sites, 90% to 95% of the resistance to working loads is provided by means of side resistance.

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 the ASD method with no information on expected settlement of drilled shafts. In the Colorado SPT-Based (CSB) design method, the allowable base resistance in kips per square foot is taken as q_{all} (ksf) = q_{max}/FS = 0.5N, and the allowable side resistance is taken as f_{all} (ksf) = f_{max}/FS = N/20. With the lack of information on the proper factor of safety (FS) embedded in the CSB design method, an 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 on SPT-N values, is uniformly applied to both cohesive and cohesionless weak rocks and to stronger rocks. Conversely, different design methods are used for different geomaterials (soils and rocks) in the AASHTO and FHWA (*4, 5, 7*) design manuals. Because the CSB design method is rather crude, most practitioners limit the allowable base resistance for geomaterials with $N > 100$ to approximately 50 ksf (ultimate to 150 ksf) and allowable side resistance to 5 ksf (ultimate 15 ksf).

Table 1 lists the measured f_{max} and q_{max} and the predicted values with the CSB design method, together with the FS calculated as the measured ultimate resistance from the load test divided by the predicted (from CSB method) allowable resistance. The tabulated results suggest the following:

- 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} = 0.5N) are very close to those measured from the load tests (q_{all} = 0.46N).
- The CSB side resistance design method for the soil-like claystone resulted in an FS of less than 2 but greater than 1, ranging from

1.3 to 1.8. This suggests that this design method worked well with less than a normal FS of 2. The main reasons for this low FS not being reflected in the performance of structures that are now in service could be any one or any combination of the following factors:

– FS for side resistance > 1.3,

– FS around 2 for base resistance,

– The very small settlements that would occur for soil-like claystone under the design loads, (expected to be less than 0.3 in. on the basis of the results of load tests), and

– The applied design loads that are often overestimated and include live loads that have yet to be applied.

- Conversely, 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 FSs 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 is appropriate and will be very cost-effective.
- Analysis of load tests performed by Turner et al. indicates very low FSs in the side resistance design methods, in the range of 0.8 to 1.6, lower than observed in this study (*1*). Turner et al. assumed f_{max} to correspond to displacements of 0.5 in., but in the current study, f_{max} for soil-like claystone corresponded to (or are very close to) the true side resistance, which occurs at greater displacements.

The updated CSB (UCSB) design method is recommended for soil-like claystone as $q_{max} = 0.92N$ ($q_{all} = 0.46N$) and $f_{max} = 0.075N$ ($f_{all} = 0.037N$). These are best-fit equations. The predicted allowable design resistance values and FS from this method also are listed in Table 1. The UCSB method will produce an FS 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 use high FSs, ranging from 2.3 to 3. It is recommended to use the UCSB design method with relatively smaller FS than the AASHTO method because (*a*) innumerable structures designed in Colorado over the last 40 years with the CSB design method have demonstrated excellent short- and long-term performance; (*b*) it is more cost-effective than the AASHTO and FHWA strength-based design method that uses a higher FS; (*c*) the use of SPT-based design is commonplace in Colorado; and (*d*) it is more consistent to obtain SPT data than unconfined compressive strength data in soil-like claystone geomaterial. FHWA was not intended to be a definitive design guide—only a default in the event that better local and regional information were not available, which with this study will have been developed for the Colorado weak rock formations. However, it should be realized by Colorado geotechnical engineers that the FS of the UCSB is close to 2 (resistance factor of 0.75), not 3 as currently assumed by CDOT engineers for the less conservative CSB method.

Assessment of FHWA and AASHTO Design Methods

For the soil-like claystone, the influence of discontinuities is not important, because these materials will be analyzed as soils. For the very hard claystone and sandstone bedrock, large RQD values were measured. Thus, the influence of discontinuities is very small and can be neglected. Therefore, in the side resistance design methods presented in the next section, it was assumed that the rock is intact, and the rock is massive with an insignificant number of joints or the joints are closed and spaced more than 10 ft in the base resistance design methods presented below.

Tables 2 and 3 describe the FHWA and AASHTO design methods and list the predicted ultimate side and base resistance values from these methods and the measured values from the O-cell load tests.

Side Resistance Design Methods

- For the soil-like claystone, the methods recommended by Turner et al. (*1*) and Horvath and Kenny, as presented in the FHWA manual (*7*), provided excellent predictions of f_{max}, better than the method recommended by AASHTO for stiff clays.
- A rough-sided rock socket could have three times the side resistance capacity of a smooth-sided rock socket, indicating that side resistance is strongly influenced by socket roughness. Rough sockets could be achieved through normal drilling or artificially by the use of shear rings (*7*). The Carter and Kulhawy method as presented in the FHWA manual (*7*), $f_{max} = \mu q_u^{0.5}$ where μ is the coloration factor, allows for three levels of roughness: $\mu = 0.92$ for smooth socket, 2.05 for intermediate roughness level most likely under normal drilling, or 2.7 for high roughness level, most likely with the use of artificial grooving. The O'Neill et al. method (*6*) allows for smooth or rough socket but the level of roughness is not accounted for in the analysis. The O'Neill method was developed for roughness patterns as observed in auger-cut clay-shales, expected to be close to the intermediate roughness level of Carter and Kulhawy.
- For the very hard claystone and sandstone bedrock, the Carter and Kulhawy and O'Neill methods for rough sockets provide reasonable predictions of f_{max}. The results in Table 2 suggest an intermediate roughness level for the Broadway and Franklin rock sockets. This roughness for both shafts was generated under normal drilling procedures. Indeed, the drillers of the Franklin test shaft indicated that the normal drilling in the blocky bedrock at the Franklin site created a very rough socket.

Base Resistance Design Methods

- The predictions for q_{max} vary significantly among different base resistance design methods because of differences in the definition of q_{max} in these methods (Table 3). For example, for the Broadway test shaft, if the 1-in. displacement criterion is selected to define ultimate resistance as suggested in the FHWA manual (*7*), q_{max} will be 128 ksf. If the 0.05*D* displacement criterion is selected, q_{max} will be 318 ksf, although the true short-term base resistance for massive cohesive rock could be higher than 900 ksf (as estimated using $q_{max} = 4.5q_u$).
- Conservative predictions for q_{max} for the soil-like claystone (I-225 and County Line) and the very hard claystone bedrock (Franklin site) can be obtained from the Canadian design method, which is endorsed by AASHTO LRFD with resistance factor of 0.5 (*7*).
- The q_{max} for the very hard sandstone bedrock was well predicted with the O'Neill method (*6*) and the Zhang and Einstein method, as presented in the FHWA manual (*7*). The Canadian design method or the fracture-based design equations ($q_{max} = 2.5q_u$) are not appropriate for very hard rocks with $q_u > 200$ ksf, because ultimate base resistance should be defined on the basis of displacement (0.05*D*), not strength to limit the shaft settlements at service loads.

CONCLUSIONS AND RECOMMENDATIONS

These conclusions and 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 this paper and presented in more details by Abu-Hejleh et al. (*3*).

This paper summarizes and analyzes the results of four axial O-cell load tests on drilled test shafts and simple geotechnical tests (SPT, UCT, and PMT) performed near the test shafts and documents the materials, layout, and construction information of the test shafts. The weak bedrock at the load test sites represents the typical range encountered in Colorado: soil-like claystone to very hard sandy claystone to very hard clayey sandstone. It is very difficult to collect reliable core specimens for UCT in the soil-like claystone (and in the fractured weak rock), but it is possible in the harder and massive claystone–sandstone. The SPT and PMT produced more consistent and reliable strength data for the soil-like claystone than UCT. On the basis of the limited amount of data collected in this study, best-fit equations to predict the unconfined strength of soil-like claystone rocks from SPT and PMT data and strength of harder claystone–sandstone rocks from PMT data are presented.

The CSB design method for drilled shafts embedded in soil-like claystone resulted in an FS that was less than 2 but greater than 1 (ranging from 1.3 to 1.8) and was found to be very conservative in the very hard claystone–sandstone. The UCSB method developed in this study is recommended for the soil-like claystone: q_{max} (ksf) = $0.92N$, f_{max} (ksf) = $0.075N$, with FS of 2 or resistance factor (required in the LRFD method) of 0.75. Justification for the use of this relatively low margin of safety in the UCSB method is provided.

For the harder claystone and sandstone shale bedrocks, the following design methods described in the FHWA manual (*7*) should be considered. For the very hard sandy claystone as at the Franklin site, the Canadian design method is recommended to estimate the unit ultimate base resistance (Table 3) with a resistance factor of 0.5. For the very hard clayey sandstone, such as at the Broadway site, a modified and somewhat more conservative form of the Zhang and Einstein design method, that is, $q_{max} = 17(q_u)^{0.51}$ with resistance factor of 0.55, is recommended. For these two types of bedrocks, the Carter and Kulhawy design method, that is, $f_{max} = 2.05\ q_u^{0.5}$ (assuming intermediate roughness level of the shaft hole sides, Table 2) with resistance factor of 0.55 is recommended. The consideration of these design methods for the harder claystone–sandstone shales is expected to lead to significant construction savings and to offset any additional testing costs. The paper also describes a method to construct an approximate curve for the load settlement curve as a function of SPT-N values for soil-like claystone and the UCT strength data for the harder claystone–sandstone. For shafts with different diameters or lengths, see Abu-Hejleh et al. for recommendation to scale the presented design recommendations (*3*).

More load test data are needed to fulfill CDOT's objective to identify the most accurate and feasible LRFD methods for geotechnical axial design methodology for Colorado's drilled shafts embedded in various weak rock types. For this purpose, Abu-Hejleh et al. developed a strategic plan for CDOT that requires the assembly and analysis of a database that contains the results of old and new load tests on drilled shafts; results of simple geotechnical tests at the load test sites; and documentation of the materials, layout, and construction of test shafts, as described in this paper (*3*). The plan provided complete details for planning and conducting new load tests on drilled shafts [purpose and promotion; location and number of load tests; details of simple geotechnical testing at the load test sites; features, limitations, costs, and types of load tests; type of test shafts (production or sacrificial); design of O-cell load test and layout of test shafts; potential savings; construction information and data and information that should be collected; and analysis procedure]. In addition, detailed recommendations for improvement of the geotechnical subsurface investigation procedures and for construction of drilled shafts are furnished (*3*). This will ensure uniformity, improve accuracy of CDOT and Colorado practices, and standardize the work performed at the load test sites.

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